PROCEEDINGS

INTERNATIONAL CONFERENCE ON CIVIL ENGINEERING FOR SUSTAINABLE DEVELOPMENT

23-24 March 2012

ICCESD

ICCESD 2012

Organized By

Department of Civil Engineering
Khulna University of Engineering & Technology
Khulna, Bangladesh

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Khulna, Bangladesh

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Khulna-9203, Bangladesh

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Department of Civil Engineering
Khulna University of Engineering & Technology (KUET)
Khulna-9203, Bangladesh
Tel: 880-41-769468, Ext. 200, Fax: 880-41-774780
E-mail: iccesd2012@gmail.com
Website: www.kuet.ac.bd/iccesd2012

ISBN: 978-984-33-4246-1 (Printed Proceedings of Extended Abstract)
978-984-33-4247-8 (CD-ROM proceedings of Full Paper)

200 Copies
Full papers are available in CD-ROM attached at the last page of this printed proceedings

Cover Design
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Khulna University of Engineering & Technology, Bangladesh

Page Layout
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Chairman of the Conference

Prof. Dr. Md. Keramat Ali Molla
Department of Civil Engineering
Khulna University of Engineering & Technology, Bangladesh

Secretary of the Conference

Dr. Md. Shahjahan Ali
Associate Professor
Department of Civil Engineering
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International Advisory Board

Scientific & Technical Committee

Organizing Committee

Preface

We are pleased to organize the International Conference on Civil Engineering for Sustainable Development (*ICCESD-2012*) for the first time of its kind in the Department of Civil Engineering of Khulna University of Engineering & Technology (KUET), Khulna, Bangladesh. KUET is one of the leading public universities in Bangladesh, which is located at the southwestern coastal area. The overall goal of this conference is to promote applied research and professional activities relevant to different branches of Civil Engineering for the sustainable development of Bangladesh to face the great challenges of the 21st century.

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We hope that this initiative will be continued in the next editions of this conference.

On behalf of the organizing committee,

Prof. Dr. Md. Keramat Ali Molla
Conference Chair
ICCESD-2012
Department of Civil Engineering
Khulna University of Engineering & Technology (KUET)
Khulna-9203, Bangladesh

Conference Program Outline for ICCESD-2012

Collection of Conference Kit:

March 22, 2012 (from 3:00~6:00 PM)
Venue: Civil Engineering Building, KUET

Day 1: March 23, 2012 (Friday)

Time	Venue	Events
8:00-8:30	KUET Auditorium	Reporting of Registered Participants
8:30-9:15	KUET Auditorium	Inaugural Ceremony of ICCESD-2012
9:15-10:45	KUET Auditorium	Keynote Session: Keynote Speech A: Prof. Dr. Ainun Nishat, VC, BRAC University Keynote Speech B: Prof. Dr. A. M. M. Safiullah, VC, AUST Keynote Speech C: Prof. Dr. Monjur Hossain, KUET
10:45-11:15	KUET Auditorium	Refreshment
11:15-12:45	CE Building	Parallel Technical Session-I (1st, 2nd & 3rd)
12:45-14:30	CE Building	Friday prayer & Lunch
14:30-16:30	CE Building	Parallel Technical Session-II (4th, 5th & 6th)
16:30-17:00	CE Building	Coffee Break
17:00-19:00	CE Building	Parallel Technical Session-III (7th, 8th & 9th)

Day 2: March 24, 2012 (Saturday)

Time	Venue	Events
8:30-10:30	CE Building	Parallel Technical Session-IV (10th, 11th & 12th)
10:30-11:00	CE Building	Coffee Break
11:00-13:00	CE Building	Parallel Technical Session-V (13th, 14th & 15th)
13:00-14:15	CE Building	Lunch
14:15-16:15	CE Building	Parallel Technical Session-VI (16th, 17th & 18th)
16:15-16:45	CE Building	Coffee Break
16:45-18:45	CE Building	Parallel Technical Session-VII (19th, 20th & 21st)
19:00-22:00	Hotel Castle Salam	Closing Ceremony and Conference Dinner

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TECHNICAL SESSIONS

1st Technical Session-March 23, 2012 — Environmental Engineering – I — Venue: Seminar Room A
Session Chair: Prof. Dr. Quazi Hamidul Bari, KUET

Time	Paper ID	Paper Title	Authors Name
11:15~11:30	ID 112	Forest – A Natural Defense to Counteract the Climate Change in Bangladesh **pp. 001-002**	S M H Uddin, M M Rahman, M B H Sikder and M M Rahman
11:30~11:45	ID 182	Consideration of Integrated Influence Scenario of Climate Change and Air Pollution at City level to frame out Urban Policy and Strategy **pp. 003-004**	M S Uddin
11:45~12:00	ID 244	The Impact of Climate Change on Agriculture in Bangladesh **pp. 005-006**	S Islam, U Klönne, L Allesson, M E Hoque and M M Anannya
12:00~12:15	ID 300	Development of Crop Yield Prediction Model For Bangladesh: A Way to Food Security in Context of Climate Change **pp. 007-008**	S Shaika, R M Mafizur and P Hasan
12:15~12:30	ID 312	Impacts of Climate Change Scenarios on Crop Simulation (Boro Cultivation) in Bangladesh using DSSAT **pp. 009-010**	T Afrin, R Rumman and F Warda
12:30~12:45	ID 316	Adapting to Climate Change in Urban Areas of Bengal Delta: A Case Study on Khulna City **pp. 011-012**	A Pervin, D M Khan, S M M Rahman, A Zaman and M M Hossain
12:45-14:30		**Friday prayer & Lunch**	

2nd Technical Session-March 23, 2012 — Transportation Engineering – I — Venue: Seminar Room B
Session Chair: Prof. Dr. Hasib Mohammed Ahsan, BUET

Time	Paper ID	Paper Title	Authors Name
11:15~11:30	ID 104	Experimental Behavior of Bituminous Mixes with Waste Concrete Aggregates **pp. 187-188**	M A Sobhan and T U Ahmed
11:30~11:45	ID 107	Effects of Waste Polyethylene on Bituminous Mixes **pp. 189-190**	M N Rahman, M A Sobhan and T U Ahmed
11:45~12:00	ID 105	Effect of Waste Concrete Aggregates on the Compressive Strength of Bituminous Mixes **pp. 191-192**	M A Sobhan and T U Ahmed
12:00~12:15	ID 206	Effect of Brick Chips as Coarse Aggregate in Bituminous Mixes **pp. 193-194**	H M Rasel, M A Sobhan, M N Rahman and S M A Aman
12:15~12:30	ID 231	Effect of Fillers on Bituminous Paving Mixes: An Experimental study **pp. 195-196**	A Rahman, S K Adhikary, Q S Hossain and S A Ali
12:30~12:45	ID 314	Development of Paratransit as a Sustainable and Effective Mode of Transport - A Study Based on Dhaka City **pp. 197-198**	H B Hasan, K M and S Munira
12:45-14:30		**Friday prayer & Lunch**	

3rd Technical Session-March 23, 2012 — Geotechnical Engineering – I — Venue: Seminar Room C
Session Chair: Prof. Dr. A M M Safiullah, VC, AUST

Time	Paper ID	Paper Title	Authors Name
11:15~11:30	ID 115	Liquefaction Characteristics of Infilled Soil at Banasree in Dhaka City **pp. 289-290**	P Saha, A R M F Uddin and A Hasnat
11:30~11:45	ID 146	Study on the Locally Available Clayey Soils for the Preparation of Liner **pp. 291-292**	S R. Saha and M Alamgir
11:45~12:00	ID 147	Model Study to Improve Lateral Load Resistance of Piles in Sand using Geotextile **pp. 293-294**	M A Sayeed and Shubhra Pk
12:00~12:15	ID 158	Model Study on the Behavior of Pile in Sand Subjected to Lateral Load **pp. 295-296**	Shubhra Pk., M A Sayeed and M A Rahaman
12:15~12:30	ID 165	Effect of Anchor Size and Cable Diameter on Anchored Earth Retaining System **pp. 297-298**	M S Alam, M Al - Amin and N Sakib
12:30~12:45	ID 181	Evaluation of Peak Liquefaction Potential using Cone Penetration Test **pp. 299-300**	R K Mazumder, E M Murad and M A Ansary
12:45-14:30		**Friday prayer & Lunch**	

4th Technical Session-March 23, 2012		Environmental Engineering – II	Venue: Seminar Room A
Session Chair: Prof. Dr. Md. Saiful Islam, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 131	Quality of Treated Water in WTP at KUET Campus **pp. 013-014**	M Akter, K M Hassan, and Q H Bari
14:45~15:00	ID 133	A Case Study on Arsenic Problem at Hariarampur Village, Bagha, Rajshahi **pp. 015-016**	M Ahmeduzzaman and B Pramanik
15:00~15:15	ID 310	Complete Decoloulization of Reactive Dye by Combination of Coagulation Adsorbtion-Membrane Filtration: A Hybrid Study **pp. 017-018**	M ZB Mukhlish,T K Deb, S Majumder, S Y Sony, M R Karim
15:15~15:30	ID 137	Assessment of Water Quality from Different Sources in Khulna District **pp. 019-020**	M M Rahman, M Rahman, T Tamanna and S K Biswas
15:30~15:45	ID 157	Sustainability of Ecosan Toilets in Waterlogged Areas: A Rural Perspective in Bangladesh **pp. 021-022**	S R Saha, K M Hassan, M M Hussain and Q H Bari
15:45~16:00	ID 135	Perfluorinated Chemicals-a New Class of Global Pollutants of the World **pp. 023-024**	B Pramanik, M Ahmeduzzaman, S Fatihah and S Zarin
16:00-16:15	ID 162	Assessment of Pharmaceutical Industries in Bangladesh - An Effective Step towards the Achievement of Environmental Sustainability **pp. 025-026**	R R Wahi, K Mahmud and S Rahman
16:15-16:30	ID 322	Non-Descript Lands in India : An Alternative Ecosystem for Seed Production **pp. 027-028**	M M Adhikary, S K Acharya and A B Sharangi
16:30-17:00	**Coffee Break**		

5th Technical Session-March 23, 2012		Water Resources Engineering – I	Venue: Seminar Room B
Session Chair: Prof. Dr. M. Monowar Hossain, Executive Director, IWM			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 124	Physical Model Studies to Support the Design of Main Spillway of the Proposed Ganges Barrage **pp. 123-124**	P Kanungoe, A K M Ashrafuzzaman, M N I Siddique and Q A Sayeed
14:45~15:00	ID 138	Temporal and Spatial Distribution of Suspended Sediment Concentration In Bhairab River **pp. 125-126**	M S Ali, T Mahjabin, M B Hossen and M Abdullah-Al-Imran
15:00~15:15	ID 321	Effects of Stream barb on Straight Channel Bed Configuration: Numerical Simulation **pp. 127-128**	S A A M Hossain, M J Uddin and M S Ali
15:15~15:30	ID 153	A Scenario-based Holistic Approach for Multipurpose Reservoir Operation Considering EFR, Irrigation and Hydropower Demand: A Case Study **pp. 129-130**	S K Adhikary, S S Atef, A D Gupta and M S Babel
15:30~15:45	ID 144	Surge Phenomena and Factors Influencing Storm Surges **pp. 131-132**	N C Sarkar, A Islam and S K D Sarma
15:45~16:00	ID 145	Tsunami Hazard Assessment in the Northern Bay of Bengal along the Bangladesh Coast **pp. 133-134**	MA Hussain, MSA Khan, A Haque, ZH Khan and MMZ Ahmed
16:00-16:15	ID 218	Modelling Proposed Water Distribution Network for Water Supply in Lohagora Pourashava of Narail District, Bangladesh **pp. 135-136**	S K Adhikary and T Chaki
16:15-16:30	ID 140	Numerical Simulation of Dead Zone flows in an Open Channel with a Side Cavity and Sudden Enlargement **pp. 137-138**	M S Ali, T Mahjabin and S J Ria
16:30-17:00	**Coffee Break**		

6th Technical Session-March 23, 2012		Structural Engineering – I	Venue: Seminar Room C
Session Chair: Prof. Dr. Md. Monjur Hossain, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 119	Performance of Adobe Blocks Randomly Reinforced with Natural Fibers **pp. 225-226**	M Z I Bhuiyan, M S Islam, M F A Kayser and M M Rahman
14:45~15:00	ID 128	Study the Role of Building Setback and Height, in terms of Indoor Temperature due to the Direct Component of Solar Radiation on west façade of Residential Buildings: Comparing 1996 and 2008 'Imarat Nirman Bidhimala' **pp. 227-228**	I Rahman
15:00~15:15	ID 129	Assessment of Seismic Vulnerability using Different Site Conditions **pp. 229-230**	M A Rahman and M S Ullah
15:15~15:30	ID 130	Sustainable Development in Drift Control of Tall Buildings Due to Wind Load: Critical Analyses of the Structural Systems **pp. 231-232**	M M Islam, A Siddique and A Murshed
15:30~15:45	ID 150	Comparison of Creep Behavior of Concrete Made by Brick Chips & Stone Chips in Bangladesh **pp. 233-234**	M S Arefin and M Alin
15:45~16:00	ID 187	Damage Identification in RC Column using Long-gage FBG Sensors **pp. 235-236**	N H M K Serker and Z Wu
16:00-16:15	ID 155	Stress Distribution of CFRP Strengthened Steel Hollow Sections under Tension **pp. 237-238**	K Shahanara and S Fawzia
16:15-16:30	ID 319	Optimization of Column **pp. 239-240**	A A Mamun and M S Bari
16:30-17:00	**Coffee Break**		

7th Technical Session-March 23, 2012		Environmental Engineering – III	Venue: Seminar Room A
Session Chair : Prof. M. M. Adhikary, Former VC, BCKV, India			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 103	Physico-chemical Characteristics of Surface Water in the Magura Municipality Area, Bangladesh **pp. 029-30**	F Akter and A K Azad
17:15~17:30	ID 302	Membrane Bioreactor Process using a Simple Ceramic Filter for Wastewater Reclamation and Reuse **pp. 031-032**	M M Hasan, M Shafiquzzaman and J Nakajima
17:30~17:45	ID 109	The Hunger, Poverty and Silence: The Invisible Bond in Social Ecology **pp. 033-034**	S Bera and S K Acharya
17:45~18:00	ID 110	KVK: An Impact Study on Transforming Social Ecology in Odissa, India **pp. 035-036**	B K Mohanty, S Bera and S K Acharya
18:00~18:15	ID 111	Estimation of Social Entropy: The Dictum of Neo Modernism in Agricultural Knowledge Environment in The Context of Global Warming **pp. 037-038**	S K Acharya, B Bhattacharya and S Bera
18:15~18:30	ID 117	Problems & Prospects of Solar Energy Usage in Bangladesh: An exploratory Study on Sylhet **pp. 039-040**	K R Tanvir
18:30-18:45	ID 122	Icthiofaunal (Fish) Biodiversity: Estimation of Erosion through Participatory Learning Action **pp. 041-042**	S Bera, S K Acharya and A Biswas
18:45-19:00	ID 313	Development of GIS Based Decision Support Database for Improved Healthcare Waste Management in Khulna City **pp. 043-044**	I M Rafizul, M S Islam, S Azim and M Alamgir
End of the Day			

8th Technical Session-March 23, 2012		Water Resources Engineering – II	Venue: Seminar Room B
Prof. Dr. M. Monowar Hossain, Executive Director, IWM			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 114	Study on Drainage Congestion in Dhaka-Narayanganj-Demra (DND) Project Area **pp. 139-140**	M A Rahman and M M Mahmood
17:15~17:30	ID 191	Sustainability of Peripheral Rivers of Dhaka as Sources of Water Supply **pp. 141-142**	M Khan and M J Haider
17:30~17:45	ID 271	Cost Effective Bank Protection for Halda River **pp. 143-144**	M N N Khan, M S Haider and A Akter
17:45~18:00	ID 245	The Impact of Sea-level rise on Mangrove Ecosystems in Bangladesh – A Review **pp. 145-146**	K Ekstedt, A Malekahmad, C Rimberth, S Islam, M E Hoque and T Tamanna
18:00~18:15	ID 260	Application of a Simple Stochastic Model for Predicting Groundwater Table Fluctuations in a Shallow Aquifer of Bangladesh **pp. 147-148**	S K Adhikary and M MRahman
18:15~18:30	ID 205	Spatial and Temporal Variation of Surface Water Quality of Sundarban Reserved Forest **pp. 149-150**	S Moazzem and R Mamtaz
18:30-18:45	ID 217	Delineating Aquifer System Delineation and Groundwater Resources Assessment in Lohagara Municipality of Narail District, Bangladesh **pp. 151-152**	S K Adhikary, T Chaki, M M Rahman and A D Gupta
18:45-19:00	ID 327	Development of an Innovative Water Treatment Technique Using Solar Still cum Sand Filter **pp. 153-154**	M A S Khan and K M Hassan
End of the Day			

9th Technical Session-March 23, 2012		Structural Engineering – II	Venue: Seminar Room C
Session Chair : Brig. General Dr. Habibur Rahman Kamal psc, Chairman, KDA			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 167	Prospect of Bangladeshi Fly Ash in Cement Production **pp. 241-242**	M M Islam and M S Islam
17:15~17:30	ID 169	Performance Based Fire Safety Management in Commercial Mixed Use Buildings of Bangladesh **pp. 243-244**	A Sarraz and M A I Chowdhury
17:30~17:45	ID 170	Analytical Study on Slender Partially Encased Composite Columns Under Eccentric Loading **pp. 245-246**	S Ali and M Begum
17:45~18:00	ID 326	Investigation of the Effect of Nylon Fiber during Concrete Rehabilitation **pp. 247-248**	M A Hossain, M M Rahman, A Z Morshed and S M Haque
18:00~18:15	ID 166	Seismic Analysis of Mud House **pp. 249-250**	M M Hossain, M Rahman and M A Hossain
18:15~18:30	ID 216	Evaluate and Analysis of Shear Strength and Bending Moment of Ferrocement Structural Elements based on Extensive Software **pp. 251-252**	M H Rashid, I M Rafizul, M M Mahmud and N H Chowdhury
18:30-18:45	ID 192	Effect of Fly Ash and Water Cement Ratio on Compressive Strength and Bond Strength of Concrete at Elevated Temperature **pp. 253-254**	A B M A Haider, N H M K Serker and T Ahmed
18:45-19:00	ID 188	2011 Sikkim Earthquake: A Case Study and the Necessity of Monitoring of Aged Structures **pp. 255-256**	N H M K Serker and T Ahmed
End of the Day			

10th Technical Session-March 24, 2012 Environmental Engineering – IV Venue: Seminar Room A Session Chair: Prof. Dr. Rezaul Karim, IUT			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 164	Study on Reusing Possibilities of Textile Wash Water Directly for Pretreatment (Scouring and Bleaching) of Cotton Goods. **pp. 045-046**	M S Osman and M A Shaid
8:45~9:00	ID 168	Methylene Blue Adsorption by Rice Husk Ash **pp. 047-048**	K M A Munim, Q H Bari and M M Rahman
9:00~9:15	ID 174	E-Waste Management in Sylhet City **pp. 049-050**	M A I Chowdhury, G M Munna and K K Kar
9:15~9:30	ID 176	Development of a Simple Ceramic Membrane Filter for Wastewater Treatment **pp. 051-052**	J K Golder, Q H Bari , M. M Hasan, K F Mallick and M S Reza
9:30~9:45	ID 189	Water Quality Monitoring in the Vicinity of Leather Industries in Bangladesh: A Case Study **pp. 053-054**	H Zahir and M A Ali
9:45~10:00	ID 252	Sustainability Options of Sanitation Projects in the Slums of Khulna City **pp. 055-056**	T K Roy
10:00~10:15	ID 193	Implementation of Reverse Logistic System as a mean of Environmental and Economical Issues **pp. 057-058**	M A Habib and H N Roy
10:15~10:30	ID 186	Sustainable Development in Construction Sectors **pp. 059-060**	Z Tafheem
10:30~11:00	**Coffee Break**		

11th Technical Session-March 24, 2012 Water Resources Engineering – III Venue: Seminar Room B Session Chair: Prof. Dr. Kh. Md. Shafiul Islam, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 303	Hydro- Morphological Study for Rehabilitation of Old Madhumati River using Mathematical Model **pp. 155-156**	M M Hossain and R M Chowdhury
8:45~9:00	ID 232	The Possible Effect of Sea Level Rise on the Livlihood of Beel Dakatia: Local Peoples' Perception **pp. 157-158**	K Syfullah, M R Islam and M S Ali
9:00~9:15	ID 257	Changing Climate and Surface & Ground Water Related Issues in Dhaka **pp. 159-160**	B M Duti , R Q Alam and M M Hossain
9:15~9:30	ID 261	Modeling River-Aquifer Interactions in a Shallow Unconfined Aquifer of Bangladesh: A Case Study **pp. 161-162**	S K Adhikary, A D Gupta, M S Babel, R S Clemente and S R Perret
9:30~9:45	ID 195	Onslaughts on the River Buriganga: Fallout of Poor Governance **pp. 163-164**	S A Haq
9:45~10:00	ID 278	Shifting of Rainfall Intensity Duration Frequency(Idf) Curves for Dhaka City **pp. 165-166**	L Pervin and M S Islam
10:00~10:15	ID 207	Estimating Shallow Groundwater Recharge from Precipitation and Water Level Observations **pp. 167-168**	S K Adhikary, T Chaki, A D Gupta and M M Rahman
10:15~10:30	ID 139	Impact of Climate Change on Floods of Bangladesh and Introducing Flood Intensity Index to Characterize the Flooding Scenario **pp. 169-170**	M S Ali, T Mahjabin and T Hosoda
10:30~11:00	**Coffee Break**		

12th Technical Session-March 24, 2012 Geotechnical Engineering – II Venue: Seminar Room C Session Chair : Prof. Dr. Md. Keramat Ali Molla, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 210	Characterization and Statistical Analysis of Geotechnical Parameters of Stabilized Organic Soil at Khulna Region **pp. 301-302**	I M Rafizul, M Assaduzzaman and M Alamgir
8:45~9:00	ID 247	Effect of Acid Rain on Geotechnical Properties of Composite Fine Grain Soil **pp. 303-304**	G Sarkar, M R Islam, M Alamgir and M Rokonuzzaman
9:00~9:15	ID 318	Determination of Shear Strength of Cohesive Soil by Rod Penetration Method **pp. 305-306**	M Abdullah, M M Tamim, S K Borman and M J Alam
9:15~9:30	ID 289	3D Finite Element Analysis to evaluate the shape effect of Rectangular Anchor Foundation in Dense Sand **pp. 307-308**	M Rokonuzzaman and T. Sakai
9:30~9:45	ID 291	Prediction of Load Carrying Capacity of Rammed Aggregate Pier using FEM **pp. 309-310**	M A Razzaque, M Alamgir, M Rokonuzzaman and M J Hossain
9:45~10:00	ID 297	The Effect of Pre-consolidation Pressure and Organic Content on the Shear Strength and Compressibility Parameters of Reconstituted Soil **pp. 311-312**	T Rabbee, I M Rafizul and M Alamgir
10:00~10:15	ID 248	Stabilization of Soft Clay Soil using Rice Husk Ash **pp. 313-314**	G Sarkar, M R Islam, M Alamgir and M Rokonuzzaman
10:15~10:30	ID 325	Finite Element Simulation of the Normal Fault Ruptures, Sand Deposit and Raft Foundation Interaction **pp. 315-316**	M Rokonuzzaman, A E Nahas and T Sakai
10:30~11:00	**Coffee Break**		

13th Technical Session-March 24, 2012		Environmental Engineering – V	Venue: Seminar Room A
Session Chair: Prof. Dr. Md. Aktarul Islam Chowdhury, SUST			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 229	Errors Involved in the Estimation of Leachate Pollution Index of Solid Waste Landfill in Bangladesh **pp. 061-062**	I M Rafizul, M M Minhaz, N H Chowdhury and M Alamgir
11:30~11:45	ID 185	Mathematical Model for Dhaka City Solid Waste Mgt. **pp. 063-064**	A B M Z Kabir, S R Shuvo
11:15~11:30	ID 236	Analysis of Leachate Characteristics in Pilot Scale Landfill Lysimeter through Statistical Approach: Bangladesh Perspectives **pp. 065-066**	I M Rafizul, M K Ahsan, S Azim and M Alamgir
11:45~12:00	ID 102	Waste Paper And Plastic Recycling – An Income Generating Activity Towards Sustainable Solid Waste Management. A Case Study- Khulna City, Bangladesh **pp. 067-068**	S M Moniruzzaman, Q H Bari and T Fukuhara
12:00~12:15	ID 208	Present Scenario of Secondary Disposal Sites for Municipal Solid Waste Management in Khulna City and Optimizing Routes for Final Disposal using GIS **pp. 069-070**	I M Rafizul, M R Kizer, S Azim and M Alamgir
12:15~12:30	ID 212	Analysis of Leachate Pollution Index and Formulation of Sub-leachate Pollution Indices of Solid Waste Landfill in Bangladesh **pp. 071-072**	I M Rafizul, N H Chowdhury, M M Minhaz and M Alamgir
12:30~12:45	ID 120	Reuses of Solid Waste in Rajshahi City, Bangladesh **pp. 073-074**	S M H Uddin, M M Islam and M B H Sikder
12:45~13:00	ID 224	Characterization and Removal of Pollutant in Landfill Lysimeter Leachate through the Small Scale Biological Treatment Techniques **pp. 075-076**	I M Rafizul, S K, D N Ahmed and M Alamgir
13:00~14:15	**Lunch**		

14th Technical Session-March 24, 2012		Water Resources Engineering – IV	Venue: Seminar Room B
Session Chair: Prof. Dr. Shawkat Osman, DUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 177	River Site Agricultural Land Protection Measures Taken By The Bamboo Bandalling Structures **pp. 171-172**	M L Rahman, B C Basak and M S Osman
11:15~11:30	ID 263	Existing Facility of Drinking Water for A Typical Arsenic Contaminated Coastal Area in the South Western Region of Bangladesh **pp. 173-174**	K M S Islam, A R M Al-Biruni and M A Rab
11:30~11:45	ID 152	Disaster Management in Bangladesh **pp. 175-176**	F Akhter
11:45~12:00	ID 320	Protection of Harmful Effect of Tsunami by Retaining System at Coastal Area **pp. 177-178**	M K Islam
12:00~12:15	ID 249	Performance of Different Types of Solar Still in Solar Desalination: A Case Study **pp. 179-180**	K M S Islam, M A Rab, A K Saha, H Khatun, A Bokshi, B C Biswas and R I Molla
12:15~12:30	ID 161	Implementation of Rain Water Harvesting A Case Study for Gazipur District in Bangladesh **pp. 181-182**	M M Islam, K Mahmud and S M Yahya
12:30~12:45	ID 328	Analyzing of Drainage Characteristics of a selected word in Khulna City Corporation **pp. 183-184**	K M S Islam, M N Hossain, M R Hossain and M A H Khan
12:45~13:00	ID 223	Water Desalination Using Basin Type Solar Still **pp. 185-186**	K M S Islam, S R Saha, M A Jafor and M A Raihan
13:00~14:15	**Lunch**		

15th Technical Session-March 24, 2012		Structural Engineering – III	Venue: Seminar Room C
Session Chair: Mr. Sayed Azizul Haque, P.Eng. , PWD			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 221	Finite Element Analysis of Composite Roof System Subjected to Various Loading by ANSYS **pp. 257-258**	M Z Islam, M J Hasan and M Afroz
11:15~11:30	ID 266	Finite Element Analysis for the Assessment of Masonry Buckling Behavior **pp. 259-260**	M A Kuddus and P R Fabregat
11:30~11:45	ID 226	Development and Evaluation of an Open Source Finite Element Analysis Framework **pp. 261-262**	M G Rashed
11:45~12:00	ID 228	Investigation of Bamboo as a Potential Reinforcement in the Concrete **pp. 263-264**	M A Sabbir, S M A Hoq and S F Fancy
12:00~12:15	ID 222	Performance of PET Bottle Fiber to Enhance the Mechanical Properties of Concrete **pp. 265-266**	M N Islam, M J Hasan and M Z Islam
12:15~12:30	ID 123	Ultimate Load Capacity Of Axially Loaded Vertical Piles From Full Scale Load Test Results Interpretations-Applied To 20 Case Histories **pp. 267-268**	A Hasnat, A R M F Uddin, E Haque, P Saha and W Rahman
12:30~12:45	ID 238	Implementation of the Building Construction Rules and Regulations in Bangladesh : The Case of Dhaka City Area **pp. 269-270**	H Jannat and K H Sohag
12:45~13:00	ID 267	Capacity of Load Bearing Masonry Wall: Experimental, Numerical and Analytical Results **pp. 271-272**	M A Kuddus and P R Fabregat
13:00~14:15	**Lunch**		

16th Technical Session-March 24, 2012	Environmental Engineering – VI		Venue: Seminar Room A
Session Chair: Prof. Dr. S. K. Acharya, BCKV, India			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 242	Community Participation in Construction and Renovation of Sanitary Latrines in the Slums of Khulna City **pp. 077-078**	T K Roy
14:30~14:45	ID 235	Salt-Brick Interaction Allowing Salt Laden Water Set Beneath To Flow Spatially within Bricks **pp. 079-080**	M S A Alam and Q H Bari
14:45~15:00	ID 250	Double Filtration for Arsenic removal from Highly Contaminated Groundwater **pp. 081-082**	K F Mallick, Q H Bari, M Shafiquzzaman, M S Reza and J K Golder
15:00~15:15	ID 209	Assessment of Sediment Quality Based on Input Materials into Shrimp Pond **pp. 083-084**	M M Hossain, K M Hassan, S R Saha, Q H Bari and S K Adhikary
15:15~15:30	ID 213	Greywater as a Resource and Its Recycling: Bangladesh Perspective **pp. 085-086**	M M Islam and K M Hassan
15:30~15:45	ID 323	Divergence of Fish-aquatic Crops in Waste 'Tal' Wetlands for Productivity and Economic Stability for Sustainable Development in India **pp. 087-088**	A M Puste, B R Pramanik, K Jana and M D Gupta
15:45~16:00	ID 251	Arsenic Removal from Highly Contaminated Groundwater by Iron Oxidation Ceramic Filter Units With Different Iron Sources **pp. 089-090**	K F Mallick, Q H Bari, M Shafiquzzaman, J K Golder and M S Reza
16:00-16:15	ID 265	Establishment of BOD and COD Relationship of Different Wastewater **pp. 091-092**	M R Karim and N Jahan
16:15-16:45	**Coffee Break**		

17th Technical Session-March 24, 2012	Transportation Engineering – II		Venue: Seminar Room B
Session Chair: Prof. Dr. Quazi Sazzad Hossain, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 178	Effects of Car Involvement in Congestion and Road Accident in Dhaka City: a Study on Sustainable Solutions **pp. 199-200**	A Siddique, S Iffat and M M Islam
14:30~14:45	ID 194	Capacity Evaluation of Roundabout Junction in Khulna Metropolitan City, Bangladesh **pp. 201-202**	M T Tariq and Q S Hossain
14:45~15:00	ID 276	Study on Pedestrian Behavior in Khulna Metropolitan City, Bangladesh **pp. 203-204**	M I Nazir, S K Adhikary, Q S Hossain and S A Ali
15:00~15:15	ID 279	Current Car Parking Situation in KC : A Case Study **pp. 205-206**	N Das, S A Ali, M A Rahman, Q S Hossain and M I Nazir
15:15~15:30	ID 305	Transportation System Management: Assessment for Implementation of General Tools in Dhaka City **pp. 207-208**	S M S Mahmud and M S Hoque
15:30~15:45	ID 306	Define the Problems Associate with an Existing Intersection & Designing it to an Ideal Intersection **pp. 209-210**	S Rahman
15:45~16:00	ID 307	Performance Evaluation of Speed Reducing Devices on National Highways of Bangladesh **pp. 211-212**	H M Ahsan, S M S Mahmud and M A Hossain
16:00-16:15	ID 317	Traffic Study and Travel Forecast Case Study: Traffic Survey at Bekutia Bridge over Khocha river ar Perojpur-Jhalokathi Road **pp. 213-214**	S Munira, K Maina and H B Hasan
16:15-16:45	**Coffee Break**		

18th Technical Session-March 24, 2012	Structural Engineering – IV		Venue: Seminar Room C
Session Chair: Prof. Dr. Quazi Azizul Mowla, BUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 274	Corrosion Effect on Strength and Covering of Reinforced Concrete Structure **pp. 273-274**	M K Howlader, M A Nayeem and M H Rashid
14:30~14:45	ID 277	Effects of Aggregate Type on Specific Heat of Concrete **pp. 275-276**	M K Howlader, M H Rashid and T Haque
14:45~15:00	ID 283	Strengthening and Performance Evaluation of Masonry Shear wall using Different FRPs **pp. 277-278**	A Rahman and T Ueda
15:00~15:15	ID 286	Properties of Fiber Reinforced Concrete using MS Nail **pp. 279-280**	C Mondol and M A Rahman
15:15~15:30	ID 287	Optimum Design Criteria of Tall building for Different Subsoil Condition **pp. 281-282**	S M Alimuz-Zaman and A Rahman
15:30~15:45	ID 288	Experimental Investigation of Shear strength for Brick-Mortar Interface for Different Mortar Strength **pp. 283-284**	K Ahmed and M A Rahman
15:45~16:00	ID 292	Effect of Sulfate Rich Environment on the Performance of Cement Mortar **pp. 285-286**	M K Hasan, M H Rashid
16:00-16:15	ID 293	Performance of Cement Mortar in Acidic Environment **pp. 287-288**	M K Hassan, M H Rashid and S Parvin
16:15-16:45	**Coffee Break**		

19th Technical Session-March 24, 2012		Environmental Engineering – VII	Venue: Seminar Room A
		Session Chair : Dr. Kh. Mahbub Hasan, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
16:45~17:00	ID 106	Evaluating the Adaptation Strategy of Livelihood Pattern in the Water Logged Area of South-West Coastal Bangladesh **pp. 109-110**	S R Mollick and A K Azad
17:00~17:15	ID 142	Assessment of Heavy Metal Contamination & Sediment Quality in the Turag River, Bangladesh **pp. 111-112**	M D Hossain, N Ferdousi and T Mahmud
17:15~17:30	ID 159	Assessment of Heavy Metal Contamination & Sediment Quality in the Sitalakhya River on the basis of Geo-accumulation index, Bangladesh **pp. 113-114**	M D Hossain, S Islam and K Nahar
17:30~17:45	ID 290	Heavy Metal Assessment in Sediment of Shitalakkhya River **pp. 115-116**	S Ahmed and F T Jahara
17:45~18:00	ID 304	Assessment of Water Quality Index Of Water Bodies Along Joydevpur-Chandra-Tangail-Hatikamrul Road **pp. 117-118**	R M Chowdhury, M N Naser and S Y Muntasir
18:00~18:15	ID 311	Removal of COD of Reactive Dyes using Alum **pp. 119-120**	M R Khan, Z B Mukhlish, T Kanti Deb, S Y Sony and M RKarim
18:15~18:30	ID 324	Empowering Farm Women in India through Enterprise Synergy **pp. 121-122**	K Brahmachari and S K Acharya
18:30~18:45	Other Technical Session running		
19:00-22:00	**Closing Ceremony and Conference Dinner**		

20th Technical Session-March 24, 2012		Transportation Engineering – III	Venue: Seminar Room B
		Session Chair : Dr. Md. Rokonuzzaman, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
16:45~17:00	ID 172	Traffic Information Deriving using GPS Probe Vehicle Data Integrated with GIS **pp. 215-216**	M M Rahman, S G Sagar, N Saha, S Sharmin and M Rahman
17:00~17:15	ID 308	Vehicle Aggressivity in Bangladesh: Case Study on Large Truck **pp. 217-218**	H M Ahsan, S M S Mahmud and M M R Bhuiyan
17:15~17:30	ID 309	Impact of Direct Local Access Road with the Classical National Highways in Bangladesh: A Case Study of Dhaka-Mawa Highway **pp. 219-220**	H M Ahsan, S M S Mahmud and M A Goni
17:30~17:45	ID 201	A GIS based Approach to Combine the Waterway and Road Network for the Solid Waste Management of Dhaka City **pp. 221-222**	M S Islam, Abdullah-Al Muyeed, L Pervin and M T Hasan
17:45~18:00	ID 329	Production Of Citric Acid From Oil Palm Empty Fruit Bunches Through Solid State Bioconversion In Tray Fermentor **pp. 223-224**	M N Bari and M Z Alam
18:00~18:45	Other Technical Session running		
19:00-22:00	**Closing Ceremony and Conference Dinner**		

21th Technical Session-March 24, 2012		Environmental Engineering – VIII	Venue: Seminar Room C
		Session Chair: Prof. Dr. Md. Abul Bashar, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
16:45~17:00	ID 315	Fire Vulnerability Assessment of High-Rise Apartments in Chittagong City: A Case of Residential Area **pp. 093-094**	S Islam, N Sakib, K M B Ali and M I Sarwar
17:00~17:15	ID 253	Water for Environmental Sanitation in the Slums of Khulna City **pp. 095-096**	T K Roy, U Kumar and S M T Islam
17:15~17:30	ID 255	Study on Biogas Generation from Weeds **pp. 097-098**	M M Islam, A K M M Islam, M A Rab, M Manjur-A-Elahi and Q H Bari
17:30~17:45	ID 116	Water Based Urbanization: An Analytical Study on Dhaka **pp. 099-100**	Q A Mowla
17:45~18:00	ID 225	Characterization and Treatment of Landfill Lysimeter Leachate: Coagulation Process using Poly-Aluminum Chloride and Ferric Chloride **pp. 101-102**	I M Rafizul, I Rahaman and M Alamgir
18:00~18:15	ID 281	Assessment of Heavy Metal Contamination & Sediment Quality in the Turag River, Bangladesh **pp. 103-104**	M D Hossain, S B Salam and T Islam
18:15~18:30	ID 108	Experimentation of Self Turning Reactor (STR) Composting System in Thailand **pp. 105-106**	A Al-Muyeed and Tetsuya Ishida
18:30~18:45	ID 282	Coal Quality Assessment of Barapukuria Coalmine, Bangladesh **pp. 107-108**	M F Howladar, M Hashan and M R Uddin
19:00-22:00	**Closing Ceremony and Conference Dinner**		

Conference Program Outline for ICCESD-2012

Collection of Conference Kit:

March 22, 2012 (from 3:00~6:00 PM)
Venue: Civil Engineering Building, KUET

Day 1: March 23, 2012 (Friday)

Time	Venue	Events
8:00-8:30	KUET Auditorium	Reporting of Registered Participants
8:30-9:15	KUET Auditorium	Inaugural Ceremony of ICCESD-2012
9:15-10:45	KUET Auditorium	Keynote Session: Keynote Speech A: Prof. Dr. Ainun Nishat, VC, BRAC University Keynote Speech B: Prof. Dr. A. M. M. Safiullah, VC, AUST Keynote Speech C: Prof. Dr. Monjur Hossain, KUET
10:45-11:15	KUET Auditorium	Refreshment
11:15-12:45	CE Building	Parallel Technical Session-I (1st, 2nd & 3rd)
12:45-14:30	CE Building	Friday prayer & Lunch
14:30-16:30	CE Building	Parallel Technical Session-II (4th, 5th & 6th)
16:30-17:00	CE Building	Coffee Break
17:00-19:00	CE Building	Parallel Technical Session-III (7th, 8th & 9th)

Day 2: March 24, 2012 (Saturday)

Time	Venue	Events
8:30-10:30	CE Building	Parallel Technical Session-IV (10th, 11th & 12th)
10:30-11:00	CE Building	Coffee Break
11:00-13:00	CE Building	Parallel Technical Session-V (13th, 14th & 15th)
13:00-14:15	CE Building	Lunch
14:15-16:15	CE Building	Parallel Technical Session-VI (16th, 17th & 18th)
16:15-16:45	CE Building	Coffee Break
16:45-18:45	CE Building	Parallel Technical Session-VII (19th, 20th & 21st)
19:00-22:00	Hotel Castle Salam	Closing Ceremony and Conference Dinner

TECHNICAL SESSIONS

1st Technical Session-March 23, 2012 Environmental Engineering – I Venue: Seminar Room A Session Chair: Prof. Dr. Quazi Hamidul Bari, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:15~11:30	ID 112	Forest – A Natural Defense to Counteract the Climate Change in Bangladesh **pp. 001-002**	S M H Uddin, M M Rahman, M B H Sikder and M M Rahman
11:30~11:45	ID 182	Consideration of Integrated Influence Scenario of Climate Change and Air Pollution at City level to frame out Urban Policy and Strategy **pp. 003-004**	M S Uddin
11:45~12:00	ID 244	The Impact of Climate Change on Agriculture in Bangladesh **pp. 005-006**	S Islam, U Klönne, L Allesson, M E Hoque and M M Anannya
12:00~12:15	ID 300	Development of Crop Yield Prediction Model For Bangladesh: A Way to Food Security in Context of Climate Change **pp. 007-008**	S Shaika, R M Mafizur and P Hasan
12:15~12:30	ID 312	Impacts of Climate Change Scenarios on Crop Simulation (Boro Cultivation) in Bangladesh using DSSAT **pp. 009-010**	T Afrin, R Rumman and F Warda
12:30~12:45	ID 316	Adapting to Climate Change in Urban Areas of Bengal Delta: A Case Study on Khulna City **pp. 011-012**	A Pervin, D M Khan, S M M Rahman, A Zaman and M M Hossain
12:45-14:30	**Friday prayer & Lunch**		

2nd Technical Session-March 23, 2012 Transportation Engineering – I Venue: Seminar Room B Session Chair: Prof. Dr. Hasib Mohammed Ahsan, BUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:15~11:30	ID 104	Experimental Behavior of Bituminous Mixes with Waste Concrete Aggregates **pp. 187-188**	M A Sobhan and T U Ahmed
11:30~11:45	ID 107	Effects of Waste Polyethylene on Bituminous Mixes **pp. 189-190**	M N Rahman, M A Sobhan and T U Ahmed
11:45~12:00	ID 105	Effect of Waste Concrete Aggregates on the Compressive Strength of Bituminous Mixes **pp. 191-192**	M A Sobhan and T U Ahmed
12:00~12:15	ID 206	Effect of Brick Chips as Coarse Aggregate in Bituminous Mixes **pp. 193-194**	H M Rasel, M A Sobhan, M N Rahman and S M A Aman
12:15~12:30	ID 231	Effect of Fillers on Bituminous Paving Mixes: An Experimental study **pp. 195-196**	A Rahman, S K Adhikary, Q S Hossain and S A Ali
12:30~12:45	ID 314	Development of Paratransit as a Sustainable and Effective Mode of Transport - A Study Based on Dhaka City **pp. 197-198**	H B Hasan, K M and S Munira
12:45-14:30	**Friday prayer & Lunch**		

3rd Technical Session-March 23, 2012 Geotechnical Engineering – I Venue: Seminar Room C Session Chair: Prof. Dr. A M M Safiullah, VC, AUST			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:15~11:30	ID 115	Liquefaction Characteristics of Infilled Soil at Banasree in Dhaka City **pp. 289-290**	P Saha, A R M F Uddin and A Hasnat
11:30~11:45	ID 146	Study on the Locally Available Clayey Soils for the Preparation of Liner **pp. 291-292**	S R. Saha and M Alamgir
11:45~12:00	ID 147	Model Study to Improve Lateral Load Resistance of Piles in Sand using Geotextile **pp. 293-294**	M A Sayeed and Shubhra Pk
12:00~12:15	ID 158	Model Study on the Behavior of Pile in Sand Subjected to Lateral Load **pp. 295-296**	Shubhra Pk., M A Sayeed and M A Rahaman
12:15~12:30	ID 165	Effect of Anchor Size and Cable Diameter on Anchored Earth Retaining System **pp. 297-298**	M S Alam, M Al - Amin and N Sakib
12:30~12:45	ID 181	Evaluation of Peak Liquefaction Potential using Cone Penetration Test **pp. 299-300**	R K Mazumder, E M Murad and M A Ansary
12:45-14:30	**Friday prayer & Lunch**		

4th Technical Session-March 23, 2012		Environmental Engineering – II	Venue: Seminar Room A
Session Chair: Prof. Dr. Md. Saiful Islam, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 131	Quality of Treated Water in WTP at KUET Campus **pp. 013-014**	M Akter, K M Hassan, and Q H Bari
14:45~15:00	ID 133	A Case Study on Arsenic Problem at Hariarampur Village, Bagha, Rajshahi **pp. 015-016**	M Ahmeduzzaman and B Pramanik
15:00~15:15	ID 310	Complete Decoloulization of Reactive Dye by Combination of Coagulation Adsorbtion-Membrane Filtration: A Hybrid Study **pp. 017-018**	M ZB Mukhlish,T K Deb, S Majumder, S Y Sony, M R Karim
15:15~15:30	ID 137	Assessment of Water Quality from Different Sources in Khulna District **pp. 019-020**	M M Rahman, M Rahman, T Tamanna and S K Biswas
15:30~15:45	ID 157	Sustainability of Ecosan Toilets in Waterlogged Areas: A Rural Perspective in Bangladesh **pp. 021-022**	S R Saha, K M Hassan, M M Hussain and Q H Bari
15:45~16:00	ID 135	Perfluorinated Chemicals-a New Class of Global Pollutants of the World **pp. 023-024**	B Pramanik, M Ahmeduzzaman, S Fatihah and S Zarin
16:00-16:15	ID 162	Assessment of Pharmaceutical Industries in Bangladesh - An Effective Step towards the Achievement of Environmental Sustainability **pp. 025-026**	R R Wahi, K Mahmud and S Rahman
16:15-16:30	ID 322	Non-Descript Lands in India : An Alternative Ecosystem for Seed Production **pp. 027-028**	M M Adhikary, S K Acharya and A B Sharangi
16:30-17:00		**Coffee Break**	

5th Technical Session-March 23, 2012		Water Resources Engineering – I	Venue: Seminar Room B
Session Chair: Prof. Dr. M. Monowar Hossain, Executive Director, IWM			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 124	Physical Model Studies to Support the Design of Main Spillway of the Proposed Ganges Barrage **pp. 123-124**	P Kanungoe, A K M Ashrafuzzaman, M N I Siddique and Q A Sayeed
14:45~15:00	ID 138	Temporal and Spatial Distribution of Suspended Sediment Concentration In Bhairab River **pp. 125-126**	M S Ali, T Mahjabin, M B Hossen and M Abdullah-Al-Imran
15:00~15:15	ID 321	Effects of Stream barb on Straight Channel Bed Configuration: Numerical Simulation **pp. 127-128**	S A A M Hossain, M J Uddin and M S Ali
15:15~15:30	ID 153	A Scenario-based Holistic Approach for Multipurpose Reservoir Operation Considering EFR, Irrigation and Hydropower Demand: A Case Study **pp. 129-130**	S K Adhikary, S S Atef, A D Gupta and M S Babel
15:30~15:45	ID 144	Surge Phenomena and Factors Influencing Storm Surges **pp. 131-132**	N C Sarkar, A Islam and S K D Sarma
15:45~16:00	ID 145	Tsunami Hazard Assessment in the Northern Bay of Bengal along the Bangladesh Coast **pp. 133-134**	MA Hussain, MSA Khan, A Haque, ZH Khan and MMZ Ahmed
16:00-16:15	ID 218	Modelling Proposed Water Distribution Network for Water Supply in Lohagora Pourashava of Narail District, Bangladesh **pp. 135-136**	S K Adhikary and T Chaki
16:15-16:30	ID 140	Numerical Simulation of Dead Zone flows in an Open Channel with a Side Cavity and Sudden Enlargement **pp. 137-138**	M S Ali, T Mahjabin and S J Ria
16:30-17:00		**Coffee Break**	

6th Technical Session-March 23, 2012		Structural Engineering – I	Venue: Seminar Room C
Session Chair: Prof. Dr. Md. Monjur Hossain, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:30~14:45	ID 119	Performance of Adobe Blocks Randomly Reinforced with Natural Fibers **pp. 225-226**	M Z I Bhuiyan, M S Islam, M F A Kayser and M M Rahman
14:45~15:00	ID 128	Study the Role of Building Setback and Height, in terms of Indoor Temperature due to the Direct Component of Solar Radiation on west façade of Residential Buildings: Comparing 1996 and 2008 'Imarat Nirman Bidhimala' **pp. 227-228**	I Rahman
15:00~15:15	ID 129	Assessment of Seismic Vulnerability using Different Site Conditions **pp. 229-230**	M A Rahman and M S Ullah
15:15~15:30	ID 130	Sustainable Development in Drift Control of Tall Buildings Due to Wind Load: Critical Analyses of the Structural Systems **pp. 231-232**	M M Islam, A Siddique and A Murshed
15:30~15:45	ID 150	Comparison of Creep Behavior of Concrete Made by Brick Chips & Stone Chips in Bangladesh **pp. 233-234**	M S Arefin and M Alin
15:45~16:00	ID 187	Damage Identification in RC Column using Long-gage FBG Sensors **pp. 235-236**	N H M K Serker and Z Wu
16:00-16:15	ID 155	Stress Distribution of CFRP Strengthened Steel Hollow Sections under Tension **pp. 237-238**	K Shahanara and S Fawzia
16:15-16:30	ID 319	Optimization of Column **pp. 239-240**	A A Mamun and M S Bari
16:30-17:00		**Coffee Break**	

7th Technical Session-March 23, 2012		Environmental Engineering – III	Venue: Seminar Room A
Session Chair : Prof. M. M. Adhikary, Former VC, BCKV, India			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 103	Physico-chemical Characteristics of Surface Water in the Magura Municipality Area, Bangladesh **pp. 029-30**	F Akter and A K Azad
17:15~17:30	ID 302	Membrane Bioreactor Process using a Simple Ceramic Filter for Wastewater Reclamation and Reuse **pp. 031-032**	M M Hasan, M Shafiquzzaman and J Nakajima
17:30~17:45	ID 109	The Hunger, Poverty and Silence: The Invisible Bond in Social Ecology **pp. 033-034**	S Bera and S K Acharya
17:45~18:00	ID 110	KVK: An Impact Study on Transforming Social Ecology in Odissa, India **pp. 035-036**	B K Mohanty, S Bera and S K Acharya
18:00~18:15	ID 111	Estimation of Social Entropy: The Dictum of Neo Modernism in Agricultural Knowledge Environment in The Context of Global Warming **pp. 037-038**	S K Acharya, B Bhattacharya and S Bera
18:15~18:30	ID 117	Problems & Prospects of Solar Energy Usage in Bangladesh: An exploratory Study on Sylhet **pp. 039-040**	K R Tanvir
18:30-18:45	ID 122	Icthiofaunal (Fish) Biodiversity: Estimation of Erosion through Participatory Learning Action **pp. 041-042**	S Bera, S K Acharya and A Biswas
18:45-19:00	ID 313	Development of GIS Based Decision Support Database for Improved Healthcare Waste Management in Khulna City **pp. 043-044**	I M Rafizul, M S Islam, S Azim and M Alamgir
End of the Day			

8th Technical Session-March 23, 2012		Water Resources Engineering – II	Venue: Seminar Room B
Prof. Dr. M. Monowar Hossain, Executive Director, IWM			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 114	Study on Drainage Congestion in Dhaka-Narayanganj-Demra (DND) Project Area **pp. 139-140**	M A Rahman and M M Mahmood
17:15~17:30	ID 191	Sustainability of Peripheral Rivers of Dhaka as Sources of Water Supply **pp. 141-142**	M Khan and M J Haider
17:30~17:45	ID 271	Cost Effective Bank Protection for Halda River **pp. 143-144**	M N N Khan, M S Haider and A Akter
17:45~18:00	ID 245	The Impact of Sea-level rise on Mangrove Ecosystems in Bangladesh – A Review **pp. 145-146**	K Ekstedt, A Malekahmad, C Rimberth, S Islam, M E Hoque and T Tamanna
18:00~18:15	ID 260	Application of a Simple Stochastic Model for Predicting Groundwater Table Fluctuations in a Shallow Aquifer of Bangladesh **pp. 147-148**	S K Adhikary and M MRahman
18:15~18:30	ID 205	Spatial and Temporal Variation of Surface Water Quality of Sundarban Reserved Forest **pp. 149-150**	S Moazzem and R Mamtaz
18:30-18:45	ID 217	Delineating Aquifer System Delineation and Groundwater Resources Assessment in Lohagara Municipality of Narail District, Bangladesh **pp. 151-152**	S K Adhikary, T Chaki, M M Rahman and A D Gupta
18:45-19:00	ID 327	Development of an Innovative Water Treatment Technique Using Solar Still cum Sand Filter **pp. 153-154**	M A S Khan and K M Hassan
End of the Day			

9th Technical Session-March 23, 2012		Structural Engineering – II	Venue: Seminar Room C
Session Chair : Brig. General Dr. Habibur Rahman Kamal psc, Chairman, KDA			
Time	**Paper ID**	**Paper Title**	**Authors Name**
17:00~17:15	ID 167	Prospect of Bangladeshi Fly Ash in Cement Production **pp. 241-242**	M M Islam and M S Islam
17:15~17:30	ID 169	Performance Based Fire Safety Management in Commercial Mixed Use Buildings of Bangladesh **pp. 243-244**	A Sarraz and M A I Chowdhury
17:30~17:45	ID 170	Analytical Study on Slender Partially Encased Composite Columns Under Eccentric Loading **pp. 245-246**	S Ali and M Begum
17:45~18:00	ID 326	Investigation of the Effect of Nylon Fiber during Concrete Rehabilitation **pp. 247-248**	M A Hossain, M M Rahman, A Z Morshed and S M Haque
18:00~18:15	ID 166	Seismic Analysis of Mud House **pp. 249-250**	M M Hossain, M Rahman and M A Hossain
18:15~18:30	ID 216	Evaluate and Analysis of Shear Strength and Bending Moment of Ferrocement Structural Elements based on Extensive Software **pp. 251-252**	M H Rashid, I M Rafizul, M M Mahmud and N H Chowdhury
18:30-18:45	ID 192	Effect of Fly Ash and Water Cement Ratio on Compressive Strength and Bond Strength of Concrete at Elevated Temperature **pp. 253-254**	A B M A Haider, N H M K Serker and T Ahmed
18:45-19:00	ID 188	2011 Sikkim Earthquake: A Case Study and the Necessity of Monitoring of Aged Structures **pp. 255-256**	N H M K Serker and T Ahmed
End of the Day			

10th Technical Session-March 24, 2012 Environmental Engineering – IV Venue: Seminar Room A Session Chair: Prof. Dr. Rezaul Karim, IUT			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 164	Study on Reusing Possibilities of Textile Wash Water Directly for Pretreatment (Scouring and Bleaching) of Cotton Goods. **pp. 045-046**	M S Osman and M A Shaid
8:45~9:00	ID 168	Methylene Blue Adsorption by Rice Husk Ash **pp. 047-048**	K M A Munim, Q H Bari and M M Rahman
9:00~9:15	ID 174	E-Waste Management in Sylhet City **pp. 049-050**	M A I Chowdhury, G M Munna and K K Kar
9:15~9:30	ID 176	Development of a Simple Ceramic Membrane Filter for Wastewater Treatment **pp. 051-052**	J K Golder, Q H Bari , M. M Hasan, K F Mallick and M S Reza
9:30~9:45	ID 189	Water Quality Monitoring in the Vicinity of Leather Industries in Bangladesh: A Case Study **pp. 053-054**	H Zahir and M A Ali
9:45~10:00	ID 252	Sustainability Options of Sanitation Projects in the Slums of Khulna City **pp. 055-056**	T K Roy
10:00~10:15	ID 193	Implementation of Reverse Logistic System as a mean of Environmental and Economical Issues **pp. 057-058**	M A Habib and H N Roy
10:15~10:30	ID 186	Sustainable Development in Construction Sectors **pp. 059-060**	Z Tafheem
10:30~11:00	**Coffee Break**		

11th Technical Session-March 24, 2012 Water Resources Engineering – III Venue: Seminar Room B Session Chair: Prof. Dr. Kh. Md. Shafiul Islam, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 303	Hydro- Morphological Study for Rehabilitation of Old Madhumati River using Mathematical Model **pp. 155-156**	M M Hossain and R M Chowdhury
8:45~9:00	ID 232	The Possible Effect of Sea Level Rise on the Livlihood of Beel Dakatia: Local Peoples' Perception **pp. 157-158**	K Syfullah, M R Islam and M S Ali
9:00~9:15	ID 257	Changing Climate and Surface & Ground Water Related Issues in Dhaka **pp. 159-160**	B M Duti , R Q Alam and M M Hossain
9:15~9:30	ID 261	Modeling River-Aquifer Interactions in a Shallow Unconfined Aquifer of Bangladesh: A Case Study **pp. 161-162**	S K Adhikary, A D Gupta, M S Babel, R S Clemente and S R Perret
9:30~9:45	ID 195	Onslaughts on the River Buriganga: Fallout of Poor Governance **pp. 163-164**	S A Haq
9:45~10:00	ID 278	Shifting of Rainfall Intensity Duration Frequency(Idf) Curves for Dhaka City **pp. 165-166**	L Pervin and M S Islam
10:00~10:15	ID 207	Estimating Shallow Groundwater Recharge from Precipitation and Water Level Observations **pp. 167-168**	S K Adhikary, T Chaki, A D Gupta and M M Rahman
10:15~10:30	ID 139	Impact of Climate Change on Floods of Bangladesh and Introducing Flood Intensity Index to Characterize the Flooding Scenario **pp. 169-170**	M S Ali, T Mahjabin and T Hosoda
10:30~11:00	**Coffee Break**		

12th Technical Session-March 24, 2012 Geotechnical Engineering – II Venue: Seminar Room C Session Chair : Prof. Dr. Md. Keramat Ali Molla, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
8:30~8:45	ID 210	Characterization and Statistical Analysis of Geotechnical Parameters of Stabilized Organic Soil at Khulna Region **pp. 301-302**	I M Rafizul, M Assaduzzaman and M Alamgir
8:45~9:00	ID 247	Effect of Acid Rain on Geotechnical Properties of Composite Fine Grain Soil **pp. 303-304**	G Sarkar, M R Islam, M Alamgir and M Rokonuzzaman
9:00~9:15	ID 318	Determination of Shear Strength of Cohesive Soil by Rod Penetration Method **pp. 305-306**	M Abdullah, M M Tamim, S K Borman and M J Alam
9:15~9:30	ID 289	3D Finite Element Analysis to evaluate the shape effect of Rectangular Anchor Foundation in Dense Sand **pp. 307-308**	M Rokonuzzaman and T. Sakai
9:30~9:45	ID 291	Prediction of Load Carrying Capacity of Rammed Aggregate Pier using FEM **pp. 309-310**	M A Razzaque, M Alamgir, M Rokonuzzaman and M J Hossain
9:45~10:00	ID 297	The Effect of Pre-consolidation Pressure and Organic Content on the Shear Strength and Compressibility Parameters of Reconstituted Soil **pp. 311-312**	T Rabbee, I M Rafizul and M Alamgir
10:00~10:15	ID 248	Stabilization of Soft Clay Soil using Rice Husk Ash **pp. 313-314**	G Sarkar, M R Islam, M Alamgir and M Rokonuzzaman
10:15~10:30	ID 325	Finite Element Simulation of the Normal Fault Ruptures, Sand Deposit and Raft Foundation Interaction **pp. 315-316**	M Rokonuzzaman, A E Nahas and T Sakai
10:30~11:00	**Coffee Break**		

13th Technical Session-March 24, 2012 Environmental Engineering – V Venue: Seminar Room A Session Chair: Prof. Dr. Md. Aktarul Islam Chowdhury, SUST			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 229	Errors Involved in the Estimation of Leachate Pollution Index of Solid Waste Landfill in Bangladesh **pp. 061-062**	I M Rafizul, M M Minhaz, N H Chowdhury and M Alamgir
11:30~11:45	ID 185	Mathematical Model for Dhaka City Solid Waste Mgt. **pp. 063-064**	A B M Z Kabir, S R Shuvo
11:15~11:30	ID 236	Analysis of Leachate Characteristics in Pilot Scale Landfill Lysimeter through Statistical Approach: Bangladesh Perspectives **pp. 065-066**	I M Rafizul, M K Ahsan, S Azim and M Alamgir
11:45~12:00	ID 102	Waste Paper And Plastic Recycling – An Income Generating Activity Towards Sustainable Solid Waste Management. A Case Study- Khulna City, Bangladesh **pp. 067-068**	S M Moniruzzaman, Q H Bari and T Fukuhara
12:00~12:15	ID 208	Present Scenario of Secondary Disposal Sites for Municipal Solid Waste Management in Khulna City and Optimizing Routes for Final Disposal using GIS **pp. 069-070**	I M Rafizul, M R Kizer, S Azim and M Alamgir
12:15~12:30	ID 212	Analysis of Leachate Pollution Index and Formulation of Sub-leachate Pollution Indices of Solid Waste Landfill in Bangladesh **pp. 071-072**	I M Rafizul, N H Chowdhury, M M Minhaz and M Alamgir
12:30~12:45	ID 120	Reuses of Solid Waste in Rajshahi City, Bangladesh **pp. 073-074**	S M H Uddin, M M Islam and M B H Sikder
12:45~13:00	ID 224	Characterization and Removal of Pollutant in Landfill Lysimeter Leachate through the Small Scale Biological Treatment Techniques **pp. 075-076**	I M Rafizul, S K, D N Ahmed and M Alamgir
13:00~14:15	**Lunch**		

14th Technical Session-March 24, 2012 Water Resources Engineering – IV Venue: Seminar Room B Session Chair: Prof. Dr. Shawkat Osman, DUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 177	River Site Agricultural Land Protection Measures Taken By The Bamboo Bandalling Structures **pp. 171-172**	M L Rahman, B C Basak and M S Osman
11:15~11:30	ID 263	Existing Facility of Drinking Water for A Typical Arsenic Contaminated Coastal Area in the South Western Region of Bangladesh **pp. 173-174**	K M S Islam, A R M Al-Biruni and M A Rab
11:30~11:45	ID 152	Disaster Management in Bangladesh **pp. 175-176**	F Akhter
11:45~12:00	ID 320	Protection of Harmful Effect of Tsunami by Retaining System at Coastal Area **pp. 177-178**	M K Islam
12:00~12:15	ID 249	Performance of Different Types of Solar Still in Solar Desalination: A Case Study **pp. 179-180**	K M S Islam, M A Rab, A K Saha, H Khatun, A Bokshi, B C Biswas and R I Molla
12:15~12:30	ID 161	Implementation of Rain Water Harvesting A Case Study for Gazipur District in Bangladesh **pp. 181-182**	M M Islam, K Mahmud and S M Yahya
12:30~12:45	ID 328	Analyzing of Drainage Characteristics of a selected word in Khulna City Corporation **pp. 183-184**	K M S Islam, M N Hossain, M R Hossain and M A H Khan
12:45~13:00	ID 223	Water Desalination Using Basin Type Solar Still **pp. 185-186**	K M S Islam, S R Saha, M A Jafor and M A Raihan
13:00~14:15	**Lunch**		

15th Technical Session-March 24, 2012 Structural Engineering – III Venue: Seminar Room C Session Chair: Mr. Sayed Azizul Haque, P.Eng. , PWD			
Time	**Paper ID**	**Paper Title**	**Authors Name**
11:00~11:15	ID 221	Finite Element Analysis of Composite Roof System Subjected to Various Loading by ANSYS **pp. 257-258**	M Z Islam, M J Hasan and M Afroz
11:15~11:30	ID 266	Finite Element Analysis for the Assessment of Masonry Buckling Behavior **pp. 259-260**	M A Kuddus and P R Fabregat
11:30~11:45	ID 226	Development and Evaluation of an Open Source Finite Element Analysis Framework **pp. 261-262**	M G Rashed
11:45~12:00	ID 228	Investigation of Bamboo as a Potential Reinforcement in the Concrete **pp. 263-264**	M A Sabbir, S M A Hoq and S F Fancy
12:00~12:15	ID 222	Performance of PET Bottle Fiber to Enhance the Mechanical Properties of Concrete **pp. 265-266**	M N Islam, M J Hasan and M Z Islam
12:15~12:30	ID 123	Ultimate Load Capacity Of Axially Loaded Vertical Piles From Full Scale Load Test Results Interpretations-Applied To 20 Case Histories **pp. 267-268**	A Hasnat, A R M F Uddin, E Haque, P Saha and W Rahman
12:30~12:45	ID 238	Implementation of the Building Construction Rules and Regulations in Bangladesh : The Case of Dhaka City Area **pp. 269-270**	H Jannat and K H Sohag
12:45~13:00	ID 267	Capacity of Load Bearing Masonry Wall: Experimental, Numerical and Analytical Results **pp. 271-272**	M A Kuddus and P R Fabregat
13:00~14:15	**Lunch**		

16th Technical Session-March 24, 2012	Environmental Engineering – VI	Venue: Seminar Room A	
Session Chair: Prof. Dr. S. K. Acharya, BCKV, India			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 242	Community Participation in Construction and Renovation of Sanitary Latrines in the Slums of Khulna City **pp. 077-078**	T K Roy
14:30~14:45	ID 235	Salt-Brick Interaction Allowing Salt Laden Water Set Beneath To Flow Spatially within Bricks **pp. 079-080**	M S A Alam and Q H Bari
14:45~15:00	ID 250	Double Filtration for Arsenic removal from Highly Contaminated Groundwater **pp. 081-082**	K F Mallick, Q H Bari, M Shafiquzzaman, M S Reza and J K Golder
15:00~15:15	ID 209	Assessment of Sediment Quality Based on Input Materials into Shrimp Pond **pp. 083-084**	M M Hossain, K M Hassan, S R Saha, Q H Bari and S K Adhikary
15:15~15:30	ID 213	Greywater as a Resource and Its Recycling: Bangladesh Perspective **pp. 085-086**	M M Islam and K M Hassan
15:30~15:45	ID 323	Divergence of Fish-aquatic Crops in Waste 'Tal' Wetlands for Productivity and Economic Stability for Sustainable Development in India **pp. 087-088**	A M Puste, B R Pramanik, K Jana and M D Gupta
15:45~16:00	ID 251	Arsenic Removal from Highly Contaminated Groundwater by Iron Oxidation Ceramic Filter Units With Different Iron Sources **pp. 089-090**	K F Mallick, Q H Bari, M Shafiquzzaman, J K Golder and M S Reza
16:00-16:15	ID 265	Establishment of BOD and COD Relationship of Different Wastewater **pp. 091-092**	M R Karim and N Jahan
16:15-16:45	**Coffee Break**		

17th Technical Session-March 24, 2012	Transportation Engineering – II	Venue: Seminar Room B	
Session Chair: Prof. Dr. Quazi Sazzad Hossain, KUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 178	Effects of Car Involvement in Congestion and Road Accident in Dhaka City: a Study on Sustainable Solutions **pp. 199-200**	A Siddique, S Iffat and M M Islam
14:30~14:45	ID 194	Capacity Evaluation of Roundabout Junction in Khulna Metropolitan City, Bangladesh **pp. 201-202**	M T Tariq and Q S Hossain
14:45~15:00	ID 276	Study on Pedestrian Behavior in Khulna Metropolitan City, Bangladesh **pp. 203-204**	M I Nazir, S K Adhikary, Q S Hossain and S A Ali
15:00~15:15	ID 279	Current Car Parking Situation in KC : A Case Study **pp. 205-206**	N Das, S A Ali, M A Rahman, Q S Hossain and M I Nazir
15:15~15:30	ID 305	Transportation System Management: Assessment for Implementation of General Tools in Dhaka City **pp. 207-208**	S M S Mahmud and M S Hoque
15:30~15:45	ID 306	Define the Problems Associate with an Existing Intersection & Designing it to an Ideal Intersection **pp. 209-210**	S Rahman
15:45~16:00	ID 307	Performance Evaluation of Speed Reducing Devices on National Highways of Bangladesh **pp. 211-212**	H M Ahsan, S M S Mahmud and M A Hossain
16:00-16:15	ID 317	Traffic Study and Travel Forecast Case Study: Traffic Survey at Bekutia Bridge over Khocha river ar Perojpur-Jhalokathi Road **pp. 213-214**	S Munira, K Maina and H B Hasan
16:15-16:45	**Coffee Break**		

18th Technical Session-March 24, 2012	Structural Engineering – IV	Venue: Seminar Room C	
Session Chair: Prof. Dr. Quazi Azizul Mowla, BUET			
Time	**Paper ID**	**Paper Title**	**Authors Name**
14:15~14.30	ID 274	Corrosion Effect on Strength and Covering of Reinforced Concrete Structure **pp. 273-274**	M K Howlader, M A Nayeem and M H Rashid
14:30~14:45	ID 277	Effects of Aggregate Type on Specific Heat of Concrete **pp. 275-276**	M K Howlader, M H Rashid and T Haque
14:45~15:00	ID 283	Strengthening and Performance Evaluation of Masonry Shear wall using Different FRPs **pp. 277-278**	A Rahman and T Ueda
15:00~15:15	ID 286	Properties of Fiber Reinforced Concrete using MS Nail **pp. 279-280**	C Mondol and M A Rahman
15:15~15:30	ID 287	Optimum Design Criteria of Tall building for Different Subsoil Condition **pp. 281-282**	S M Alimuz-Zaman and A Rahman
15:30~15:45	ID 288	Experimental Investigation of Shear strength for Brick-Mortar Interface for Different Mortar Strength **pp. 283-284**	K Ahmed and M A Rahman
15:45~16:00	ID 292	Effect of Sulfate Rich Environment on the Performance of Cement Mortar **pp. 285-286**	M K Hasan, M H Rashid
16:00-16:15	ID 293	Performance of Cement Mortar in Acidic Environment **pp. 287-288**	M K Hassan, M H Rashid and S Parvin
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		Session Chair : Dr. Kh. Mahbub Hasan, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
16:45~17:00	ID 106	Evaluating the Adaptation Strategy of Livelihood Pattern in the Water Logged Area of South-West Coastal Bangladesh **pp. 109-110**	S R Mollick and A K Azad
17:00~17:15	ID 142	Assessment of Heavy Metal Contamination & Sediment Quality in the Turag River, Bangladesh **pp. 111-112**	M D Hossain, N Ferdousi and T Mahmud
17:15~17:30	ID 159	Assessment of Heavy Metal Contamination & Sediment Quality in the Sitalakhya River on the basis of Geo-accumulation index, Bangladesh **pp. 113-114**	M D Hossain, S Islam and K Nahar
17:30~17:45	ID 290	Heavy Metal Assessment in Sediment of Shitalakkhya River **pp. 115-116**	S Ahmed and F T Jahara
17:45~18:00	ID 304	Assessment of Water Quality Index Of Water Bodies Along Joydevpur-Chandra-Tangail-Hatikamrul Road **pp. 117-118**	R M Chowdhury, M N Naser and S Y Muntasir
18:00~18:15	ID 311	Removal of COD of Reactive Dyes using Alum **pp. 119-120**	M R Khan, Z B Mukhlish, T Kanti Deb, S Y Sony and M RKarim
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18:30~18:45	Other Technical Session running		
19:00-22:00	**Closing Ceremony and Conference Dinner**		

20th Technical Session-March 24, 2012		Transportation Engineering – III	Venue: Seminar Room B
		Session Chair : Dr. Md. Rokonuzzaman, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
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17:00~17:15	ID 308	Vehicle Aggressivity in Bangladesh: Case Study on Large Truck **pp. 217-218**	H M Ahsan, S M S Mahmud and M M R Bhuiyan
17:15~17:30	ID 309	Impact of Direct Local Access Road with the Classical National Highways in Bangladesh: A Case Study of Dhaka-Mawa Highway **pp. 219-220**	H M Ahsan, S M S Mahmud and M A Goni
17:30~17:45	ID 201	A GIS based Approach to Combine the Waterway and Road Network for the Solid Waste Management of Dhaka City **pp. 221-222**	M S Islam, Abdullah-Al Muyeed, L Pervin and M T Hasan
17:45~18:00	ID 329	Production Of Citric Acid From Oil Palm Empty Fruit Bunches Through Solid State Bioconversion In Tray Fermentor **pp. 223-224**	M N Bari and M Z Alam
18:00~18:45	Other Technical Session running		
19:00-22:00	**Closing Ceremony and Conference Dinner**		

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		Session Chair: Prof. Dr. Md. Abul Bashar, KUET	
Time	**Paper ID**	**Paper Title**	**Authors Name**
16:45~17:00	ID 315	Fire Vulnerability Assessment of High-Rise Apartments in Chittagong City: A Case of Residential Area **pp. 093-094**	S Islam, N Sakib, K M B Ali and M I Sarwar
17:00~17:15	ID 253	Water for Environmental Sanitation in the Slums of Khulna City **pp. 095-096**	T K Roy, U Kumar and S M T Islam
17:15~17:30	ID 255	Study on Biogas Generation from Weeds **pp. 097-098**	M M Islam, A K M M Islam, M A Rab, M Manjur-A-Elahi and Q H Bari
17:30~17:45	ID 116	Water Based Urbanization: An Analytical Study on Dhaka **pp. 099-100**	Q A Mowla
17:45~18:00	ID 225	Characterization and Treatment of Landfill Lysimeter Leachate: Coagulation Process using Poly-Aluminum Chloride and Ferric Chloride **pp. 101-102**	I M Rafizul, I Rahaman and M Alamgir
18:00~18:15	ID 281	Assessment of Heavy Metal Contamination & Sediment Quality in the Turag River, Bangladesh **pp. 103-104**	M D Hossain, S B Salam and T Islam
18:15~18:30	ID 108	Experimentation of Self Turning Reactor (STR) Composting System in Thailand **pp. 105-106**	A Al-Muyeed and Tetsuya Ishida
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19:00-22:00	**Closing Ceremony and Conference Dinner**		

Technical Papers

Environmental Engineering

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

FOREST – A NATURAL DEFENSE TO THE CLIMATE CHANGE IN BANGLADESH

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ABSTRACT

Bangladesh is the most vulnerable country in the world to human induced changes in the global climate. The possible impacts of climate changes in Bangladesh through the sea level rise, temperature rise, salinity rise and rainfall rise, and many other things. Climate change also will have significant impacts on water resources as there is very close connection between the climate and hydrological cycle. Rising in temperatures will increase evaporation and lead to increases precipitations, frequent and extreme floods and droughts in different regions and different times. Climate change and forests are significantly interlinked components in biosphere. Now, 17% of land covered with forest which is lower than minimum requirement (25% area) of a country. In the severe attack of SIDR, AILA, the forests of southern part of Bangladesh especially Sunderbans is act as a natural defense. Due to rapid population growth, urbanization, agricultural activities destroyed trees/forests can which increase greenhouse gas in the atmosphere. In this way our rich biodiversity has been seriously degraded due to human induced problems. The need for forest conservation is being increasingly realized during recent days. Conservation and sustainable use of forest may be key force to counteract the climate change related problems. The study suggested that systematic and proper planning and implementation of forest project is urgent need to combat the climate change.

Keywords: *Forest, Bangladesh, Groundwater quality, Climate change*

1. INTRODUCTION

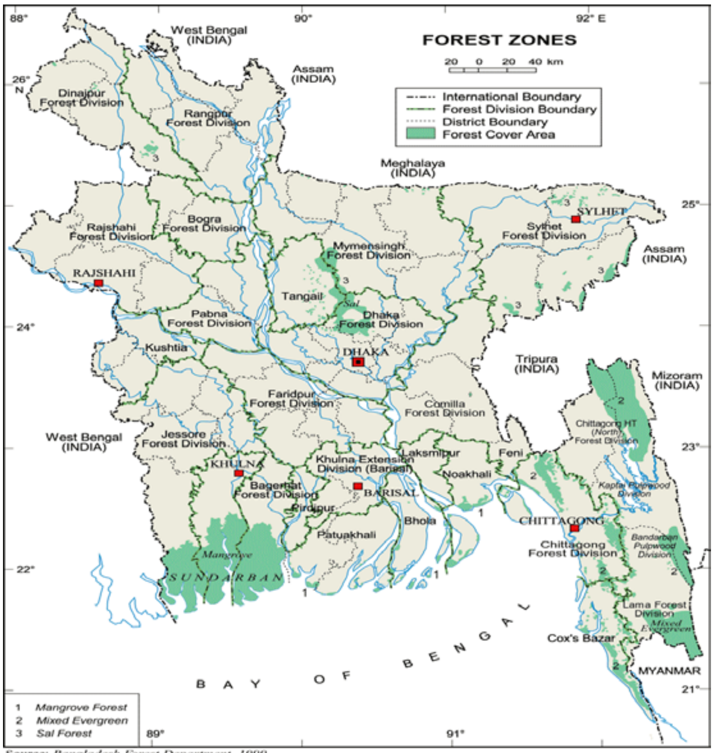
Bangladesh is one of the most vulnerable countries to the impact of climate change. Approximately a third of the Earth's land mass is covered by forest, which supports water protection and 80% of terrestrial biodiversity. Forests are not only ecological assets, but they also sustain economic growth by providing timber and non-timber consumptive goods and livelihoods for approximately one-fourth of the world's population. In 2004, trade in forest products was estimated at $327 billion. Therefore, uncontrolled deforestation has devastating consequences for the environment and the global economy. Forested land is under increasing pressure due to high population growth, rising consumption rates for timber and non-timber products, unsustainable harvesting and economic development policies that promote expansion agriculture into marginal lands. As a consequence, deforestation has increased at an alarming rate of 13 million hectares per year, which is about the size of Alabama. Deforestation and forest degradation account for approximately 18% of global greenhouse gas (GHG) emissions, more than the entire global transportation sector and second only to the energy sector. The continued loss of forests in the world not only threatens livelihoods, but also the environment.

One way to constrain the impacts of climate change is to reduce emissions from and through the forest sector as a complement to ongoing climate change policies. Collaborative forest rehabilitation efforts around the world are a start, but they cannot be done by governments and the private sector alone. Drastic changes in earth's climate systems both on global and regional scales, particularly after the industrial revolution have been recognized. The global climate change, which assessed at the macro level, is the outcome of myriads of micro level assessments, happening at the regional and local levels. Forests, when sustainable managed, can have a central role in climate change mitigation and adaptation. By strengthening forest management practices, FAO supports countries to achieve sustainable forest management, which is an effective framework for forest based climate change mitigation and adaptation. Sustainable forest management also contributes to food security, poverty alleviation, economical development, and sustainable land use, in the wider context of sustainable development. Good forest management secures the survival of forest ecosystems and enhances their environmental, socio cultural and economic functions. It can both maximize forests' contribution to climate change mitigation and help forests and forest-dependent people adapt to new conditions caused by climate change. Improved forest management practices for climate change mitigation and adaptation should be planned and implemented in tandem, as they are closely linked. Forests are, of course, managed not only for climate change, but for multiple, usually complementary, objectives: production of goods, protection of soil, water and other environmental services, conservation of biodiversity, provision of socio-cultural services, livelihood support and poverty alleviation. Accordingly, climate change mitigation and adaptation efforts must provide synergies and be balanced with other national and local forest. The objective of the study is to highlight various ways of the forest can play an important role to counteract impact of climate change which will be natural defence in Bangladesh.

2. METHODOLOGY

2.1 Study area

Bangladesh occupies a unique geographic location (20° 34‘ N- 26° 38‘ N latitudes to 88° 01‘ E -92° 41‘ E longitudes) spanning a relatively short stretch of land between the mighty Himalayan mountain chain and open ocean. The broad physiographic regions are classified as – flood plains occupying about 80%, terrace about 8% and hills about 12% of the land area with a population of 14.23 million, a density of 965 persons per sq km (Population census, 2011). Bangladesh enjoys generally a sub-tropical monsoon climate. Winter begins in November and ends in February. In winter, temperatures fluctuate from minima of 7.22^{o}C-12.77^{o}C to maxima of 23.88^{o}C-31.11^{o}C. The monsoon starts in July and continues until October. The monsoon accounts for 80% of the total rainfall. Average annual rainfall varies from 1,429 mm to 4,338 mm (BBS, 1996). Bangladesh, being a tropical country, enjoys a wide range of bio-diversity covering both wild and cultivated land. Of the total area of Bangladesh (147,570 sq. km.), agricultural land makes up 64%, forest lands account for almost 18%, whilst urban areas are 8% of the area. Figure 1: map of Bangladesh showing forest distribution.



Source: Bangladesh Forest Department, 1999

Figure 1: Map of Bangladesh showing forest distribution

Water and other land uses account for the remaining 10%. Of a total area of 2.46 million hectors (ha) under forest, about 1.46 million ha are under the management of the Forest Department. Approximately 0.73 million ha are depleted and denuded state-owned forests, known as *unclassified state forests*, under the control of the civil administration and subject to various disturbances, particularly through shifting cultivation by tribal people, since time immemorial and the remaining 0.27 million ha are homestead forests (Table 1).

Table 1: Forest classification in Bangladesh (Laskar, 1998)

Organization-wise distribution	**Area of Forests (million ha)**	**Percentage of forest land against total area of the country**
Forest managed by Forest Department (National Forests)	1.46	10.15
Unclassified state forests (managed by civil department)	0.73	5.06
Village homestead	0.27	1.88
Total	2.46	17.09

National production forest is located in the hilly region of Chittagong, Cox's Bazar & the Chittagong hill tracts and also in the tidal plain to the Southwest and to a small extent in the Modhupur and Barind tracts. These are not only inadequate and unevenly distributed throughout the country, but also yield is very low. Due to heavy population pressure, national forest is not only shrinking but is being depleted through theft of forest produce, encroachment and uncontrolled grazing. About 0.27 million ha of homesteads are located in 68,000 villages. Such areas provide a fragmented forest resource base. Due to the population explosion and high prices of forest produce, these village woodlots are also being rapidly depleted. Consumption rates of timber, 0.01076 m^3 per head and firewood, 0.0654 m^3 per head, are very low compared with those of other developing countries; yet the annual felling rate greatly exceeds the annual increment, leading to profound adverse effects on the environment. If the rate of depletion is allowed to continue, particularly in the northern part of the country, all unclassified state forests will be converted into barren savannah-land. In that condition, it would be extremely difficult to rehabilitate the land suitable for human use. The state owned forests (Table 2) of Bangladesh are distributed in three zones: a) Hill forests in the greater districts of Chittagong, Chittagong Hill Tracts, and Sylhet; b) Inland forests in the central and northern zones; and c) Littoral forests in the delta and coastal regions.

Table 2: Status of the state-owned forest land (in ha) in Bangladesh (Banglapedia, 2005)

Forest type	**Reserve forest**	**Protected forest**	**Vested forest**	**Acquired forest**	**BWDB and khas**	**Unclassed state forest**	**Total**
Hill	5,94,383	32,303	2,636	11,004	-	721,344	13,61,670
Inland	68,140	2,689	19,985	31,198	-	-	1,22,012
Littoral	6,56,579	-	-	6	1,01,526	-	7,58,111
Total	13,19,102	34,992	22,621	42,208	1,01,526	7,21,344	22,41,793

By the sustainable forest management, we have to maintain and enhance the economic, social and environmental value of all types of forests, for the benefit of present and future generations. Now management of forests is imperative for can combat the climate change which is a crucial issue for us and for increasing the extent of forests as carbon stocks, while still considering the multiple forest functions and dimensions. The objective of this article is discussion on how in future forest can contribute to natural defence to the climate change in Bangladesh.

2.2 Data Collection

The study has been conduct by the secondary data. Secondary data have been collected from different sources like journals, books, published documents and the Internet.

3. DISCUSSION

3.1 Natural Disasters and The Climate Change Challenge in Bangladesh

Bangladesh is one of the Least Developed Countries (LDCs) in the world. Bangladesh is prone to many kinds of natural disasters. The case of Bangladesh illustrates the injustice of climate change; this densely populated country, accommodating 2.4% of the world's population, is responsible for only 0.1% of global carbon emissions (HDR, 2007-08). Of these natural disasters, the most important ones to mention are tropical cyclones with associated storm-surges, floods, droughts, tornadoes and river-bank erosions. Besides these disastrous weather systems the occurrences of earthquakes at times make significant impact both on social life and topography of the country. Tropical cyclones are frequent in the Bay of Bengal. Immediately pre-monsoon and post-monsoon periods are the seasons when cyclones and depressions form in the Bay of Bengal. Bangladesh has the worst record of cyclones and storm surges in the world. They destroy crops, damage infrastructure, homes and vital installations, and cause widespread health hazards for the people. Occasionally, tropical cyclones also cause enormous numbers of casualties. The cyclone disasters in 1970 (300,000 dead) and 1991 (138,000 dead) are among the worst natural disasters in the world. On November 15, 2007, Cyclone Sidr struck Bangladesh. Winds of up to 285 KPH accompanied by a massive tidal surge killed over 3300 people, and destroyed huge numbers of houses as well as crops, roads and infrastructure. Cyclone Sidr was by no means minor. Some of the damage included: 539,744 houses completely destroyed and 885,280 partially damaged; 237,782 ha of farmland completely destroyed, 595,612 ha partially damaged; 1,157,939 livestock perished; 4,065,316 trees (many of them fruit-bearing) lost; 2,240 schools destroyed, 11, 490 damaged; 1,523 km of road destroyed, 4,048 km partially damaged; 1,687 bridges damaged or destroyed, and 1,406 km of embankment (World Vision, 2009 & Times Online, 15 November 2007). Floods in Bangladesh are a complex phenomenon. They pose enormous threats to the population, but at the same time, moderate floods contribute to the fertility of the land. Extensive river floods cause great disruption and damage to infrastructure and loss of crops. Floods indirectly contribute to the concentration of land ownership and wealth as small landowners are forced to sell their property as a result of flooding. Bangladesh has always experienced some degree of flooding. In August-September, 1988; Bangladesh experienced an unprecedented flood causing loss of 1657 human lives and again during July-September, almost two-third area of the country was inundated causing widespread damage to the economy of the country ever before. Bangladesh’s vulnerability to floods is two-fold. Firstly, rising sea levels are inundating mangrove swamps and salinity is becoming a major problems. A system of dykes originally built to protect the coastline against storm-related tidal surges may hold back the rising sea (though not the salinity) but they might also prevent freshwater floods from draining easily into the Bay of Bengal. Hence protection from saltwater floods actually increases vulnerability to freshwater floods (Justin Huggler, 2009). Most of Bangladesh lies less than 10m above sea level. The country could be described as one vast, low-lying river delta; it’s vulnerable to floods from the Ganges (known as the Padma in Bangladesh), Brahmaputra and Meghna rivers. As climate change takes its toll in South Asia, the melting of glaciers that feed those rivers is likely to exacerbate annual floods before potentially leaving millions of people with an acute water shortage. Virtually every source of income relied upon by the poor population of Bangladesh is therefore under direct threat as a result of climate change. UNDP predicts that sea level rise alone could create 25 million Bangladeshi climate refugees (UNDP, 2008). Bangladesh experiences drought conditions at some intervals of time. Records show that Bangladesh had, in the recent past, drought conditions having disastrous crop failure. Droughts of 1957 and 1972 were of severe nature also. Crop failure by drought comes as a significant strain also to the socio-economic structure of Bangladesh. Along the courses of the mighty rivers, the Jamuna, the Meghna, etc. erosion every year takes away chunks of land causing displacement of large number of people and losses of properties. Due to recurrence of such erosion displaced people are forced to come to cities for their earning. This adds to the growth of slum areas, scarcity of land in the cities and also creates various social problems. Tornadoes cause localised devastation and demand an immediate response. A severe tornado hit Tangail district of the country on 13th May 1996 causing 540 deaths and injuring about 34000 people. List of major natural disaster in Bangladesh since 1970 are shown in Table 3. So, Bangladesh is natural prone zone of the world.

Table 1: A list of major natural disaster in Bangladesh (Rashid, FED, Dhaka)

Year	Type of Disaster	Death
1970	Cyclone	3,00,000
1972	Drought	+
1973	Cyclone	103
1974	Floods followed by famine	30,000
1978/79	Drought	+
1983	Cyclone	343
1985	Cyclone	11,069
1987	Floods	1,657
1988	Floods	2,379
1988	Cyclone	5 708
1989	Drought	+
1991	Cyclone	1,38,868
1998	Flood	918
2007	Flood	
15 Nov, 2007	SIDR	>3300
2008	Aila	

\+ No Data found

3.2 Forests A natural defence to climate change

The most essential service provided by forests is their role as a natural carbon sink or 'the green lungs of the world'. As part of the global carbon cycle, forests can absorb, store and convert vast amounts of carbon that would otherwise escape into the atmosphere as CO_2. The conservation and rehabilitation of these natural carbon stores are essential to climate change mitigation efforts. So we consider forest a natural defence to the climate change in Bangladesh. Carbon sequestration by forests has attracted much interest as a mitigation approach, as it has been considered a relatively inexpensive means of addressing climate change immediately. Around 13 million hectares of forest were converted to other uses or lost through natural causes each year between 2000 and 2010. The world has an estimated 850 million hectares of degraded forests, which could potentially be restored and rehabilitated to bring back lost biodiversity and ecosystem services, and, at the same time, contribute to climate change mitigation and adaptation. Forest management practices that increase carbon sequestration include: afforestation, reforestation and forest restoration; increase of tree cover through agroforestry, urban forestry and tree planting in rural landscapes; enhancement of forest carbon stocks (in both, biomass and soils) and sequestration capacity through the modification of forestry management practices. Deforestation and forest degradation, whether due to human activities or natural causes, result in carbon stock reductions and greenhouse gas emissions, as well as loss or impairment of other forest goods and services, threatening livelihoods, environmental functions and other socio-economic values. Forest conservation endeavours to rectify such deleterious impacts that cause forest degradation, and ultimately deforestation. Forest threats include fires, pests and diseases, poor management and harvesting, overexploitation, grazing and other disturbances. Over 40% of the world's O_2 is produced from the rainforests. Forests contribute to the balance of O_2, CO_2 and humidity in the air. A tree releases 8-10 times more moisture into the atmosphere than the equivalent area of the ocean. Forests protect watersheds which supply fresh water to rivers. Deforestation leads to soil erosion and rivers being silted, which reduces access to clean water. Forests are home to over 80% of terrestrial biodiversity. Forests build resilience to natural disasters nearly 330 million hectares of forest are designated for soil and water conservation, avalanche control, sand dune stabilization, desertification control or coastal protection (FAO, 2010). Mangrove forests such as Sunderban which act as a barrier against tsunamis, cyclones and hurricanes. Forests cover 31% of global land area and cover combat land degradation and desertification by stabilizing soils, reducing water and wind erosion and maintaining nutrient cycling in soils.Forests are a key part of the climate change solution due to carbon in forests exceeds the amount of carbon currently in the atmosphere. FAO, 2010 estimates that the world's forests store 289 gigatonnes (Gt) of carbon in their biomass alone. 17.4% of global greenhouse gas emission resulted from deforestation and forest degradation. Forests offer the quickest, most cost-effective and largest means of curbing global emissions.

3.3 Sustainable practices of forest management and use

Sustainable management practices can ensure that productive or multipurpose forests continue to store carbon and maintain their capacity to provide other goods and services for the benefit of current and future generations. Management practices need to be actively planned, tailored to each specific ecosystem and evolving situations, so as to avoid overexploitation and forest degradation. Forest management plans and practices will also need to be adapted to modifications caused by climate change, both gradual and abrupt. Adaptive forest management will be essential to address arising challenges and reduce forest vulnerability. Adaptation measures might include, for example, selection of pest-resistant or drought-tolerant varieties, use of stocks from a range of provenances, underplanting of genotypes of species adapted to expected new climate conditions or assisted natural regeneration of functional species. The measures need to be adapted to forest condition (primary, secondary, degraded) and the specific site which shown in Figure 2.

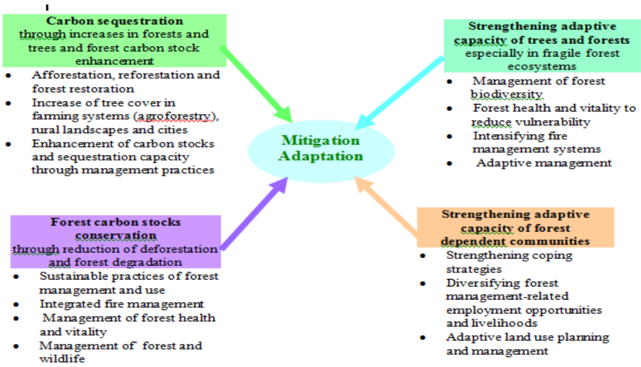


Figure 2: Strategic framework for forests and climate change.

Increasing forest area and density through afforestation, reforestation and forest restoration results in increased absorption of CO_2 from the atmosphere. Once the trees are harvested, new trees can grow in their place and continue to sequester carbon. Planted forests today cover around 264 million hectares and absorb an estimated 1.5 gigatonnes of carbon from the atmosphere each year. The rates of carbon sequestration on forest land depend on the management practices adopted, the tree species involved, and the geographic area covered. For the conversion of agricultural land to forests by way of afforestation, for example, sequestration rates will vary considerably depending on the region and species involved. In accordance with tree growth, carbon sequestration rates share a standard pattern of initially rising rates followed by gradually declining rates. On average, a planted forest in a temperate zone can sequester about 4 tonnes of carbon per hectare each year.

3.4 Management of fragile forest ecosystems

Some forest ecosystems are particularly vulnerable to climate change in Bangladesh:

- mountain forests and watersheds;
- dryland forests;
- coastal forests.

These ecosystems, with their unique features and resources, also have an important roe for climate change mitigation on a global scale, as well as providing many other evironmental, economical, social and cultural functions. Management measures are often critical to reduce their vulnerability and facilitate their adaptation to climate change. Mountains are among the regions most affected by climate change. The expected increase in temperature and extreme weather events will amplify hazards in mountains worldwide and change the hydrological cycle, in which mountains play a key role. Sound forest management and afforestation of degraded areas reduce erosion, increase slope stability and hence contribute to the provision of essential environmental services such as a regulated water flow and protection against natural hazards. Sustainable mountain development and collaborative and integrated approaches to watershed management are therefore vital for climate change mitigation and adaptation. Climate change is a huge concern in the drylands, as it is expected to exacerbate degradation caused by human activities. Global warming is expected to cause a decrease in rainfall and an increase in extreme weather conditions such as long periods of drought (with few exceptions), an increase in the frequency and intensity of wildfires and loss of biodiversity. Severe water scarcity and increased desertification are likely, thus causing a vicious circle of forest and land degradation. Climate-related changes are likely to result in species range shifts and altered tree productivity, adding further stress to forest ecosystems and putting at risk the livelihoods of local communities that rely on the forest for their survival. Coastal forests – including mangroves, beach forests, and lowland moist tropical forests – have a number of important roles: environmental: protecting against shoreline erosion and surge-tide damage, providing wildlife refuges, safeguarding water quality, stabilizing land, trapping sediments, providing nutrients to inshore waters; social: protecting human settlements, offering aesthetic and recreational values; economic: as productive ecosystems, providing income-generating opportunities for the local population; providing nursery grounds for many commercially valuable fish and shellfish species. The risk of sea-level rise due to climate change, combined with existing threats caused by population pressure such as overexploitation, conversion to other uses (ports, resort development and other infrastructure as well as aquaculture and rice cultivation) and marine pollution, poses a threat for many natural coastal forests. Their protection and restoration are important to mitigate the impact of climate change, while adaptive management will need to be adopted to secure their existence.

4. CONCLUSION

Bangladesh is the most vulnerable country in the world to human induced changes in the global climate. The possible impacts of climate changes in Bangladesh through the sea level rise, temperature rise, salinity rise and rainfall rise, and many other things. Climate change also will have significant impacts on water resources as there is very close connection between the climate and hydrological cycle. Bangladesh illustrates the injustice of climate change. Of these natural disasters, the most important ones to mention are tropical cyclones with associated storm-surges, floods, droughts, tornadoes and river-bank erosions. The most essential service provided by forests is their role as a natural carbon sink or 'the green lungs the world.' As part of the global carbon cycle, forests can absorb, store and convert vast amounts of carbon that would otherwise escape into the atmosphere as CO_2. The conservation and rehabilitation of these natural carbon stores are essential to climate change mitigation efforts. So we consider forest a natural defence to the climate change in Bangladesh. The study suggested that systematic and proper planning and implementation of forest project is urgent need to combat the climate change.

ACKNOWLEDGEMENTS

The authors would like to thank Mst. Nahida Aziz for advice to write this manuscript.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CONSIDERATION OF INTEGRATED INFLUENCE SCENARIO OF CLIMATE CHANGE AND AIR POLLUTION AT CITY LEVEL TO FRAME OUT URBAN POLICY AND STRATEGY

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ABSTRACT

The impact nature of climate change at city level is diverse and context oriented. In different contexts the impacts of climate change have different dimensions. Apart from the context and relevant impact analysis this study is focused to embody the general atmospheric expression of climate change and the influence scenario of Air pollution on the general impact scenario of climate change. The study has also tried to draw out the influence matter of climate change on the air quality. In today's science and environment the issue of climate change is a burning one. The impact of climate change has the basic focus to the physical content and expression of the atmospheric condition. The root of climate change is in the alteration of atmosphere. The composition of atmosphere is changing due to the impact of Green House Gas (GHG). The changes in atmosphere have likely influenced temperature, precipitation storms and sea level. In this study the nature of this influence is discussed in different levels and how are these atmospheric influences aggravated due to air pollution and vice versa. It is found that the air pollution can influence the climate change situation by aggravating and reducing the influenced effects. There is very small evidence that air pollution can change the emission of GHG. So in integrated situation the concern is about to cope and to minimize the integrated effect. In this case (integrated level) the policy and tools should be ***focus to the effects*** *instead to the emission standards of the air and GHGs.*

On the other hand the issue of air pollution is local. In a contextual situation it is a regional as well as a global problem. Especially the influence of climate change situation creates effects on the emission situation of Air Pollutants. In some cases it becomes as a fact to reduce the emissions and in some cases it acts in opposite direction. So in this case the first focus goes to the ***emission situation and standards*** *and the policy and planning tools have to deal with the emission control policy and strategy.*

Keywords: *Climate change, Atmospheric Expression, Air pollutants, GHGs Emission, Climate Change Impact*

1. INTRODUCTION

Climate change is the most important issues in today's technological and industrial activity based productive world. It is considered as a global problem due to the global problem of rising global temperature, sea level rise, and dramatic changes in weather patterns. The severity of climate change varies according the geographical location of the area. Some places of the earth are more vulnerable than the other parts of the world from this climatic expression. The scenario of climate change over the world is depicted by the Intergovernmental Panel on climate change (IPCC). In its fourth assessment report, 2007 the issue of climate change are addressed from different angles. Some very basics and preliminary are addressed here to conceptualise the scenario of the climate change in the world. The global mean temperature has up to 2005 risen by almost .8°C and the change expected by 2100 is as large as glacial-interglacial changes in the past, which were commonly spread out over 10000 years. The release of CO_2 gases in earth's atmosphere are considered as one of the main causes of global climate changes. Atmospheric CO_2 has now reached levels unprecedented during the past several million years. Principal threats are greatly reduced biodiversity (species extinction), changes in the atmospheric precipitation pattern, more frequent weather extremes and sea level rise.

Air quality and air pollution has become an increasingly an urban problem throughout the world as urbanization and industrialization have intensified during the last few decades. More than 3 billion people—about half the world's population—are now concentrated in urban areas, and by 2010 the global urban population is expected to swell to more than 4 billion (UNEP, 1999).

By one estimate, more than 1 billion people are currently exposed to harmful levels of air pollution (Schwele, 1995). Severe particulate air pollution is a chronic problem in much of Asia, primarily as a result of coal combustion in factories and power plants, and the use of coal and wood for cooking and home heating. Motor vehicles are an increasingly important contributor to air pollution in much of the world, with more than 600 million vehicles in use, a number that could double within the next 25 years (Dunn, 1996). Automobiles are the dominant source of air pollution in many Latin American cities, including Sao Paulo, Santiago, and Mexico City, where they have had to restrict automobile use in an effort to manage severe air pollution episodes (UNEP, 1999). U.S. air quality, as measured at thousands of monitoring stations across the country, has shown steady improvement over the past 20 years due in part to the implementation of air quality regulatory programs and cleaner technologies for motor vehicles and stationary pollution sources. There are, however, still many areas of the country that are not in compliance with ambient air quality standards for pollutants such as ozone and PM (EPA, 1999).

In such a context this study is conducted to find out the inter influence of air pollution and climate change influence at different levels and how are the atmospheric changes (due to climate change) aggravated due to air pollution and vice versa.

2. METHODOLOGY

The study was conducted through literature review. Number of relevant journal papers, books and other documents on the subject were studied and analysed and finally the influence relation of climate change and air pollution have been framed out. Based on the influenced relation a theoretical flow diagram of influence nature was developed. In the following the stepwise analysis of different literatures are described to conduct the total study.

2.1 Scientific Understanding of Climate Change and Air Pollution

At the beginning of the study the attempt was given to understand the scientific basic of climate change and air pollution. Air pollution and climate change are the two problems that are taking places at different levels. From this consideration this understanding exploration was carried out. At this stage the individual happenings of climate change and air pollution was studied.

2.2 Identification of Common Source/Causes and Drivers

This step is the continuation of the previous step. At this stage the focus was given to find out the common source/causes and drives of climate change and air pollution.

2.3 Identification of the Common Influence Nature of the Climate Change and Air Pollution

In this step the influence nature of the climate change and air pollution was identified. The object is to find out the way of influence. In case of identification of the influence different scientific research paper were gone through. Some differentiated findings were found in different papers. Beyond the existing scientific complexities the influence relationship of climate change and air pollution was tried to depict in a very simple way.

3. COMMON DOMAINS OF AIR POLLUTION AND CLIMATE CHANGE

Usually air pollution is differentiated into three broad categories: ambient, indoor, and transboundary. Ambient air refers to the air close to the ground that is in direct contact with the living world; indoor air pollution refers specifically to air within buildings, whether at the workplace or in the home; and transboundary air pollution is used to refer to pollutants that have entered the upper atmosphere and travel far from their source. Air pollution of all three types is strongly affected by climate—precipitation, wind, temperature, radiation—and thus by changes in climate or "climate change". At the same time, air pollution is thought to be one of the major contributors to the present situation of "climate change". So, there exists a scientific relation in their way of taking place.

Climate change and air pollution are closely linked, although in applied scientific research and even more in political negotiations they have been largely separated. Many of the traditional air pollutants (APs) and greenhouse gases (GHGs) have not only common sources, but may also interact physically and chemically in the atmosphere causing a variety of environmental impacts on the local, regional and global scales (Bytnerowicz et al,2007).

There are important linkages between emissions of air pollutants and climate-relevant gases. These linkages exist because: (i) air pollutants have a radioactive forcing too (ii) air pollutants and greenhouse gases have common sources (iii) control of air pollutants and greenhouse gases result in joint benefits (IIASA, 2004).

In recent years it has been increasingly recognised that air pollution and climate change are linked in several ways and they could be beneficially addressed by integrated policy (Swart et al, 2004).

The push for policy integration comes mainly from consideration of implementation costs (Raes, 2006). It is estimated that cost of reaching the 2010 air pollution objectives in the conventions Gothenburg Protocol could be reduced by at least 5 billion if European countries cut CO_2 in line with the Kyoto protocol (without CO_2 trading). Similar results have been found for China and Mexico (UNECE, 2010). This is particularly true in developed countries, where cheap air pollution control technologies are widely implemented and further reductions are likely to require structural and behavioural measures. In developing countries current economics growth and the supporting development of energy production systems, provide the opportunity to tackle air pollution and GHG emissions simultaneously (Raes, 2006).

The ancillary benefits from GHG policies in developing nations may be even more significant relative to the cost of these policies than those measured in the U.S and Europe because of lower existing levels of pollution control and lower efficiency in energy use in these countries (Burtraw and Toman, 1997). Hence, implementing climate policies, for example in order to achieve greenhouse gas (GHG) emissions targets, can significantly reduce the costs of meeting air quality targets and vice versa.

4. IMPACT SCENARIO OF CLIMATE CHANGE WITH THE INFLUENCE OF AIR POLLUTION

According to the Environmental Protection Agency (EPA, 2009) USA, recent climate change impacts are addressed in the physical environment of temperature, precipitation, storms, sea level changes and ocean acidification. In the following only the temperature and precipitation are discussed as they are influenced by the level of air quality in regional and local scale. The integrated city level impacts of those two influences of atmospheric changes will also be discussed in the following.

4.1 Atmospheric Influence: Increased Temperature

The influenced impact of atmospheric change is the increased world temperature. The temperatures are increased in different levels. Increased temperature is observed in surface level, tropospheric level. Since mid 1970s the average surface temperature has warmed about 1°F (EPA, 2009). The warming trend is seen in both daily maximum and minimum temperatures increasing at a faster rate than maximum temperature (IPCC, 2007). On the other hand tropospheric temperature is increased by .22°F near the surface and by.27°F in mid troposphere per decade within the period of 1958-2006 (NOAA, 2008). The casual relation of this increased temperature is diverse due to its multiple and complex factors. As a concluding remark of IPCC it is said that warming in global average surface temperature that has occurred since the mid-20th century is very likely a result of human activities (IPCC, 2007). This increased global temperature has diverse effects on different systems of the world. The expected rise in global temperatures will affect human health, comfort, life styles, food production, economic activity, and residential and migration patterns (IPCC, 1990).

This increased global temperature is influenced by the air pollutants aerosols. During recent years evidence has emerged showing that particles suspended in the air actually affect the radiation balance (EPA, Sweden 2009). The aerosols not only help to increase the temperature it has opposite impact to cool the climate. But the magnitude of this effect is about 30% of the magnitude of the warming effect of by CO_2 (EPA, Sweden 2009). In view of the fact that combustion is the major source of anthropogenic particles, soot, ozone and CO_2 changes in combustion will affect all its pollution components. In general, CO_2, soot and ozone are warming components, while most other particles cool the climate. According to the IPCC (2007), on a global scale soot and ozone will, produce combined global heating of about 0.8 W/m^2, while particles will have a cooling effect of about -1.6 W/m^2. IPCC (2007) assessed the total direct climate effect by aerosols to be -0.5 ± 0.4 W/m^2. It should be noted that this includes both heating by soot and cooling by the particles. The heating by soot is estimated at about 0.2 ± 0.15 W/ m^2. Recently, reports have emerged that imply much greater heating by soot (Ramanathan and Carmichael, 2008). It should be noted that the uncertainties in the estimates for soot and particles are quite large while in theory it should be possible to balance emission reductions of soot, particles and ozone so that the resulting global climate impact is small. Regions with extensive soot emissions would probably experience a cooler climate if soot emissions were substantially reduced (EPA, Sweden 2009). But, without real inclusion of aerosols in the climate Model it is nearly impossible to interpret the casual factors for regional as well as global climate changes during the last century.

4.2 Atmospheric Influence: Precipitation

The impacts of atmospheric change on precipitation also have two directions. In some places of the world it is positive and in some places of the world it has negative impacts. According IPCC 2007, Precipitation has generally increased over land north of 30°N from 1900-2005, but has mostly declined over the tropics since the 1970s. Globally there has been no statistically significant overall trend in precipitation over the past century, although trends have varied widely by region and over time. On the negative way it has become significantly wetter in eastern parts of North and South America, Northern Europe, and Northern and Central Asia, but drier in the Sahel, the Mediterranean, southern Africa and parts of southern Asia.

The resultant of this positive and negative force of global change on rate of precipitation is influenced by the air pollutants in micro scale. Air born carbon and particles usually causes the large reduction of solar radiation at the surface ($-4.4Wm^{-2}$) will result in reduced evaporation and in turn reduced precipitation. Of course this will be countered by increased precipitation from the GHGs warming. It is likely that the reduction in precipitation will occur in the tropics where the dimming is the largest and the increase in precipitation will occur in the extra tropics where the GHGs warming is larger than the tropical warming (Ramanathan and Feng, 2009).

Air Born Carbon's impact over Asia: regionally, Air Born Carbons may have played a very large role in the widespread decrease in precipitation in Africa and in South Asia (the Indian summer monsoon) and the widespread retreat of glaciers in the Hindu Kush-Himalaya-Tibetan region. The former is due to dimming and the latter is due to solar heating of elevated layers by Air Born Carbons. The best explanation for the precipitation reduction downwind of urban areas is the precipitation suppression brought about by the increased number of small particles in the atmosphere from pollution. (Jirak et al, 2006). So it is evidenced that with the global change the air pollution has significance influence in the changing rate of precipitation at regional and local level.

Table 1: Summary of the Discussion

Impacts due to atmospheric Changes of GHG	**Nature of Affects of Major Air pollutants on influenced impact of atmospheric change.**			
	NO_2	CO	SO_2	Particulate Matter (PM)
Increased Te mperature	No Direct Effect	No Direct Effect	No Direct Effect	Influence Positively and Negatively
Precipitation	No Direct Effect	No Direct Effect	No Direct Effect	Influence positively and negatively

4.3 Controlling Air Pollutants and Influence on Climate Change Impacts:

Controlling other air pollutants can positively and negatively affect the impact of climate change. On this issue a research was conducted by Ronald G. Prinn (2005) and his team under the joint programme on Science and Policy of Global Change. The findings of the conducted research are summarized in Table 2.

Table 2: Air Pollutants and Influence on Climate Change Impact

Controlled Pollutants	Impact Components of Climate Change					
	Sulfate Aerosols	OH	O_3	CH_4	Temperature	Sea Level
Capping* SO_x	Decrease	Increase	No effect	No Effect	Increased highly	Rise Highly
Capping* NO_x	No Effect	Decrease	Decrease	Increase	Slightly Increase	Negligible Rise
Capping* CO and VOC	Increase	Increase	No Effect	Decrease	Small Decrease	Negligible Decrease
Capping* NO_x,CO and VOC	No Effect	No Effect	Decrease	Increase	Decrease more than the previous one	Decrease more than the previous one
Capping* all emissions	Decrease	No effect	Decrease	Increase	Increase** (less than the case of SO_x cap)	Increase** (less than the case of SO_x cap)

* For capping the air pollutants Emissions they used the emission level of 2005, whereas CO_2 emissions are predicted for 2100. They have done this experiment at global level, northern hemisphere and southern hemisphere. Here only the global level results are depicted.
** The increasing trend seems very antic. But the writes explains as the effect of increasing rate CO_2.

From the table it is understandable that Placing caps on SO_x leads to lower sulfate aerosols, less reflection of sunlight back to space by these aerosols (direct effect) and by clouds seeded with these aerosols (indirect effect), and thus to greater radiative forcing of climate change due to solar radiation. Enhanced radiative forcing by these aerosol and CH4 changes combined leads to more warming and sea level rise. On the other hand placing caps on NO_x alone also leads to decreases in OH and thus increases in CH_4. These OH decreases and CH4 increases are lessened (but not reversed) when there are simultaneous NO_x, CO and VOC caps. Increases in CH_4 lead to greater radiative forcing. In general, placing caps on NO_x alone, or NO_x, CO and VOCs together, leads to lower ozone levels, and thus less radiative forcing of climate change by this gas. Hence these impacts on climate of the pollutant caps partially cancel each other.

5. IMPACT SCENARIO OF AIR POLLUTANTS WITH THE INFLUENCE OF CLIMATE CHANGE

The term air pollution has local, regional and in some cases global dimensions. The non local process of air pollution is affected by metrology and thus by climate change. Air pollution results from the combination of high emissions and unfavourable weather (Jacob et al, 2009). So during the era of rapid changes of climate it is necessary to understand the impacts of climate change on air quality for the purposes of decision making regarding air quality management. In this discussion it will be tried to draw out the pictures of interrelated impacts of climate change on air quality. From the different literature reviews the discussion is carried out. The main aim is to find out that how different pollutant's creation process is stimulated due to the impact of climate change.

In a very general concept according the EPA, Sweden that the air pollution is affected in number of ways by the metrological system. They are:

➔Natural emissions of bio-genic volatile organic compounds (VOC) are emitted more readily in warm and sunny conditions;

➔Sea salt and soil dust particle emissions occur more effectively in windy conditions;

➔Photochemical reactions are dependent on available solar radiation**,** which is dependent on cloudiness;

➔In the planetary boundary layer, the part of the atmosphere closest to the ground, turbulence acts to dilute pollutants;

➔ Many chemical reactions and physical phase-transformations are dependent on temperature;

➔Wet deposition, i.e. scavenging of compounds with rainfall, is dependent on amount and intensity of precipitation;
➔ Dry deposition, i.e. compounds removed from the atmosphere by attaching to terrestrial surfaces, depends on meteorology in many ways. One important factor is vertical transport through the atmospheric boundary layer, which means that it is dependent on turbulence in the boundary layer
By influencing the meteorological process climate change can interact strongly with air pollution. Due to this effect of climate change on air pollution the emission standards that leading to acceptable levels of air pollution in one year can cause severe air pollution episodes in another. The impact nature of meteorological variation on air pollutants is given in the following table. The table is derived from the Jacob, 2009.

Table 3: Impact Magnitudes of Climatic Variables on Air Pollutants

Variable	Ozone	PM
Temperature	++	-
Regional Stagnation	++	--
Wind Speed	-	-
Mixing Depth	=	--
Humidity	=	+
Cloud Cover	-	-
Precipitation	=	--

Source: Jacob,Darrell . Winner, 2009

Results are summarized as consistently positive (++), generally positive (+), weak or variable (=), generally negative (-), and consistently negative (--).
The sum up of the discussion can be written as the sensitivity of ozone to climate change is particularly high in urban areas, reflecting the high potential for ozone formation. This is not true for the background ozone (the ozone that is formed naturally in normal condition). Background ozone is not correlated with temperature, and is expected instead to decrease in the future climate as a result of increasing water vapour (Jacob et al, 2009).
On the other hand the response of PM to climate change is more complicated than that for ozone because of the diversity of PM components, compensating effects, and general uncertainty of applied model’s (In this case the Model is GCM=Global Circulation Model) projections of the future hydrological cycle. Observations show little useful correlation of PM with climate variables to guide inferences of the effect of climate change. Rising temperature is expected to have a mild negative effect on PM due to volatilization of semi-volatile components (nitrate, organic), partly compensated by increasing sulfate production. Increasing stagnation should cause PM to increase. Precipitation frequency, which largely determines PM loss, is expected to increase globally but to decrease in Southern North America and Southern Europe. PM is highly sensitive to mixing depths but there is no consensus among models on how these will respond to climate change.

6. CONCLUSION

In urban planning the consideration of these uncertainties and probable casual relation should be considered. The establishment of this causal relationship and concerning policy application will help to make cost-effective decision to apply climate change policy in urban planning scheme with relation to the air pollution. From the discussion it is clear that the air pollution can influence the climate change situation by aggravating and reducing the influenced effects. There is very small evidence that air pollution can change the emission of GHG. So in integrated situation the concern is about to cope and to minimize the integrated effect. In this case (integrated level) the policy and tools should be **focus to the effects** instead to the emission standards of the air and GHGs. On the other hand the question of air pollution is local. In a contextual situation it is a regional as well as a global problem. Especially the influence of climate change situation creates effects on the emission situation of Air Pollutants. In some cases it becomes as a fact to reduce the emissions and in some cases it acts in opposite direction. So in this case the first focus goes to the **emission situation and standards** and the policy and planning tools have to deal with the emission control policy and strategy.
In the following schematic diagram presents the influence scenario of Air Pollution and Climate change. The interacted relationship is presented in the same diagram.

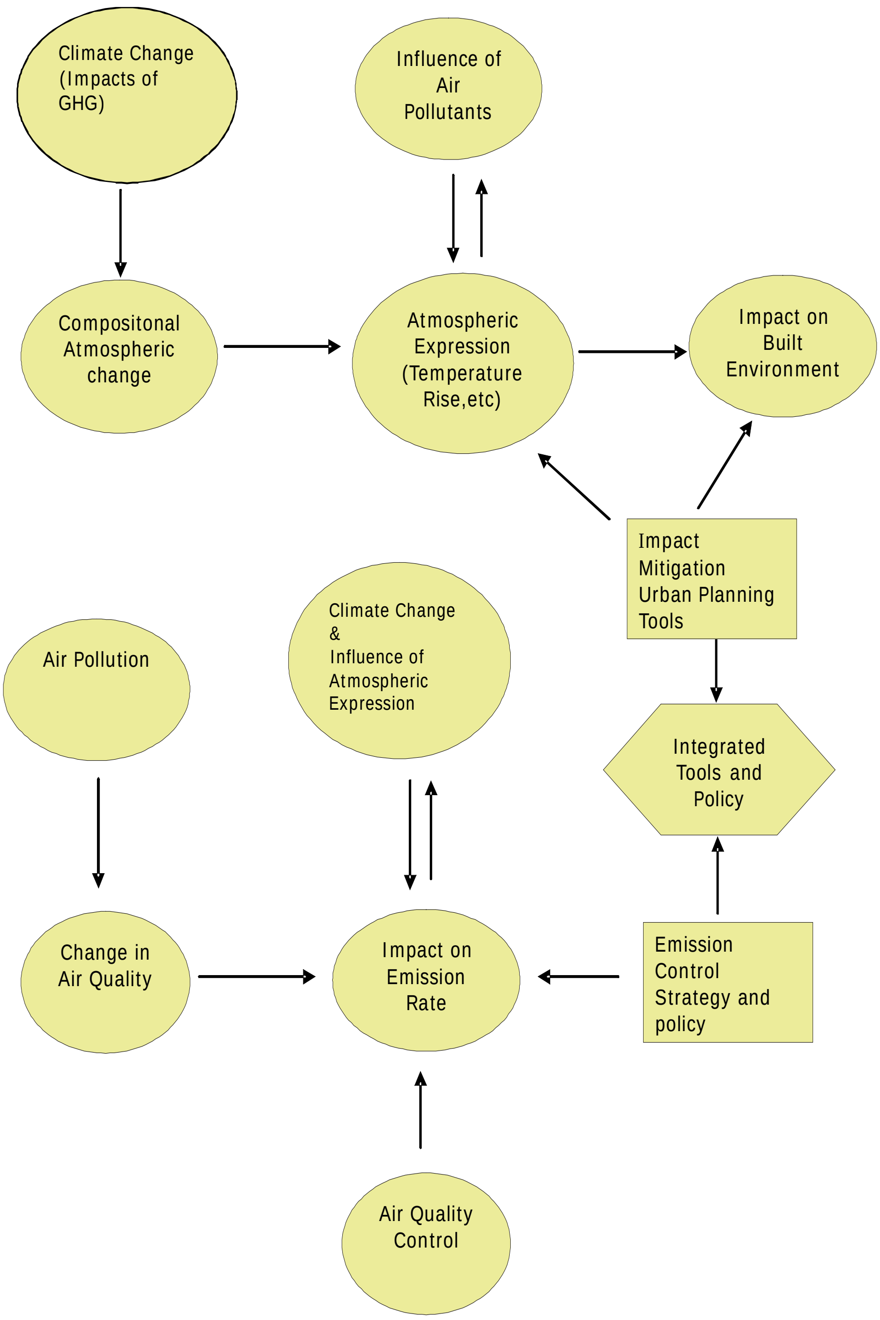

Figure 01: Integrated Influence Scenario of Climate change and Air Pollution
(Source: Author Derived by doing the analysis of different Studies)

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

THE IMPACT OF CLIMATE CHANGE ON AGRICULTURE IN BANGLADESH

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ABSTRACT

Alteration in the climate has a subsequent impact on nature. Consequently, works pertaining to agriculture is seasonally affected. Among the many faces of the eastern-world, Bangladesh has its due share of impact owing to climate change. The Agricultural system is highly sensitive to weather and climate variables such as temperature, precipitation, light and extreme events as well as low capacity of adaptation. This system is already under threat in much of the north-western side of Bangladesh due to rising temperatures and continuous drying. Increasing drought periods challenge the viability of agriculture in some regions. This article reviews the scope and nature of the climate change challenge and evaluates the impact of climate change on the agriculture of Bangladesh. From the review, it is perceptible that the climate of Bangladesh is already showing significant changes, apparent by changes in average temperature, in the amount of rainfall and in the intensity and frequency of extreme events. The review also revealed that according to a number of GCM's, the extreme events like cyclone, floods, and storm surges are expected to continue in the 21st century. Climate change will negatively affect the agriculture production of Bangladesh. In particular, as a result of current and expected climate change, the area suitable for agriculture, the length of growing seasons and yield potential, especially in the north-western and southern coastal (due to salinity intrusion) regions, are suspect to decrease. These impacts will affect all components of food security (food availability, food accessibility, food utilization and food stability) of millions of people and hence increase the risk of hunger in the country. The review thus once again proves the validity of the general consensus that Bangladesh is the most susceptible country to climate change.

Keywords: *Climate change, Agriculture, Food security, Yield potential, Bangladesh.*

1. INTRODUCTION

Bangladesh is located in the tropical monsoon region. Its climatic condition is characterized by high temperature, heavy rainfall, repeatedly excessive humidity, and considerably distinguished seasonal diversification. In Bangladesh, from the climatic point of view, three different seasons can be observed- the cold dry season (November - February), the pre-monsoon hot season (March - May), and the rainy monsoon (June - October). The month, March is also referred to as the spring season and some other period from mid-October to mid-November may be called the autumn season (Banglapedia, 2006). In winter, temperature ranges from a minimum of 7°C – 13°C to a maximum of 24°C-31°C. The pre-monsoon months are hottest and in this period recorded maximum temperature is 36.66°C. In the western part, summer temperature rises up to 41°C or more. Average annual temperature ranges from 19°C-29°C. The amount of average annual rainfall varies from 1429 – 4338mm. About 80% of the rainfall occurs during the monsoon period, (UNFCCC, 2002). The single most significant sector of Bangladesh's economy is agriculture which productivity is highly dependent on climatic condition. In this sector about 84% of the total population of Bangladesh are directly or indirectly engaged (Shahid, 2010). In this country nearly 100 different kinds of crops are presently grown and rice is the principal one which covers about 79% of the total cropped area. Wheat, jute, potato, oilseeds, pulses, tobacco, cotton, sugarcane, fruits, and vegetables are other significant crops (Banglapedia, 2006). Seasonal characteristics and different variables of climate (temperature, rainfall, humidity, day-length etc.) affect the agricultural crops. Different disasters such as floods, droughts, soil and water salinity, cyclone and storm surges impede the crops production in this low deltaic country. This study presents the impact of climate change on agriculture.

2. METHODOLOGY

A proper methodology is always necessary for the successful completion of a research study. It is helpful regarding the organization of the experiences, observations, examinations and analysis of found data and information and their logical interpretation in a systematic process to achieve the ultimate goal and the objectives of the research. In this paper the data and information which is used collected from the secondary sources.

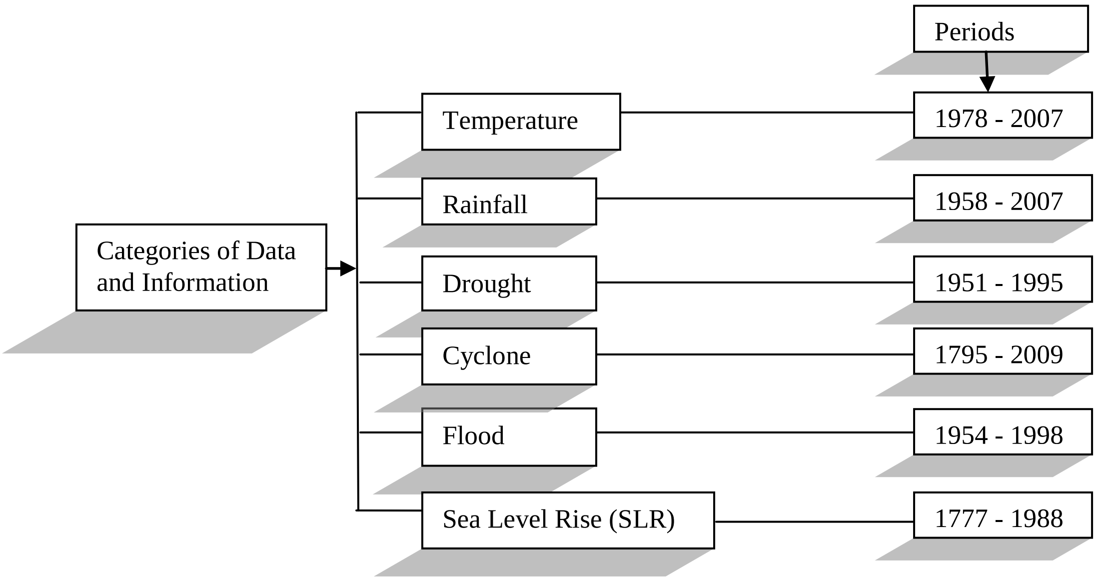

Fig 1: Categories of data and information in different periods

Furthermore, most of the information retrieved from the IPCC, 2007 fourth assessment report.

3. RESULT AND DISCUSSION

3.1 CLIMATE VARIABILITY AND CLIMATE CHANGE IN THE PAST AND PRESENT

In Bangladesh temperature is gradually increasing due to climate change induced by a rapid increase of anthropogenic activities. The mean annual temperature of Bangladesh has increased by 0.31°C over the past two decades (Islam, 2009). The temperature is generally increasing in the monsoon season from June to August and in this season average maximum and minimum temperatures show an increasing trend annually at a rate of 0.05°C and 0.03°C, respectively (Karim et al., 2008).

Table 1: Monthly average trends value of daily maximum, minimum and mean temperature (°C) during last 30 years period (1978-2007).

Month	Max	Min	Mean
Jan	-2.3	-1.9	-2.0
Feb	4.2	4.1	4.2
Mar	1.6	1.9	1.7
Apr	0.4	1.2	0.9
May	2.9	2.1	2.5
Jun	3.3	1.0	2.2
July	3.1	1.8	2.5
Aug	3.9	1.5	2.7
Sep	3.0	0.9	1.9
Oct	1.5	1.5	1.5
Nov	0.7	0.9	0.8
Dec	2.5	0.8	1.6
Average	2.1	1.3	1.7

(Data extracted from Islam, 2009)

In the table, it represents that during the last 30 years trends of daily maximum temperature exhibit higher value in summer season (June to September) with trends of more than 3°C.

The most important climatic factor is rainfall which determines the agricultural production of Bangladesh. The extreme high or low rainfall and the variability of precipitation are very important for the agriculture and thus for the economy of the country. Over Bangladesh for the period 1958–2007 the mean annual rainfall was 2488mm. The deviation of annual rainfall varies from +413 to -571 mm from the mean annual rainfall. The analysis of annual rainfall shows that the annual average rainfall is increasing at a rate of +5.525mm/year in Bangladesh. The increase in rainfall in western part of Bangladesh might be an effect of global climate change (Shamsuddin, 2010).

Sea level rise (SLR) is the variation in the average level of the surface of the sea. SLR along the coast of Bangladesh is one of the most critical variables that may increase the susceptibility to global climate change. A study was carried out on relative SLR of the Bay of Bengal based on 22 years (1977-1988) by the Meteorological Research Council of the South Asian Association for Regional Cooperation. They measured sea-level data and observed that sea levels at Hiron Point, Char Changa and Cox's Bazar have been rising by 4.0, 6.0 and 7.8 mm/year, respectively. The results expose that in the last century the rate of SLR along the coast of Bangladesh is much higher than the global rate of 1.0-2.0mm/year (Karim, 2008).

Storm surges with associated cyclones are an extremely serious natural hazard in the coastal area of Bangladesh. The intensity of the cyclones peaks during pre-monsoon (April-May) and post monsoon (October-November). Coastal areas and offshore islands are inundated by storm surges during a cyclone. Due to long and shallow continental shelf, the amplitude of cyclonic storm surge wave increase during the crossing of the coastal area of Bangladesh (UNFCCC, 2002). Cyclonic data from the years 1795 to 2009 show that the intensity and the frequency of cyclones are increasing. Frequency of weaker cyclones is decreasing but intensity of the weaker storm is rising (Rana et al., 2011). This disaster causes heavy loss of lives and property damage to the coastal area and highly damage the agriculture which lead to annual economic losses (Dube, 2009).

Drought is defined as a prolonged and continuous period of dry weather with abnormal insufficient rainfall. In Bangladesh, droughts are common in the northwestern districts and lead to crop loss. The following graph presents drought prone areas in cropping seasons.

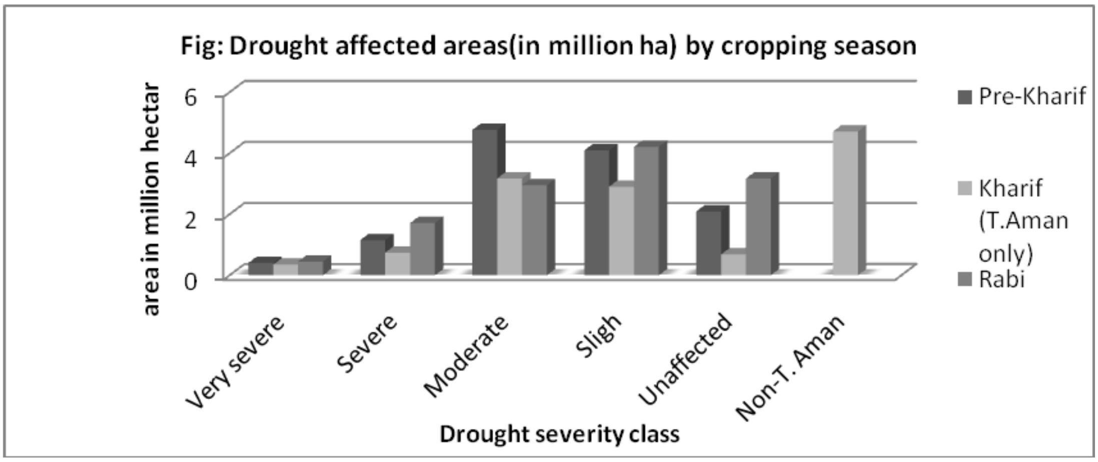

Figure 2: Shows the amount of land affected by drought in the cropping season (Data extracted from Ahmed, 2006)

Fig. 2 shows the areas (in million hector) affected by drought in the cropping seasons. Near about 0.5 million hector land is affected very severely. In the pre-kharif season above 4 million hector land is moderately affected by drought. In the kharif season, only T. Aman (near about 5 million hector) rice is affected by drought condition.

Pre-monsoon drought is called Rabi and Pre-Kharif drought as well as post-monsoon drought is known as kharif drought. Drought is very common in the districts of Rajshahi, Bogra, Pabna, Dinajpur, Rangpur and Kustia (Northwestern side of the country). Drought occurred 19 times within the 30 years (1960-1961). Very severe droughts hit the country in the year of 1951, 1961, 1975, 1979, 1981, 1982, 1984, 1989, and 1995. More than 47% of the country is vulnerable to drought, in this region 53% of the population lives (Ahmed, 2006).

Flood is the most common water related natural hazard in the deltaic floodplain country Bangladesh. Different complex series of factors are responsible for flood e.g. huge inflow of water from upstream catchment areas coinciding with heavy rainfall in the country, a low floodplain gradient, congested drainage channels, the major rivers converging inside the country, tide and storm surges in the coastal areas and polders that increase the intensity of floodwater outside protected areas. Severe floods occur at intervals of 7-10 years which damage crops property very highly and catastrophic floods occur at an interval of 20-50 years or more which almost totally destroy crops in adjoining floodplain (Ahmed, 2006).

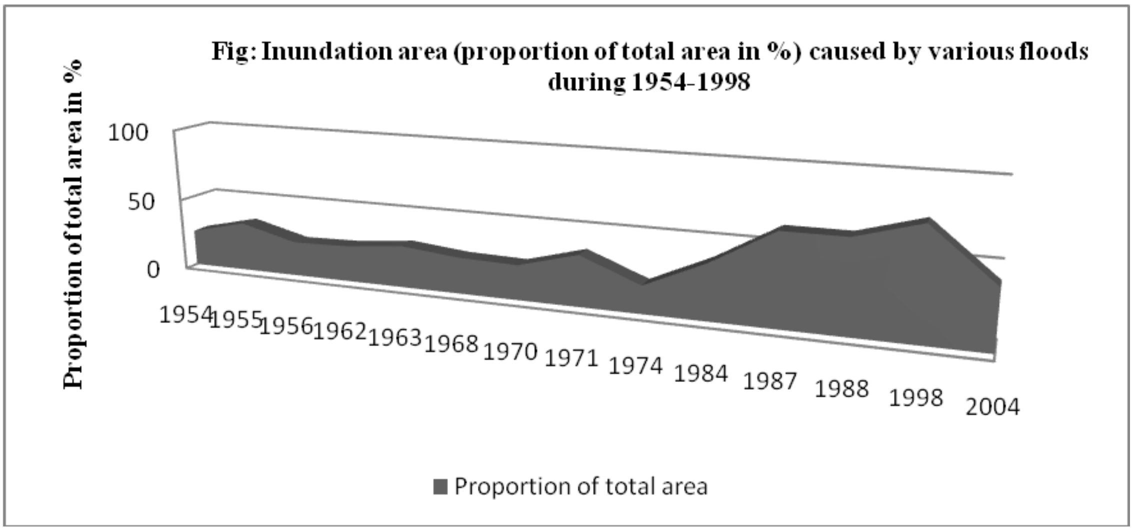

Figure 3: Total inundation area by various floods in the past (Data extracted from Ahmed, 2006)

From the figure (fig. 3) it can be observed that 20% of the land has been inundated every year due to flooding and from the year 1988 to 1998 above 60% land was inundated.

3.2 FUTURE CLIMATE PROJECTIONS

Being the low-lying, subsiding country with the mega delta of the three big rivers as Bangladesh is, a lot of research and simulations of its future climate have been done. The models used are mostly General Circulation Model (GCM) (e.g. Mizra et al. 2003), an Atmospheric Ocean General Circulation Model (AOGCM) (Unnikrishnan et al., 2006), or Hydrologic Models (e.g. Mizra et al., 2011; Karim and Mimura, 2008) to project changes in the hydrological cycle. The models are set up with different forcing and the results often show a larger change than the projected global change. The temperature e.g. is projected to increase more than the global mean temperature is (UNFCCC, 2007).

The change in precipitation patterns in Bangladesh will also change. The summer precipitation is likely to increase while most models show a decreased precipitation in the cooler, drier months, December-January-February (DJF) (IPCC, 2007), leading to more extreme droughts. The start and end dates of the monsoon might shift as well (Mizra, 2011), the monsoon arrives at about the same date every year with a standard deviation of seven days. In a future with a warmer atmosphere the variability of the arrival will possibly enhance. Yet another projected change in precipitation pattern is that of extreme rainfall, the events of which will increase along with an increase in interannual variability of daily precipitation in the summer monsoon.

3.3 IMPACTS OF CLIMATE CHANGE

A warmer atmosphere will result in a warmer ocean and the part of the ocean that warms first is the surface. Sea surface temperature (SST) or, more specifically, the thermal energy available in the upper ocean layer is one of the drivers of tropical storms like cyclones (Karim and Mimura, 2008). Many studies show that the frequency of tropical storms does not correlate with SST, while it does correlate with the strength of the storm. With a warming of the SST, the cyclones will intensify (Emanuel, 2005).

When the ocean warms its volume gets larger through thermal expansion, giving a sea level rise (SLR). Bangladesh is a low-lying country and the whole Bengal basing is slowly subsiding, generally due to tectonic movements (Alam, 1996), these two factors together give that a SLR of half a meter would submerge 20% of the county’s habitable land (Houghton, 2009).SLR will have directly affected more than one million people living in the Ganges-Brahmaputra-Meghna delta and indirectly far more by 2050 (IPCC, 2007).

Along with the storms come storm surges, which have a great impact on the coastal areas. As the storm surge approaches the shore it amplifies due to the bottom topography and cause damages as it hits the coast (Unnikrishnan et al., 2006). As the SST and the intensity of the tropical storm increase, the storm surges are likewise projected to become higher, leading to severe damage on the coastal area (Karim and Mimura, 2008).

A rise in the SST by 2C or 4°C may increase the surge height by 21% and 49% respectively. With a SLR the amplification of the storm surge is less, because the bottom friction on the wave changes.

The extreme rainfall events will increase as mentioned above during the monsoon season. The heavy rain will cause more and deeper inundations in the low-lying river basins (Mizra, 2011).

Climate change will finally also increase the water stress in Bangladesh. As the dry season gets drier with global warming, the groundwater level drops, the SLR then results in salt intrusion in the groundwater further into the country. The glaciers of the Himalayas will melt and the river runoff will initially increase in winter or spring and decrease in summer or autumn. Finally as the glaciers disappear the river runoff will decrease in all seasons (IPCC, 2007).

3.4 CLIMATE CHANGE AND AGRICULTURE

The sectors most sensitive to climate change are the water and the agricultural sector. The agricultural production is highly susceptible to changes in temperature, precipitation and length and intensity of dry periods, which puts the food security at risk. As mentioned before, Bangladesh is a country dependent on agriculture. This sector is by far the main consumer of water (UNFCCC, 2002). The share of available freshwater used for agricultural production is likely to increase in the future in order to keep up with the growing demand for food. The problems of today are reduced land productivity as a result of land degradation and a decline in soil fertility, as well as natural disasters which are responsible for a loss of cultivable land of 1% every year (UNFCCC, 2002).

Climate change is likely to aggravate the problems that agricultural production is already facing today. In the whole region of South Asia, crop yield is likely to decrease by 30%. However, there are significant regional differences in the response of wheat, maize and rice to a changing climate. Rice and wheat production in Bangladesh is likely to decline by 8% and 32%, respectively, until the year 2050 (IPCC, 2007). A different GCM-coupled crop modeling exercise (Ahmed, 2006) estimates a drop in production by almost 30% for rice and 60% for wheat for a moderate climate change scenario. Although the increased concentration of CO_2 has a fertilizing effect in the beginning, with temperatures rising further it is very likely to be outweighed by stress due to higher temperatures and changes in the precipitation regime (Ahmed, 2006; IPCC, 2007).

There will be a temperature rise throughout the whole year. It is estimated that for every 1°C increase in the minimum temperature during the growing season, the yield of rice declines by 10% (IPCC, 2007). Temperature rise is also going to affect the length of the growing season: for rice, it will shorten from 145 days now to 140 days in 2025 and 130 days in 2100 (Shahid, 2011).

Precipitation changes affect rice growth, as it is dependent on the timely arrival of the monsoon, on the amount of precipitation and little or no flooding (Mirza, 2002).Flooding are the climatic events that mostly affect rice growth, as it cannot sustain water levels exceeding 90 cm.

In areas with an increase in rainfall, greater surface runoff and thus flooding will result. The flood of 1998 for example caused a loss of 50% in agricultural production, leading to a decline in national GDP of 3% (Mirza, 2002). Moreover floods lead to erosion and sedimentation, causing reduced productivity or even loss of arable land.

On the other hand are substantial reductions in yields also expected in areas where precipitation is projected to decrease? That is because 60% of the cropped area is rain-fed. In combination with higher temperatures that increase evapotranspiration, the water demand by plants is going to rise. In the dry northwest and northeast of Bangladesh, potential evaporation exceeds precipitation from November until April, leading up to a soil moisture deficit and increasing the need for irrigation (Shahid, 2011). As climate change is going to increase the risk of drought, the demand for irrigation will be growing. For an increase in temperature of 1°C, irrigation demand is going to rise by 10% in the arid and semi-arid regions of Bangladesh (UNFCCC, 2002). In these regions, the pattern of irrigation water demand does not principally depend on precipitation; more important are soil characteristics and evapotranspiration (Shahid, 2011). Reduced rainfall in combination with more irrigation will increase groundwater withdrawal and lead toa falling groundwater level. In coastal areas this will cause a penetration of sea water and salinization of soils, which negatively affects soil fertility. However, Shahid (2010) argues that the growing demand for irrigation may partly be outweighed by the shortened growing period, leading to a reduction in irrigation days.

Climate change does not only alter crop production, but also the areas of production: a northward shift of the agricultural zones is likely to occur (IPCC, 2007).

Apart from changes in the temperature and precipitation regimes, one of the most serious threats to the low-lying country is sea level rise, because it would destroy the homes of 15 million people and good agricultural land by 2050 (Houghton, 2009). Most of the inundated agricultural land would be lost because crops cannot handle the excessive moisture.

The increasing intensity of tropical cyclones and storm surges that originate in the Bay of Bengal and the ensuing flooding cause inland penetration of seawater. Already today it is estimated that salt water extends over 150 km inland (Houghton 2009). The increased soil salinity is unsuitable for the most crops that now are grown. Without the development of salt tolerant crop varieties, much of the agricultural land will be lost.

3.5 ADAPTATION STRATEGIES FOR AGRICULTURE

Although the agricultural sector is highly vulnerable to changes in climate, very little has so far been done to work out adaptation potentials for the country. Among the physical adaptations to extreme events are measures related to the handling of drought, floods and salinity (UNFCCC 2002). To counteract the increasing likelihood of drought, farmers should switch from a high-risk rain-fed agriculture to an agriculture based on extended irrigation. Higher irrigation efficiency through new technologies is required, besides must the access to these technologies be secured. Crop diversification that includes a switch to crops with low water demands plays a key role in order to maintain food security and land productivity, the same goes for fertilization. Damages from floods could be minimized by selecting appropriate crops as well as adjusting the timing of planting. For example shall rice planting during the monsoon season, which is going to experience higher precipitation, be abandoned, and be carried out during winter season instead. Another adaptation strategy practicable in water logged areas is to create floating gardens with the help of free-floating aquatic plants, where then vegetables can be grown. This is an indigenous practice, which demonstrates that it is important to take the knowledge of traditional farmers into consideration when working out adaptation strategies. The increased threat to agricultural production from soil salinity could be faced by salt resistant crops and changes in irrigation techniques.

These possible and necessary physical adaptations are for the most part still theoretical. Their implementation depends on the institutional adaptations made. These comprise, among others, investments in research for new crop varieties and irrigation technologies. In order to reduce the pressure on groundwater resources in dry areas, it is essential to develop surface water resources for irrigation (Shahid 2011). Infrastructure investments in education and information programs are required to raise awareness and thus give farmers a chance to respond to the threats that are to come. Early warning systems also play a crucial role. Among the preventive measures are embankments to protect land from inundation. Yet, these measures often require large-scale financial and institutional support by the government.

Another conceivable adaptation measure, especially in areas prone to inundation due to sea level rise, is a relocation of the agricultural production. However, the agricultural land that would be lost could hardly be replaced elsewhere because there are almost no other regions suitable for agriculture. The majority of arable land is already under cultivation. The decrease in arable land combined with a growing demand for food due to population growth calls for an intensified agricultural production that increases yields per unit of water, time, land and energy.

3.6 ADAPTATION AND MITIGATION

In 2008 the Bangladeshi scientists and politicians developed Bangladesh Climate Change Strategy and Action Plan (BCCSAP), which is a follow up, more strategic and longer term adaptation plan than the former National Adaptation Program of Action (NAPA), supported by the UN (Huq, 2011). Bangladesh's adaptation focuses are several.

Comprehensive disaster management, Bangladesh has during the past decades developed a program of warning the population about approaching storms and disasters. This has given people time to travel to cyclone-shelters, which has saved many lives in the last cyclones that hit the country. The BCCSAP will take this a step further, aiming at enhancing the preparedness and improving the forecasts so that they will be more accurate both the long- and the short term.

Infrastructure development, infrastructure repeatedly gets damaged in storms and floods. In the BCCSAP, the aim is to repair and strengthen existing infrastructure, plan, design and build needed infrastructure and start strategic planning of future infrastructure needs.

Research and knowledge management, this policy aims towards an increased understanding in the impacts of climate change. The BCCSAP also aims at connecting climate change, poverty and health as well as climate change, poverty and vulnerability, to identify when the poor population is most vulnerable to climate change.

Mitigation and low-carbon development, although Bangladesh is focusing mostly on the adaptation part, the BCCSAP also aim at decreasing the greenhouse gas emissions, which are already low in the country. This is done by afforestation and exploring new energy technologies (ministry of environment and forests, 2008).

4. CONCLUSION

Bangladesh is one of the country's most susceptible to climate change in the world. A large part of its area is low-lying and is therefore vulnerable to inundation due to sea level rise and high amplitude storm surges. The agricultural sector makes up for the most important part of the economy and employs the vast majority of the population. The largest part is crop agriculture, above all rice and wheat. The crops are dependent on climatic factors, such as temperature, precipitation and extreme events. With climate change, temperatures will rise and precipitation regime is going to change. Drier regions are going to become drier, whereas extreme rainfalls are going to increase, resulting in floods as the rivers Ganges, Brahmaputra and Meghna break their banks. Flooding reduce agricultural land, which cannot be replaced elsewhere because most of the land suitable for agriculture is already in use. With increased drought and water stress on crops, irrigation demand rises, putting pressure on groundwater resources. Consequently, the groundwater level drops every year.

There are a number of possible adaptation measures to face the effects of climate change on agricultural production, but not many are implemented yet. Among the physical adaptation measures are those facing the increase in droughts, floods and soil salinity – such as the diversification of crops, the extension of irrigation and water saving techniques, like waste water management and desalination of sea water. The success of these measures hinges on government policies and institutional arrangement as well as investments in new techniques. Regional cooperation also plays a crucial role.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DEVELOPMENT OF CROP YIELD PREDICTION MODEL FOR BANGLADESH: A WAY TO FOOD SECURITY IN CONTEXT OF CLIMATE CHANGE

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ABSTRACT

Global food security is at greater risk due to abrupt climate change. Developing countries like Bangladesh face various unpredictable food scarcities in past few decades. Development of a crop yield prediction model like EPIC may ensure prediction of crops in future years considering climate change effect. This study represents the development of EPIC Model for Transplanted Aman. Furthermore, it ensures the crop yield prediction model with specified error range in context of Dhaka district. As a comprehensive mathematical cropping system point model, EPIC simulate the crop yield considering climatic data, soil data, management data and operation data for any specified crop. Hence it represents the climate change effect on crop yield. A prior simulation of sensitivity analysis shows the most sensitive parameter being 'Nitrogen in rainfall' and 'Nitrogen content of soil'. The development evolves the steps to feed the model with eight years (1993-2000) of climatic data of Dhaka district and by simulating yield over and over with different value of a fixed parameter, while all other parameters are set in standard values. By comparing the simulated yield with actual yield and hence adjusting the value of fixed parameter within standard range for Bangladesh, calibrated value of sensitive parameter is found. After calibration, validation is done by feeding the model with calibrated value and check the range of validity comparing with actual value. By expanding this study for the whole country, the total crop yield can be predicted considering climate change effect on crop production which may lead to the more secure crop status of this country and in future a hunger free world.

Key word: *Bangladesh, Agriculture, Climate change, Crop yield prediction model, EPIC*

1. INTRODUCTION

Climate variability, its impacts and vulnerability are worldwide growing concern. The most direct economical influence of global climate change is due to its direct impact on agriculture and crop yields (Cruz et.al. 2007). The Fourth Assessment Report of the IPCC considers agriculture as one of the most vulnerable systems to be affected by climate change in the south Asian region. Bangladesh being one of the countries of the region and a flat deltaic floodplain, it is very likely that the climate change vulnerability would increase. In developing countries, 11 percent of arable land could be affected by climate change, including a reduction of cereal production in up to 65 countries, about 16 percent of agricultural GDP (FAO Report of 31st Session. 2005). Bangladesh has been identified as one of the countries with the highest level of vulnerability to climate change (McCarthy et al., 2001). A great majority of the people of Bangladesh is dependent on agriculture for their livelihood and crop agriculture in this region is highly susceptible to variation of climate system. The climate of Bangladesh is changing and it is becoming more unpredictable every year. As a consequence of future climate change agriculture of developing countries like Bangladesh is likely to be effected by many environmental hazards including frequent floods, droughts, cyclones, and storm surges that damage life, property and agricultural production; this would also lead to risk of hunger and water resource scarcity with enhanced climate variability. Agriculture is the single most important sector of Bangladesh's economy. 80% of the population is engaged in agriculture (66% of the labor force). Fifty-seven percent of the labor force is engaged in the crop sector which represents about 78% of the value added in the agricultural sector. The share of agriculture to the total Gross Domestic Products (GDP) is about 21.77 % and the overall GDP growth rate is around 4.49%, which is not high enough to significantly impact the country's poverty scenario. The share of this sector is 4.9% to the total export earnings. While the share of crop sector in GDP is 12.19% (2005-06 Fiscal Year). The agriculture sector grew at 4.49% per annum and contains 63 per cent of the employment. Rice and jute are the principal crops; sugarcane, potato, pulse, tea and tobacco are also important crops. Within the crop sector (rice, wheat, pulses and jute), rice dominates, with an average 71% share of the gross output value of all crops. Considering the circumstance stated above, the necessity of investigating the impacts of climatic change on agriculture was realized. In this study new methodology for local estimation of crop productivity is proposed. This methodology uses Erosion Productivity Impact Calculator (EPIC) model. With the climate change data provided by the Bangladesh Meteorological Department (BMD) and Intergovernmental Panel on Climate Change (IPCC) development of EPIC is done for scenarios of 1993-2006 for Transplant Aman.

2. SCOPE OF THE STUDY

2.1 *The Selection of Crop*

Rice is the staple food of about 135 million people of Bangladesh. Each year milled rice consumption of Bangladesh people is about 168 kg/capita/year which provide the 76% of the total calorie intake and 65% of total protein intake. (FAO online database; World Development Report, 2002. World Bank). In terms of total rice production Bangladesh occupies 4th position in the World (USDA. 2001). The study includes one major rice types of Bangladesh - Transplanted Aman

2.2 Selection of Study Area

The study area includes five upazilla of Dhaka district namely, Savar, Dohar, Thamrai, Nawabganj and Keraniganj.

2.3 Selection of Soil Type

There are 13 types of soil series in Dhaka district. Total arable land area of Savar Thana is 24007 ha, in which Bramhaputra Alluvium soil covers 2231 ha (9.29%), Tejgaon soil covers 115 ha (0.48%). Total arable land area of Dhamrai Thana is 26720 ha, in which Bramhaputra Alluvium soil covers 1006 ha (3.76%), Savar Bazar soil covers 5561 ha (20%), Sonatala soil covers 747 ha (2.79%) and Silmandi soil covers 1336 ha (5%). Total arable land area of Nawabganj Thana is 19585 ha, in which Bramhaputra Alluvium soil covers 215 ha (1.1%). Total arable land area of Keraniganj Thana is 13421 ha, in which Bramhaputra Alluvium soil covers 365 ha (2.71%), Savar Bazar soil covers 387 ha (2.88%), Sonatala soil covers 463 ha (3.45%) and Silmandi soil covers 1819 ha (13%). So the total area covered by five soil series is 12.25% of the total arable land area.

2.4 Selection of the range of Climatic Data

For sensitivity analysis, sixteen years (1993-2008) climatic data from Dhaka Weather Station was considered.

3. METHODOLOGY

Epic model is a point based model, which means it takes the data as input of a particular location and gives the output. It takes various climatic data, management data, crop operation data, and soil data as input parameters for a specified period of time. It gives output as form of biomass (kg/ha), yield (kg/ha), average stress days for biomass and root growth in form of water, nitrogen, phosphorus, temperature and air. Climatic data relates to geography, climatic condition of the study area and management data relates to the crop management and the general practices in cultivation of study crops. Soil data relates to the specific characteristics of soil and condition of the water table in the study area.

In context of Bangladesh, the data set needed for sensitivity analysis can be divided in two parts-
First, the data which are available like: daily air temperature maximum, minimum and average, daily rainfall, wind velocity, humidity, solar radiation etc. of any particular location which are easily available in meteorological stations. Also the soil data are available for the particular series of soil. Management data comes from fertilizer management, weed management, irrigation practices of the farmers of the study area.
Second, there are several data which are not available in our country easily. These data needs more labor, time and money which is not feasible for prediction of yield of crop a small land area.

Considering these two types data and their availability, sensitivity analysis is done for a particular crop to assess how the crop responds in terms of yield for different sets of data. As previous yield data is available, so comparison can be easily made between different input parameters using Microsoft Excel spread sheet. Also using SPSS 17 software evaluation of Person’s coefficient is done, which help correlate the parameter to the yield and specifies how the model respond to different input parameters. From this study, a conclusion can be drawn that Nitrogen in rainfall and Nitrogen content of soil have more effect on predicting the yield by this model.

Furthermore, calibration of EPIC model is done using standard values for all other parameters, variation of a specific parameter is done and hence adjustment towards the actual output value. T.Aman is calibrated in respect to condition of Bangladesh, so that the people of Bangladesh can use the EPIC model with less difficulty. EPIC model can be used without calibrated in United States of America, as this model considers the condition of that country. There are some parameters that can be considered as fixed value, as Bangladesh is a relatively small country, so variables like: N concentration in rainfall varies little over the country and this little variation has less effect on crop yield.

3.1 Calibration of Epic model for T.Aman

3.1.1 Data collection, processing and analysis

Data collection was the most initial step of the study. It was a huge work. Different Government and Non-Government organization personal, library, website were used to collect relevant data. Weather data was collected from Meteorological Department Dhaka, Soil data from SRDI (Soil Resource Development Institute), Crop data from BARC (Bangladesh Agricultural Research Council).

Data required for feeding the model like monthly total precipitation (fig 1), monthly average precipitation (fig 2), standard deviation (fig 3) and skewness coefficient of rainfall (fig 4), monthly average maximum air temperature (fig 5), monthly average minimum air temperature (fig 6), standard deviation of maximum temperature (fig 7), standard deviation of minimum temperature (fig 8) are done with the help of worksheet.

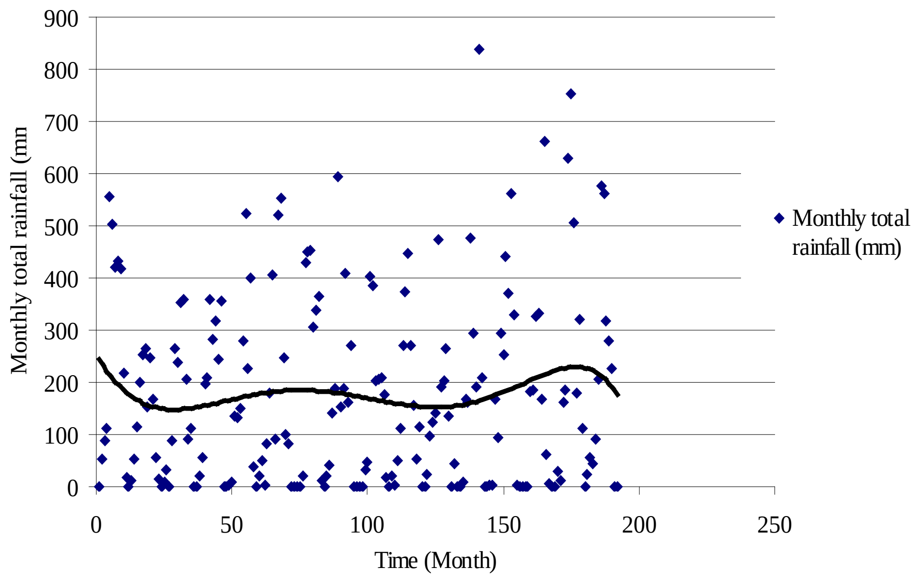

Figure 1: Variation of monthly total precipitation with time

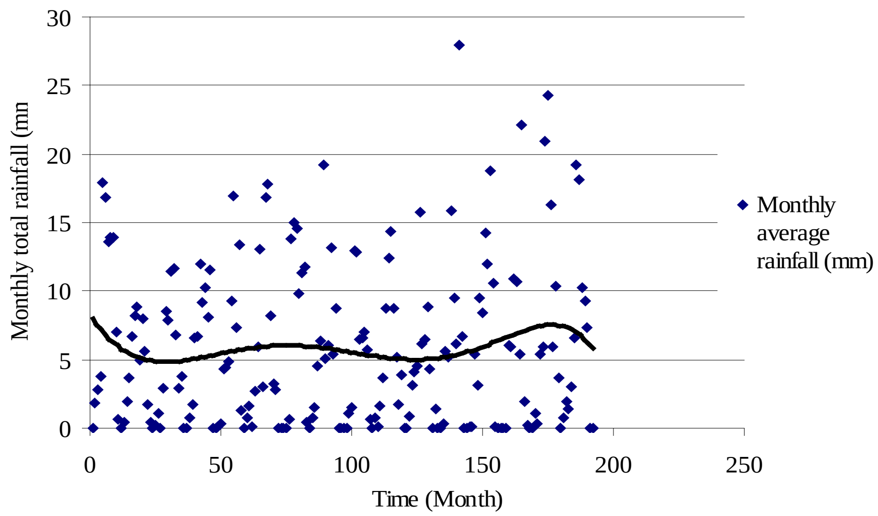

Figure 2: Variation of monthly average precipitation with time

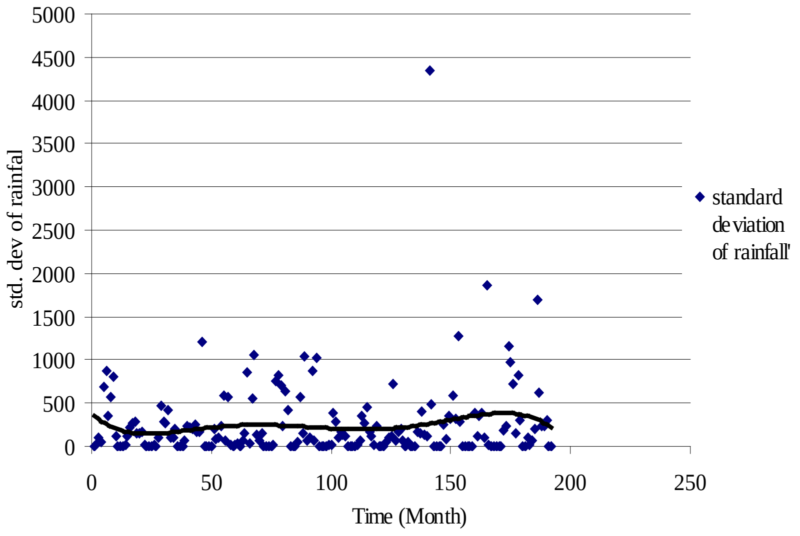

Figure 3: Variation of standard deviation of precipitation with time

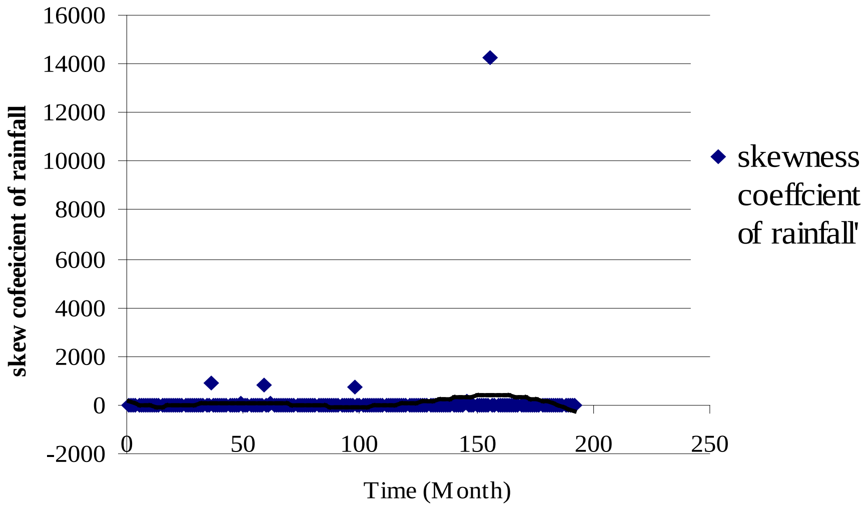

Figure 4: Variation of skew ness coefficient of precipitation with time

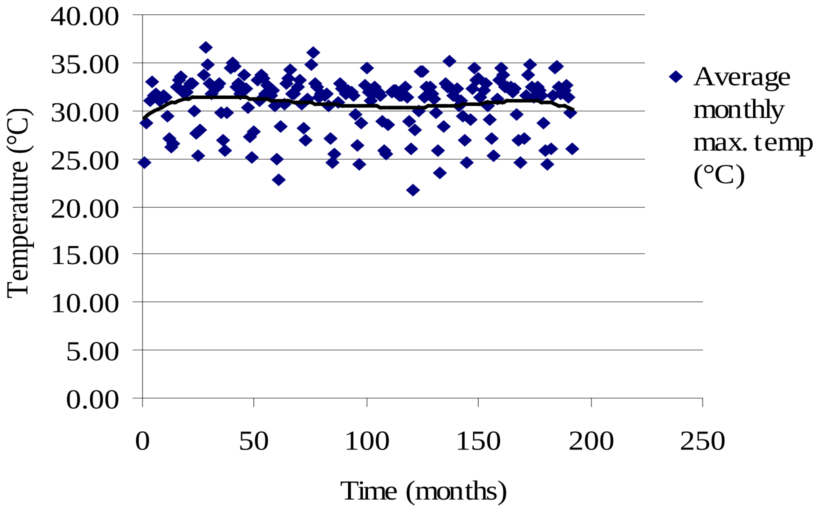

Figure 5: Variation of monthly average maximum air temperature with time

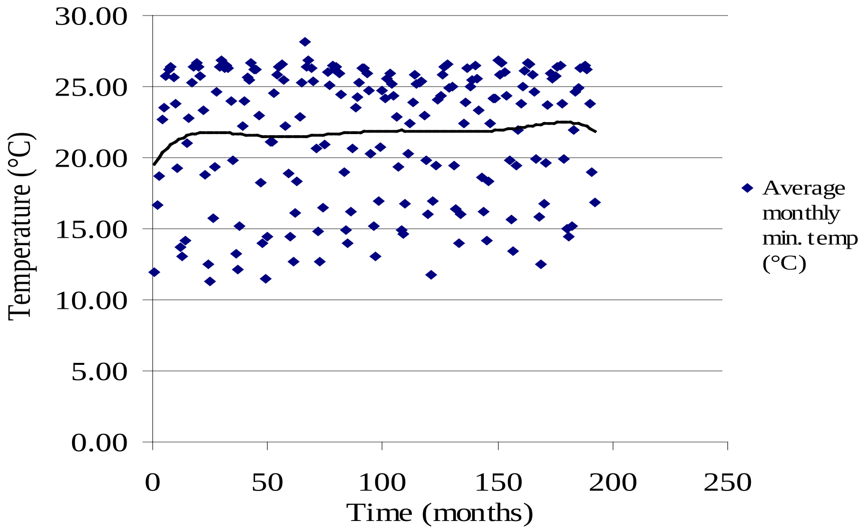

Figure 6: Variation of monthly average minimum air temperature with time

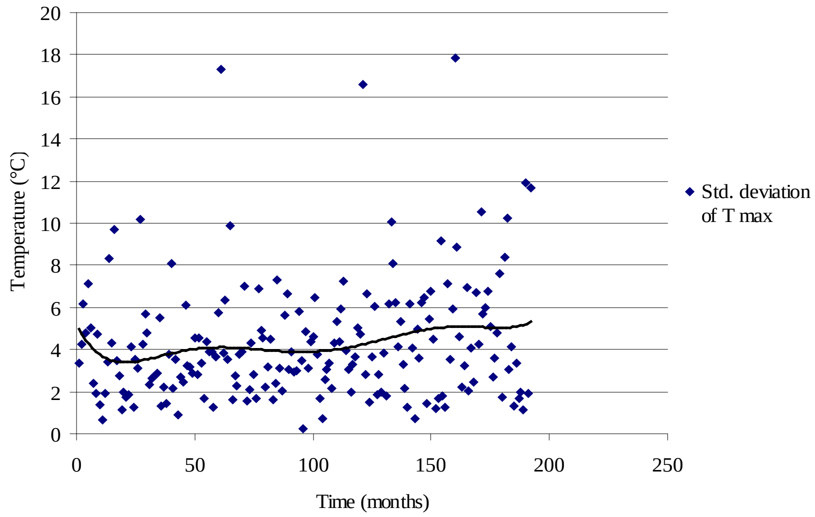

Figure 7: Variation of standard deviation of maximum temperature with time

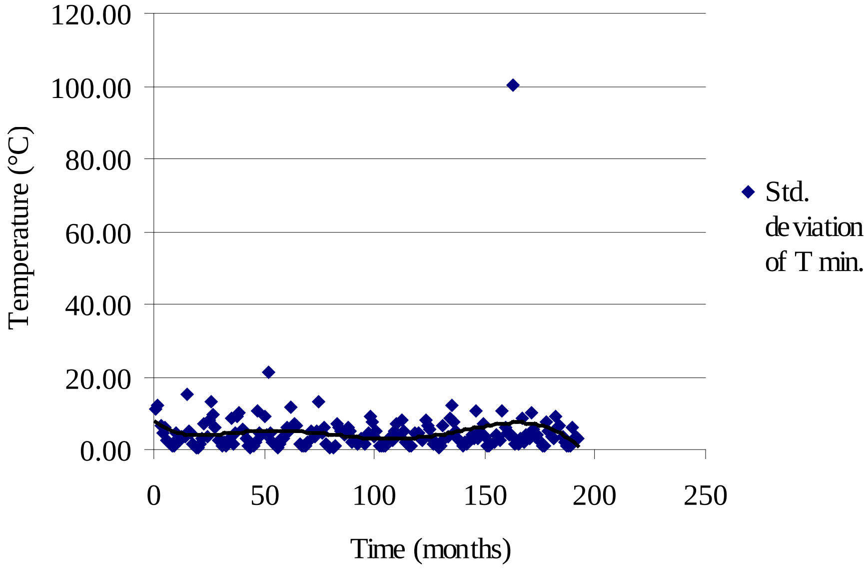

Figure 8: Variation of standard deviation of minimum temperature with time

3.1.2 Data Set for Calibration

The data set used for calibration of T.Aman is stated in table 1 and table 2. Here EPIC model is calibrated using climatic data from 1993 to 2000 and making simultaneous adjustment to shift the model yield close to the actual yield of Dhaka district.

Table 1: Calibration DataSet of T.Aman for average conc. of N in rainfall

Year	**Prod** (ton/ha)	**ALLUVIUM**					**SONATALA**					**SAVAR BAZAR**					**CALIB**	
		Conc. of N in Rainfall (PPM)					Conc. of N in Rainfall (PPM)					Conc. of N in Rainfall (PPM)					Conc.	
		0.5	0.8	1	1.2	1.5	0.5	0.8	1	1.2	1.5	0.5	0.8	1	1.2	1.5	0.5	0.8
1993	1.943	2.8	2.9	2.9	2.9	3.0	2.8	2.9	3.0	3.0	3.1	2.8	2.9	2.9	2.9	3.0	2.8	2.90
1994	1.901	2.3	2.3	2.4	2.4	2.5	2.3	2.4	2.4	2.5	2.5	2.2	2.3	2.3	2.4	2.5	2.3	2.33
1995	1.851	2.7	2.7	2.7	2.8	2.8	2.8	2.8	2.8	2.9	2.9	2.7	2.7	2.8	2.8	2.8	2.7	2.73
1996	1.741	2.4	2.7	2.7	2.7	2.8	2.7	2.9	2.9	2.9	3.0	2.8	2.9	2.9	2.9	2.9	2.6	2.83
1997	2.055	3.5	3.6	3.7	3.7	3.8	3.5	3.7	3.7	3.8	3.9	3.6	3.7	3.7	3.8	3.8	3.5	3.67
1998	1.904	3.5	3.6	3.6	3.7	3.8	3.5	3.6	3.7	3.7	3.8	3.5	3.6	3.6	3.6	3.7	3.5	3.60
1999	1.539	1.2	1.3	1.3	1.3	1.3	1.5	1.5	1.5	1.6	1.6	1.3	1.3	1.4	1.4	1.4	1.3	1.37
2000	2.112	3.7	3.8	3.8	3.8	3.9	3.7	3.8	3.8	3.8	3.9	3.6	3.7	3.7	3.7	3.8	3.7	3.77

Table 2: Calibration DataSet of T.Aman for organic N content

Year	**Prod** (ton/ha)	**ALLUVIUM**					**SONATALA**					**SAVAR BAZAR**					**C**
		ORGANIC N CONTENT (PPM)					ORGANIC N CONTENT (PPM)					ORGANIC N CONTENT (PPM)					OR
		30	150	200	250	300	30	150	200	250	300	30	150	200	250	300	0.5
1993	1.943	2.80	3.00	3.10	3.20	3.30	2.80	3.20	3.30	3.40	3.50	2.80	3.00	3.10	3.20	3.30	2.80
1994	1.901	2.30	2.50	2.60	2.70	2.80	2.30	2.60	2.70	2.80	2.90	2.20	2.50	2.60	2.70	2.80	2.27
1995	1.851	2.70	2.90	3.00	3.00	3.10	2.80	3.00	3.10	3.20	3.30	2.70	2.90	2.90	3.00	3.10	2.73
1996	1.741	2.40	2.80	2.90	3.10	3.20	2.70	3.10	3.20	3.30	3.40	2.80	3.00	3.10	3.10	3.20	2.63
1997	2.055	3.50	3.90	4.00	4.20	4.30	3.50	3.90	4.20	4.30	4.40	3.60	3.90	4.00	4.10	4.20	3.53
1998	1.904	3.50	3.80	3.90	4.00	4.10	3.50	3.90	4.00	4.10	4.30	3.50	3.70	3.80	3.90	4.00	3.50
1999	1.539	1.20	1.40	1.40	1.50	1.50	1.50	1.60	1.70	1.80	1.80	1.30	1.50	1.50	1.60	1.60	1.33
2000	2.112	3.70	3.90	4.00	4.10	4.20	3.70	3.90	4.00	4.10	4.20	3.60	3.80	3.90	4.00	4.00	3.67

3.2 Validation of Epic model for T.Aman

Validation means, the act of testing for compliance with a standard, certify conformance to a standard or to prove something to be sound or logical. Validation is done in EPIC model after calibration to check validity of the calibrated value for six years (2000-06). Using the calibrated value of specific parameters and considering standard value for other parameters, the yield is determined by running model and then comparison between model yield and actual yield is done. By Microsoft Excel (table 3) the correlation between these two values has been found.

Table 3: Validation Data Set of T.Aman

Year	**Production (ton/ha)**	**Yield in different types of Soil**			**Validated yield (ton/ha)**
		alluvium	**savar bazar**	**saonatala**	

2001	2.841	2.1	2	2.1	2.067
2002	2.779	1.8	1.7	1.9	1.800
2003	3.076	2.2	2.1	2.4	2.233
2004	3.400	1	1	1.1	1.033
2005	2.862	2.5	2.5	2.5	2.500
2006	2.650	2.7	2.6	2.7	2.667

4. RESULT AND DISCUSSION

EPIC model is calibrated for Transplanted Aman using climatic data of the year 1993-2000. Two parameters were calibrated. The management parameter *"Nitrogen in rainfall (ppm)"* has been calibrated to a value of *0.5 ppm*, which can be seen in fig 9. The soil parameter *"Organic N content"* has been calibrated to a value of *30 ppm,* which can be seen in figure 10.

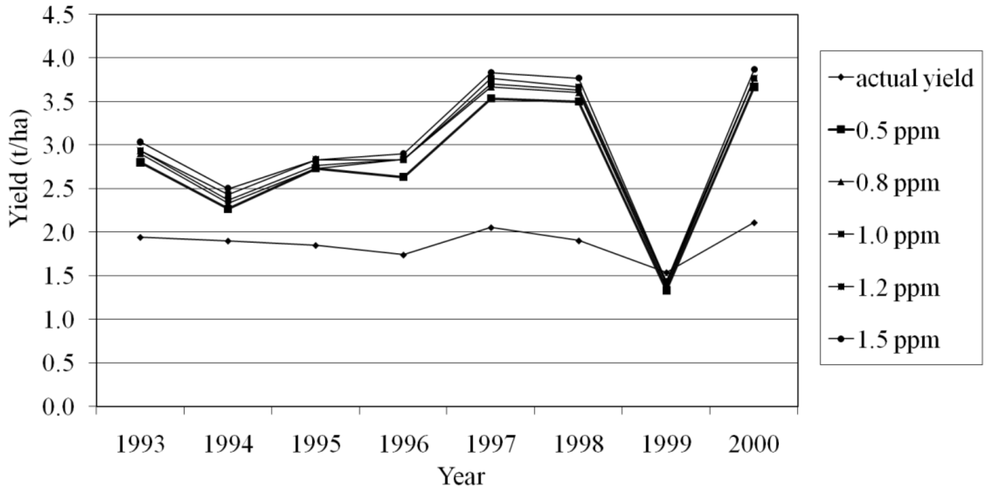

Figure 9: Calibration of T.Aman for management parameter -Nitrogen conc. in rainfall.

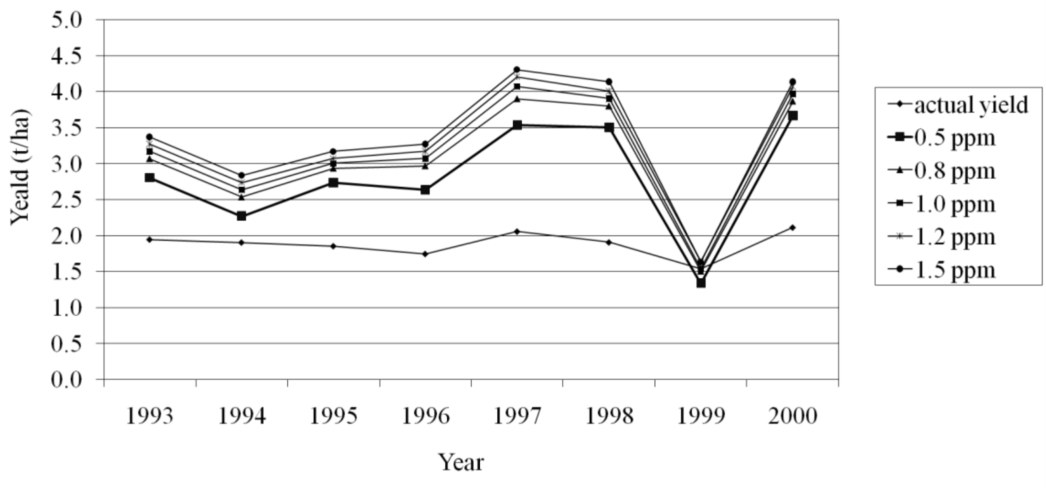

Figure 10: Calibration of T.Aman for soil parameter-Organic N content

Throughout the comparison between the actual and model simulated yield (fig 11) it can be seen that, the actual value varies almost about ±35 % than the simulated value. This may be due to the cause that, the model is simulated assuming the ideal condition while inputting the management parameters.

Also EPIC is not a variety sensitive model, so yield variability for genetically different variety is not taken into account during simulation of the prediction.

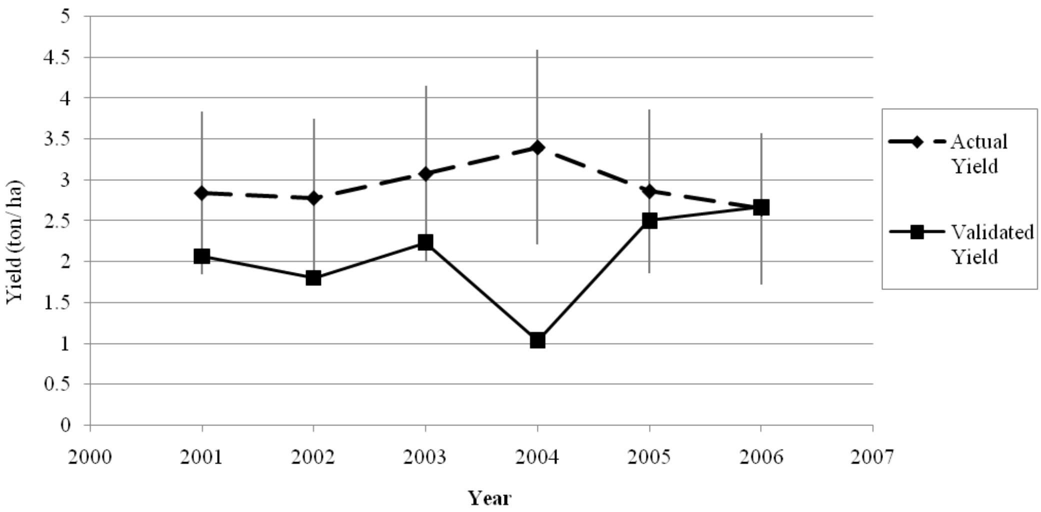

Figure 11: Validation of T.Aman for calibrated parameters from 2001- 2006

5. CONCLUSIONS

Calibration of T.Aman involves running the model with a set of climatic data ranging from 1993-2000, with a different value of the most sensitive parameter and hence correlate the model simulated yield with actual yield. By running the Model again and again with a series of parameter values, the simulated yield was adjusted to the actual yield. In this study Organic Nitrogen Content and Nitrogen Concentration in Rainfall have been found as the most sensitive parameters, so the attempt was made to calibrate these two values. The calibrated value of Organic Nitrogen Content is 0.5 ppm and for Nitrogen Concentration in Rainfall is 30 ppm.

Validation check of the calibrated value is done with the same set of data, but climatic data and actual yield was taken from 2001-2006. Validation was done with the two calibrated values. During the comparison with the actual yield to the model simulated yield, it can be seen from figure 11 that five model simulated yield deviate about 35% from the actual yield, but one yield is pretty far from the actual yield, which is 2004 years yield. The actual yield of 2004 also shows variation from the yield of year 2001, 2002, 2003, 2005 and 2006.

This study is of practical use to predict future yield of T.Aman cultivated on five specific soil type of Dhaka district. The parameters calibrated are difficult to measure for each time the model is run, so for future run these values can be set fixed for Dhaka district and simulate the future crop yield with +35% deviations.

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IMPACTS OF CLIMATE CHANGE SCENARIOS ON CROP SIMULATION (BORO CULTIVATION) IN BANGLADESH USING DSSAT

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ABSTRACT

Climate change is a potential threat towards attaining the objective of increasing the rice yield in Bangladesh. This assessment was to investigate the impact of possible climate change on yield of two varieties of Boro rice (BR3 and BR 14) cultivation using software DSSATv4 since it is sensitive to temperature and precipitation patterns. Among the existing 37 meteorological stations, 11 representative stations have been selected for spatial comparison. Crop yield under normal and changed climate scenarios, climatic parameter predicted by PRECIS model for the years 2008, 2030, 2050 and 2070 have been used for comparison. Analyses show that-compared to 2008, predicted average reductions of yields of BR 3 variety for the selected stations are about 22% in 2030, 28% in 2050 and 55% in 2070 and those for BR 14 variety are about 12% in 2030, 26% in 2050 and 63% in 2070. Results also appear that without any water and nitrogen stress i.e. non-stress condition gives higher yield.

Keywords: *Bangladesh, Climate Change, DSSAT model, PRECIS model, BORO Cultivation , BORO yield*

1. INTRODUCTION

The global climate change is one of the most significant environmental issues of the present world. The multidimensional effects of global climate change are evident now. Among the many effects, scientists are predicting that the tropical and subtropical countries will be more vulnerable to the potential impact of global warming through the effects on crops, soils, insects, weeds, and diseases (Bangladesh State of the Environment, 2001). Bangladesh is in the subtropical region. Therefore, the agriculture of this country will be affected. The effects of climate change are already evident in the agro-ecosystem of the country (IPCC, 2007).

Rice is the staple food for over 140 million Bangladeshis who obtain more than 70% of their total calorie from rice. The per capita rice consumption in Bangladesh is higher than that in any other country where rice is the staple (Bangladesh Rice Foundation, 2002).World rice production must increase by 1% annually to meet the growing demand for food that will result from population growth and economic development. During the past decades, the temperature of Bangladesh is significantly increased. Variability has been observed in the precipitation pattern as well. Production of rice in Bangladesh is highly susceptible to variations in these climatic parameters. In this chapter, an attempt has been made to analyze the adaptation potential of rice of the country in a warmer world.

A number of simulation studies have been carried out to assess impacts of climate change and variability on rice productivity in Bangladesh using the CERES-Rice model (e.g., Mahmood et al., 2003; Mahmood, 1998; Karim et al., 1996). These studies mainly focused on the effects of higher air temperature and atmospheric CO_2 concentration on rice yield. It may be noted that weather data requirement for CERES-Rice model include daily maximum and minimum air temperatures, daily precipitation and daily solar radiation, all of which could affect rice yield significantly. Therefore, in this study, future climate scenarios, including daily maximum and minimum temperatures, precipitation and solar radiation, for the selected locations of Bangladesh have been generated and used for predicting yield of Boro rice. The main objectives of this assessment was to investigate whether there is any impact of possible climate change on yield of two varieties of Boro rice –BR 3 and BR 14.

2. METHODOLOGY

A number of types of climate prediction models are being extensively used to prepare future climate change scenarios. In this study, prediction made by a regional climate model named 'Providing Regional Climates for Impacts Studies' (PRECIS) has been used. Prediction data of four selected years were used for the analyses: for the year 2008, 2030, 2050 and 2070. The yield of two Boro varieties (BR 3 and BR 14) have been simulated in the present study for the years 2008 (representing present time), 2030, 2050 and 2070, using DSSATv4 (Decision Support System for Agrotechnology Transfer, version 4) modeling system.

2.1 Selection of the Study Area

In this assessment, among 37 meteorological stations, 11 representative stations i.e. Khulna and Barisal from south-west region and Chittagong from south-west region, Feni , Comilla and Jessore from south-east region, Faridpur, Dhaka and Tangail from central region and Sylhet and Mymensingh from the eastern region of Bangladesh have been selected. Fig.1 shows the location of the stations used in this analysis. A simulation study was conducted to assess the vulnerability of Boro production to potential climate change. As a study area, more stations from the south-west region and the south-east region (Debsharma, 2003) of Bangladesh have been chosen as these regions are very sensitive to change of temperature and precipitation, and some of these stations are major rice growing areas in Bangladesh.

2.2 The Software: DSSAT Version 4

DSSAT model refers to Decision Support System for Agrotechnology Transfer, which enables its users to match the biological requirements of crops to the physical characteristics of land so that objectives specified by the user may be obtained. The Rice model of the DSSAT model system (v4) simulates crop growth, development and yield taking into account the effects of weather, management, genetics, and soil water, CO2 and N-system information provided as inputs to the model.

The decision support software consists of-

1) A Data Base Management System (DBMS) to enter, store, and retrieve the 'minimum data set' needed to validate, list and use the crop models for solving problems;

2) A set of validated crop models for simulating processes and outcomes of genotype by environment interactions; and

3) An applications program for analyzing and displaying outcomes of long-term simulated agronomic experiments.

2.3 Weather Data

The 'tools' menu of the DSSAT models includes a program WeatherMan, which is designed to simplify or automate many of the tasks associated with handling, analyzing, and preparing weather data for use with crop models or other simulation software. WeatherMan has the ability to translate both the format and units of daily weather data files, check for errors on import, and fill-in missing or suspicious values on export. WeatherMan can also generate complete sets of weather data comprising solar radiation, maximum and minimum temperature, rainfall, and photosynthetically active radiation. These data were obtained for the selected years of 2008, 2030, 2050 and 2070 from a regional climatic parameter forecasting model named 'Providing Regional Climates for Impacts Studies', (PRECIS), developed by The Hadley Centre of United Kingdom. The PRECIS outputs, which were used in the DSSAT model, include daily maximum temperature (Tmax) in ^{o}C, daily minimum temperature (Tmin) in ^{o}C, daily incoming solar radiation (SRAD) in MJ/m^{2}/day, and daily precipitation (RAIN) in mm.

2.4 Simulation of Yield of Rice using ICSim

ICSim is a MS Windows application program that is used as an add-on to DSSAT v4, which refers to as Introductory Crop Simulation Model (version 4.0.2). ICSim uses the DSSAT v4 files, directories, and crop models and communication protocol to calculate the yield of any specified variety of crop. In particular, this program is used in this study to specify weather, soil, and main practices to simulate crop growth and yield using the crop models BR 14, and BR 3 (Boro) and data sets contained in DSSAT v4 as weather (.WTH) files. The program also is used computes potential yield, assuming no limitations in water and nitrogen for the same inputs to provide an immediate demonstration of the concept of yield analysis by comparing potential yields with yields that may be limited by water or nitrogen constraints or both.

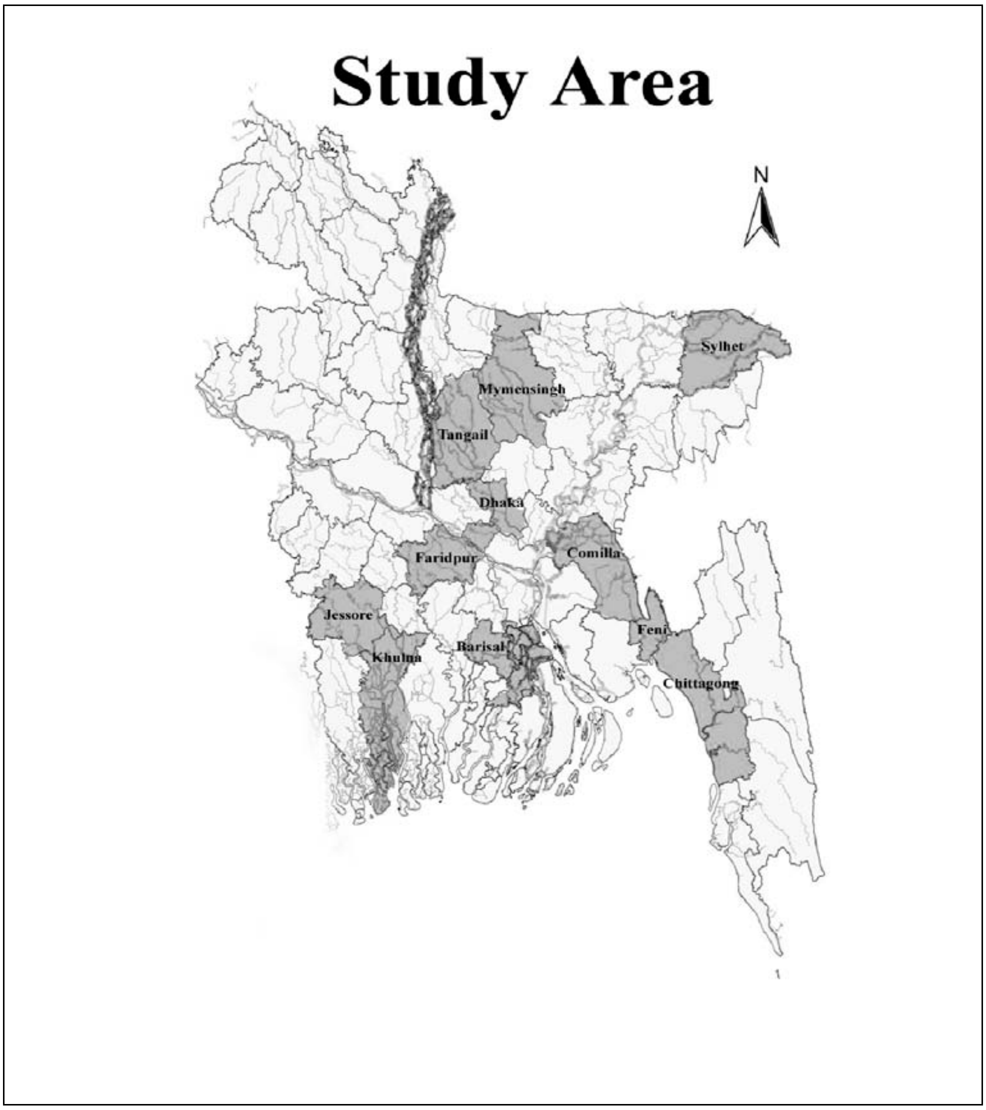

Figure 1. The location of the stations used in this analysis.

2.5 Crop Management Data used in the Model simulation

It may be noted that weather data requirement for DSSAT model include daily maximum and minimum air temperatures, daily precipitation and daily solar radiation, all of which could affect rice yield significantly. Therefore, in this study, future climate scenarios, including daily maximum and minimum temperatures (^{o}C), precipitation (mm) and solar radiation (MJ/m^2 /day), for the selected locations of Bangladesh have been collected and used for predicting yield of the cultivars of Boro. Table 2 depicts the Crop Management Data used for the model simulation:

Table 2. Crop Management Data

Parameter	**Input data**
Planting method	Transplant
Transplanting date	1st January
Plant population	33 plants per m^2
Row spacing	20 cm
Fertilizer (Urea) application	(3times in total growing period)
18 days after transplanting	30 kg ha^{-1}
38 days after transplanting	70 kg ha^{-1}
56 days after transplanting	30 kg ha^{-1}
Application of irrigation (method-Flood)	Total 855 mm in 14 applications
Day 1	80mm
Day 7	80mm
Day 14	80mm
Day 21	70mm
Day 28	70mm
Day 35	70mm
Day 42	70mm
Day 50	70mm
Day 60	50mm
Day 70	50mm
Day 80	45mm
Day 90	40mm
Day 100	40mm
Day 110	40mm
Amount of CO_2	
year 2008	379ppm
year 2030	427ppm
year 2050	465ppm
year 2070	503ppm

2.6 Soil Characteristic

The model uses a detailed set of specific inputs coefficients that account for differences in growth and development among cultivars. In this study, default soil profile i.e. Medium Silty Clay used for clayey soil, Medium Silty Loam used for silty clayey loam and silty loam. The Soil data used in the simulations are as follows in Table 1:

Table 1. Soil Data

Station	Latitude	Longitude	Soil Type
Dhaka	23.78	90.38	Medium Silty Loam
Faridpur	23.93	89.85	Medium Silty Loam
Mymensingh	24.73	90.42	Medium Silty Loam
Sylhet	27.90	91.88	Medium Silty Loam
Tangail	24.25	89.93	Medium Silty Loam
Jessore	23.18	89.17	Medium Silty Clay
Feni	23.02	91.39	Medium Silty Loam
Comilla	23.46	91.18	Medium Silty Loam
Chittagong	23.37	91.80	Medium Silty Loam
Khulna	22.82	89.55	Medium Silty Clay
Barisal	22.70	90.36	Medium Silty Loam

2.7 Selection of Rice variety

The Decision Support System for Agrotechnology Transfer, version 4 (DSSAT) is a modeling system which is variety-specific. This model can calculate the yield of certain variety of crops in response to various environmental conditions. In predicting crop growth and yield, the model takes into effect of weather, crop management, genetics, and soil water, C and N. The model uses a detailed set of crop specific genetic coefficients, which allows the model to respond to diverse weather and management conditions. Therefore, in order to get reliable results from model simulations, it is necessary to have the appropriate genetic coefficients for the selected cultivars.
For this study, the two varieties of Boro rice, (BR3 and BR 14) are selected because of the availability of their genetic coefficients in this model.

3. RESULT AND DISCUSSION

The assessment gives a general idea of increasing and decreasing trend of yield of different varieties of rice over next 60 years. This analysis takes into account only the environment parameters like precipitation, maximum and minimum temperature, solar radiation and carbon dioxide. Some ideal crop management parameters (i.e. plant population, row spacing etc.) are assumed to provide a common base of comparison.2 cultivars are considered in the analysis, which are Boro (BR 3 and BR 14) for the selected meteorological stations in the all over Bangladesh.

3.1 Effect of climate change on BR 3 Cultivar

Table 3 shows predicted BR3 rice yield for the selected locations. From this table, it is observed that yield more or less reduces across all the stations particularly of the southern region of Bangladesh over the time period. Simulated BR3 rice yield ranged from 3029kg/ha (Comilla) to 6061 kg/ha (Jessore) in 2008, from 1059 kg/ha (Sylhet) to 4273kg/ha (Khulna) in 2030, from 2867 kg/ha (Dhaka) to 3937kg/ha (Chittagong) in 2050 and from 2883kg/ha (Jessore) to 4773 kg/ha (Mymensingh) in 2070. Table 3 shows that these significant reductions of rice yields will occur due to predicted changes in climatic condition. From the table 3, with respect to 2008, yields decreased in 2030, 2050 and 2070. But in 2070, yields increased at some of the selected stations with respect to yields in 2030. Compared to 2008, predicted average reductions of yields of BR3 variety for the selected stations are about 22% in year 2030, 28% in year 2050 and 55% in year 2070. Maximum yield was predicted at Jessore in 2008. For 2030, 2050 and 2070 yield reduction of 13%, 22% and 30% respectively occurs at Khulna. The reduction of yield in the other selected stations is also given in Table 3. Fig 2 - 11 shows individually predicted yield of BR3 and BR14 rice variety for selected stations. Fig 12-15 shows station wise predicted yield of selected rice variety.

3.2 Effect of climate change on BR 14 Cultivar

Table 3 also portrays the predicted BR14 rice yield for the selected locations. It has been observed that yield reduces across all stations and over the time period (except for slightly deviated results in the north eastern zone). Simulated BR14 rice yield ranged from 1958kg/ha (Faridpur) to 4873 kg/ha (Jessore) in 2008, from 724kg/ha (Sylhet) to 3106 kg/ha (Khulna) in 2030, from 1700kg/ha (Faridpur) to 3052kg/ha (Mymensingh) in 2050 and from 2044kg/ha (Tangail) 3505 kg/ha (Mymensingh) in 2070. From observing the Table 3, it is clearly understood that these significant reduction of rice yields will occur due to predicted changes in climatic condition particularly in the major rice yielding districts of Bangladesh, i.e. the southern region. From the Table 3, with respect to 2008, yields decrease in 2030, 2050 and 2070 in general, except for some deviations in the central and north eastern zones, which might be attributed to use of erroneous data.

In general, at the stations of the lower southern part of the country known as major rice-producing districts, rice yield is decreased almost linearly and significantly up to the year 2070 from 2008. However, some stations in the eastern Bangladesh showed a decreasing trend from 2008 to 2050 and then an increasing trend in 2070.

An assessment was made of yield in non-stressed condition which means computation of the potential yield, assuming no limitations in water and nitrogen for the same inputs to provide an immediate demonstration of the concept of yield analysis by comparing potential yields with yields that may be limited by water or nitrogen constraints or both. This has shown that yield in non-stressed condition increases roughly 15-35% in 2008, 30-60% in 2030, 20-30% in 2050 and 2-13% in 2070 (for BR 3 BORO).

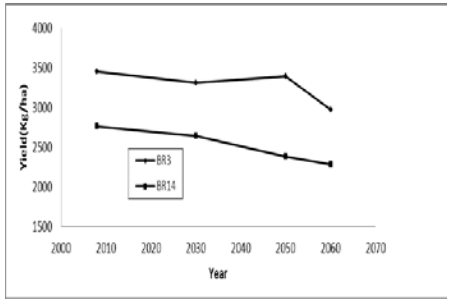

Figure 2. Predicted yield of BR3 and BR14 varieties of rice for Barisal.

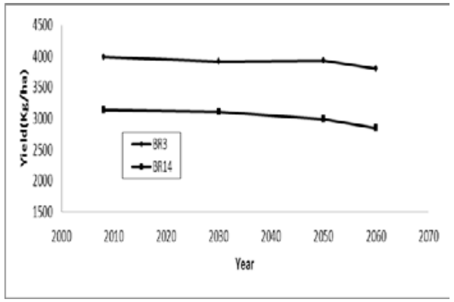

Figure 3. Predicted yield of BR3 and BR14 varieties of rice at Chittagong.

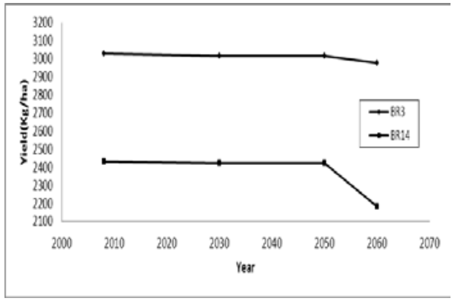

Figure 4. Predicted yield of BR3 and BR14 varieties of rice at Comilla.

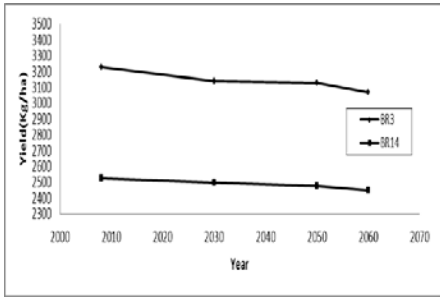

Figure 5. Predicted yield of BR3 and BR14 varieties of rice at Feni.

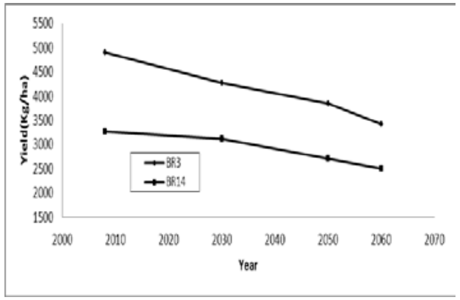

Figure 6. Predicted yield of BR3 and BR14 varieties of rice at Khulna.

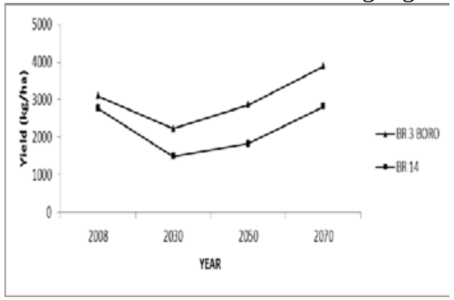

Figure 7. Predicted yield of BR3 and BR14 varieties of rice at Dhaka.

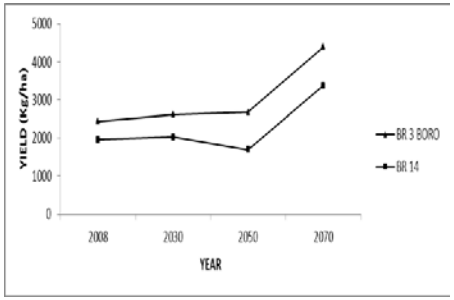

Figure 8. Predicted yield of BR3 and BR14 varieties of rice at Faridpur.

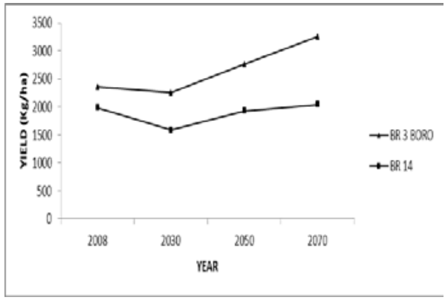

Figure 9. Predicted yield of BR3 and BR14 varieties of rice at Tangail.

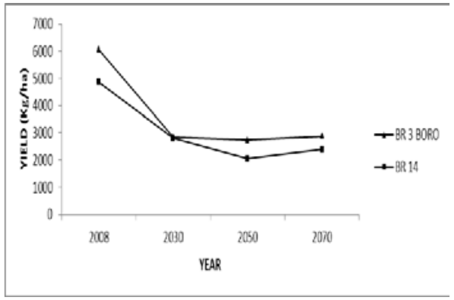

Figure 10. Predicted yield of BR3 and BR14 varieties of rice at Jessore.

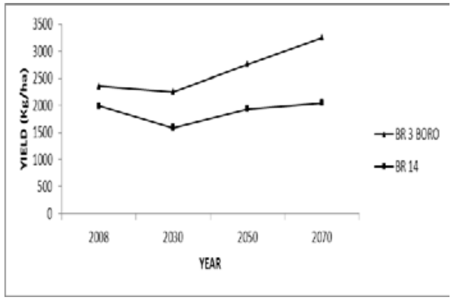

Figure 11. Predicted yield of BR3 and BR14 varieties of rice at Mymensingh.

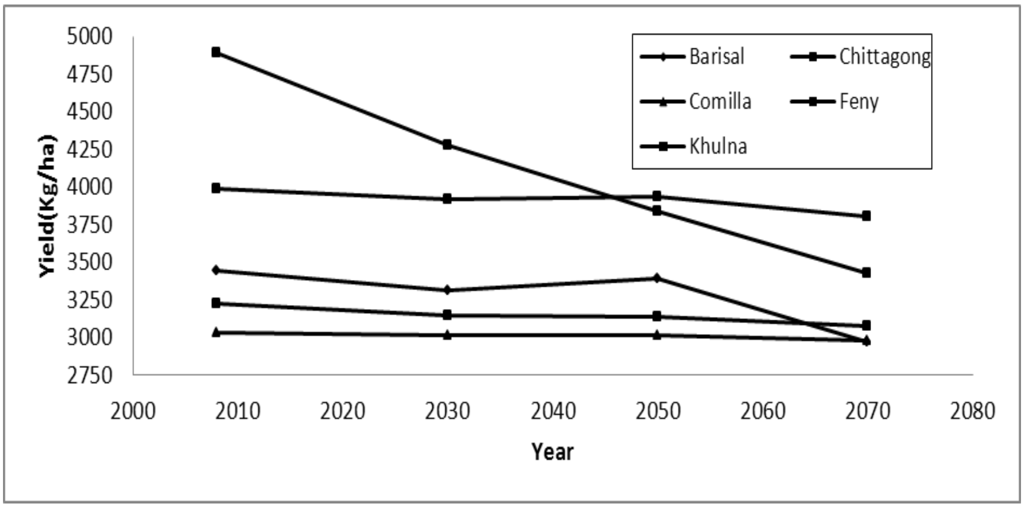

Figure 12. Predicted yield of BR3 variety of rice at different stations of southern region.

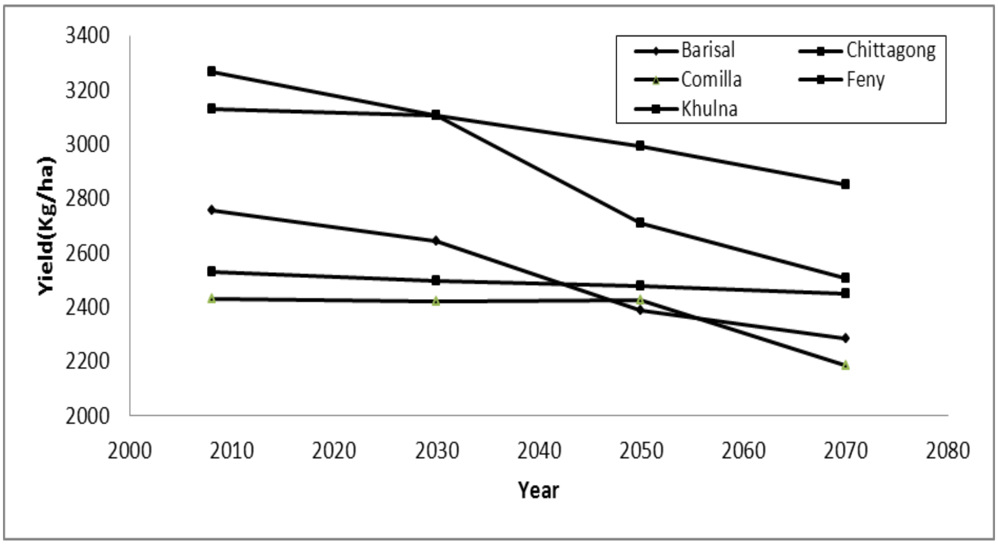

Figure 13. Predicted yield of BR14 variety of rice at different stations of southern region.

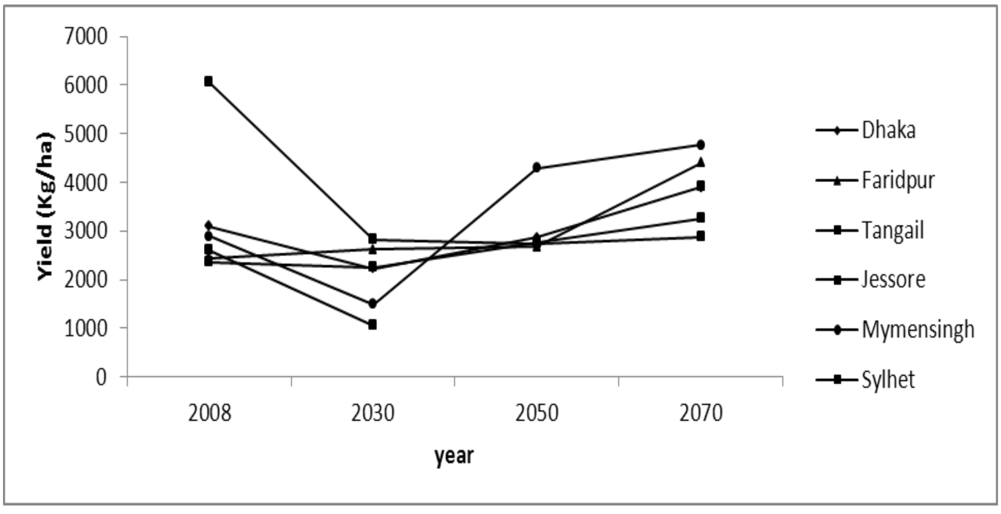

Figure 14. Predicted yield of BR3 variety of rice at different stations of central and eastern region.

Table 3. Predicted yield Boro rice (kg/ha) at selected locations for the years 2008, 2030, 2050 and 2070.

Station		Barisal		Chittagong		Comilla		Feni		Khulna		Dhaka		Faridpur		Tangail		Jessore		Mymensingh		Sylhet	
		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar		Cultivar	
		BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14	BR3	BR14
2008	Yield (Dry Weight)	3445	2758	3987	3129	3029	2432	3227	2528	4895	3266	3097	2769	2446	1958	2355	1989	6061	4873	2898	2392	2607	1772
2030	Yield (Dry Weight)	3311	2641	3914	3103	3013	2422	3140	2497	4273	3106	2229	1492	2625	2022	2252	1586	2830	2794	1498	985	1059	724
2050	Yield (Dry Weight)	3386	2387	3937	2993	3014	2423	3131	2476	3839	2709	2867	1824	2675	1700	2760	1931	2743	2047	4293	3052	****	****
2070	Yield (Dry Weight)	2972	2283	3799	2849	2977	2183	3070	2449	3422	2503	3892	2817	4401	3375	3253	2044	2883	2398	4773	3505	3909	2969
% change in yield for 2030(respect to 2008)		-3.9	-4.2	-1.8	-0.8	-0.5	-0.4	-2.6	-1.2	-13	-4.9	-28	-46	7.3	3.3	-4.4	-5.4	-53	-43	-48	-58	-59	-59
% change in yield for 2050(respect to 2008)		-1.7	-13	-1.2	-4.3	-0.4	-0.4	-2.9	-2.1	-22	-17	-7.4	-34	9.4	-13	17.2	-38	-54	-58	48.1	27.6	****	****
% change in yield for 2070(respect to 2008)		-14	-17	-4.7	-8.9	-1.7	-10	-4.9	-3.1	-30	-23	25.7	1.7	79.9	72.4	38.1	-68	-52	-51	64.7	47	49.9	68
% change in yield for 2050(respect to 2030)		2.3	-9.6	0.58	-3.5	0.03	0.04	-0.3	-0.8	-10	-13	28.6	22.3	1.9	-16	22.6	21.8	-3.1	-27	186	209	****	****
% change in yield for 2070(respect to 2050)		-12	-4.3	-3.5	-4.8	-1.2	-9.9	-1.9	-1.1	-11	-7.6	35.8	54.4	64.5	98.5	17.9	5.9	5.1	17.1	11.2	14.8	****	****

****values are missing for the Station Sylhet for the year of 2050.

4. CONCLUSIONS

Agriculture practices in Bangladesh are largely dependent on precipitation and rainfall patterns. Severe drought in winter and extreme rainfall during monsoon is more or less common in Bangladesh with significant crop losses during both extreme conditions. The main scope of this assessment was to understand the nature of risks which are caused by climate changes, where natural and human systems are likely to be most vulnerable and what may be achieved by adaptive responses. After analyzing all the predicted data, it can be said that climate change has threatening effects on yield of crop production. This assessment has been conducted with some limitations. In spite of limitation, efforts were made to complete the assessment with minimum error.

ACKNOWLEDGEMENTS

The Author is very grateful to Dr. M. Ashraf Ali, Professor of Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET) for his enormous support and encouragement and authority of Bangladesh Meteorological Department (BMD) who have helped wholeheartedly.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ADAPTING TO CLIMATE CHANGE IN URBAN AREAS OF BENGAL DELTA: A CASE STUDY ON KHULNA CITY

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ABSTRACT

Several researches and studies have predicted that significant changes to the rainfall pattern and evapotranspiration will occur in the Bengal delta due to climate change. In addition to the increase in sea level along the coast, increase in the flood flows of Ganges, Brahmaputra and Meghna Rivers are expected. Changes to the climate and hydrology in the Bengal delta compounded with the low topography, large population density, inadequate infrastructure and lack of institutional capacity has made this area one of the most vulnerable to the adverse impacts of climate change. Climate change impacts in the urban areas of Bengal delta will be severe. The existing drainage system in large cities is already proved to be inadequate under increasing frequency of higher intensity rainfalls events. In the climate change scenario, when the higher intensity storms are predicted to occur more frequently, the water logging problem will aggravate further. The existing water supply sources would also become vulnerable because of prolonged and more frequent dry spells. Increased salinity intrusion in the southern part of the delta will be a great challenge to water supply in the southern districts. This paper presents a case study conducted for the Khulna city where climate change impacts were assessed on the drainage system and water supply sources using mathematical models. It identified several adaptation measures against climate change impacts and made a financial evaluation on the adaptation options.

Keywords: *Climate change, Sea level rise, adaptation, urban areas, mathematical models*

1. INTRODUCTION

Bangladesh has a high probability of being heavily affected by climate change due to increase in sea level along the coast and increase in the flood flows of Ganges, Brahmaputra and Meghna Rivers. Changes to the climate and hydrology in the Bengal delta compounded with the low topography, large population density, inadequate infrastructure and lack of institutional capacity has made this area one of the most vulnerable to the adverse impacts of climate change in the world. In the context mentioned, a study on 'Strengthening the Resilience of the Water Sector in Khulna to Climate Change' was carried out by IWM jointly with Alterra of the Netherlands in 2009-10 financed by the Asian Development Bank (ADB). The case study presented herein draws from the experience and outcome of this study.

The city of Khulna, being located in the coastal area of Bangladesh, and influenced by tides from the Bay of Bengal, is highly vulnerable to climate change. The increasing salinity intrusion in the rivers adjacent to the city and the anticipated sea level rise might have a major impact on the water resources and water and drainage infrastructure of the city and its surrounding areas. The city experiences frequent water logging during rainy season. The possible increase in precipitation due to climate change coupled with sea level rise will make the situation even worse.

The analyses of impacts of climate change scenarios on flooding, urban drainage and salinity intrusion were conceived to be based on dynamic mathematical modelling followed by presentation of results in a Geographic Information System (GIS). The outputs from the modelling exercises were further used by different sectoral specialists to determine the impacts on socio-economy, urban planning and public health aspects.

2. METHODOLOGY

The approach to assessing climate risks and impacts consists of the following steps: (i) making climate change projections for Khulna in 2030 and 2050, (ii) developing socio-economic development scenarios for Khulna in 2030 and 2050, (iii) developing and calibrating three mathematical models, (iv) running the models for 2030 and 2050, using the projected climate and socio-economic changes, (v) assessing impacts through GIS mapping, (vi) identifying and analyzing adaptation options. A flow diagram showing the approach and work flow is given in Figure 1.

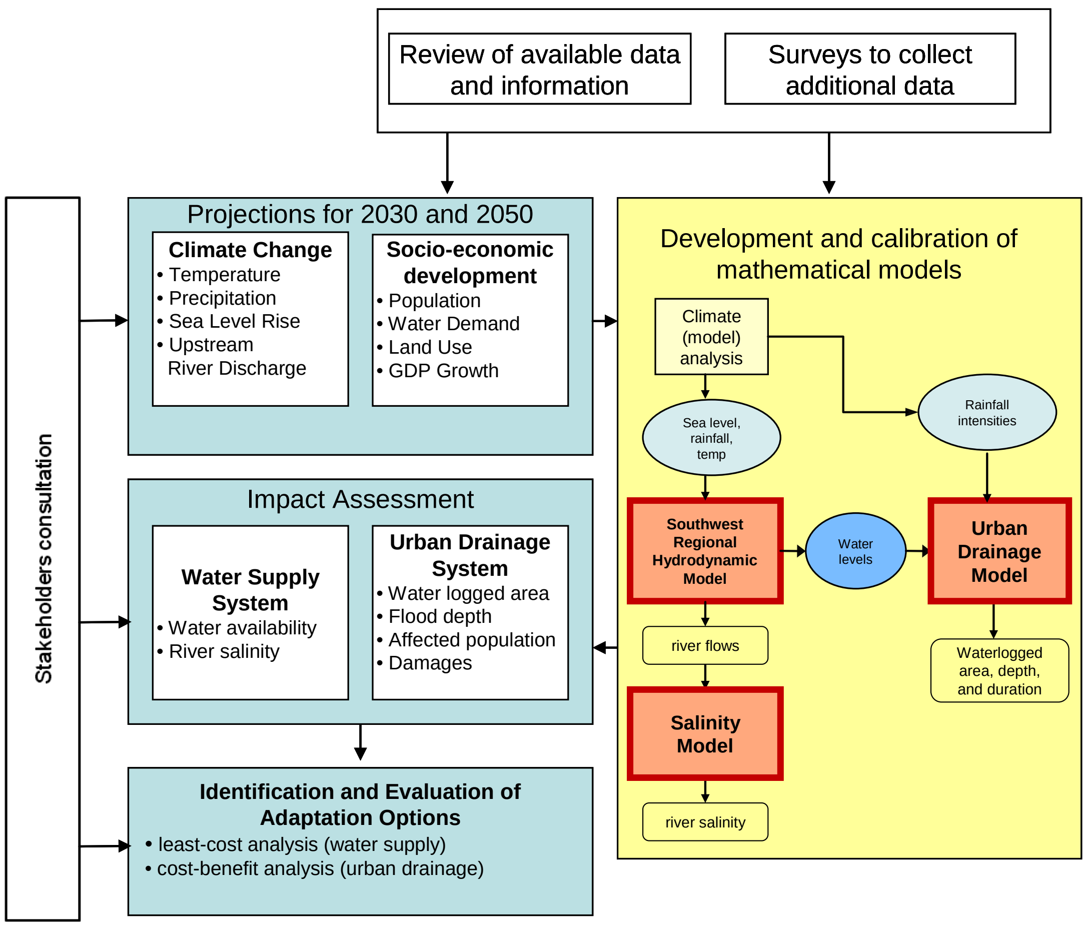

Figure 1: Flow diagram of activities

3. OVERVIEW OF STUDY AREA

The study area is about 45 km^2 under the Khulna City Corporation (KCC). The KCC area is bounded in the east by the Bhairab-Rupsha River, in the west by the Khudi Khal-Mayur River and in the south by the Harintana Khal. In the north lie the low lying plains of Teligati. According to the land use survey undertaken for Khulna's Masterplan (Aqua-Sheltech, 2002), about 46% of the built-up area is occupied by residential housing, 15% is under industrial use, about 5% is under commercial use. The remaining land use in the built-up area consists of transport infrastructure, official buildings, community and defence, facility parks and water bodies. The land use pattern of Khulna has been substantially influenced by the flow of the Rupsha and Bhairab rivers. As a deltaic plain the land is flat and poorly drained. The whole metropolitan area is approximately 2.5m above the mean sea level. Such low topography of the city is an obstacle to the development of a proper land use structure.

The present water supply to Khulna is mainly from groundwater sources drawn from both deep and shallow tube wells. To cope with current insufficient supply and increasing demand, the Khulna Water Supply and Sewerage Authority (KWASA) plans to construct a new treatment plant for surface water. The gradual increase in salinity in Khulna has been evident since mid seventies. Salinity near Khulna was the highest in 2007 according to records of past 32 years. Sea level rise and prolonged dry weather are expected to increase further the salinity levels in Khulna. Khulna currently suffers recurring and worsening water logging problems. The situation could be further exacerbated by increased rainfall and SLR caused by climate change.

4. FUTURE SCENARIO

4.1 Climate Change Scenarios

Two climate change scenarios from the Intergovernmental Panel on Climate Change (IPCC) were considered: A2 and B1. A2 is termed as business-as-usual scenario, with heterogeneous, market-led world, and more rapid population growth but less rapid economic growth than A1. B2 is termed as sustainable development scenario, with lower CO_2 emissions. Considering these two scenarios, statistical downscaling was undertaken from a global climate model (GCM) (ECHAM5-MPI-OM) to a finer resolution (0.5 X 0.5^o), because statistical downscaling is preferred for rainfall analyses, the main concern of the study. The model outputs were bias corrected for rainfall and temperature. Findings include in Table 1.

Table 1: Summary Features of Climate Projections

Scenario	**A2**	**B1**
Temperature	Average monthly temperature rise by 2050 would vary from +0.5^{o}C in October to +1.7^{o}C in January and February	Average monthly temperature rise by 2050 would vary from +0.5^{o}C in June, July and August to to +1.5^{o}C in February and April
Rainfall	Annual rainfall would increase by about 5% by 2050 (1,860 mm/year) from the reference period	Annual rainfall would increase by about 9.3% by 2050 (1,739 mm/year) from the reference period
Seasonal Rainfall	Increase in July-September by 4.6% but decrease in December-February by 26.3%	Increase in July-September by 10.5% but decrease in December-February by 46.2%
Rainfall Intensity	50 mm or more rainfall in 6 hours would increase from 4.2 times per year to 5.9 times per year in 2050	50 mm or more rainfall in 6 hours would marginally increase from 4.2 times per year to 4.25 times per year in 2050

(Source: ADB 2010)

These results are, however, based on the outcomes of a single GCM. Other models indicate similar changes, but there are still large uncertainties about the possible change in rainfall in Bangladesh.

As for the SLR, two different levels, plausible high and plausible low, were used in the different model runs, due to the high uncertainty in the levels. The plausible low scenarios, 10 cm in 2030 and 20 cm in 2050, are about the mid-range of the IPCC scenarios, and the plausible high scenarios, 25 cm in 2030 and 40 cm in 2050, assume significant melting of land ice. This is in line with the findings of recent new models and researches (IPCC, 2007).

4.2 Socio-economic Scenarios

For the study, numerous assumptions were made with regard to socio-economic scenarios of Khulna in 2030 and 2050. Socio-economic surveys, various census data, literature review, discussions with relevant agencies and experts became the basis of scenario development. Key parameters are shown inTable 2 .

Table 2: Key Parameters for Socio-economic Development

Parameter	Assumption	Remarks
Population in KCC area	2% growth per annum: (i)976,000 in 2010:(ii)1,450,000 in 2030, and (iii) 2,155,000 in 2050	Population distribution among the wards (total 31) is assumed to be maintained.
Water Demand	Per capita domestic demand of 120 liters per day in 2030, 150 litres per day in 2050	100% of the population in the KCC area will be served by the proposed water supply system.
GDP Growth Rate of Khulna	6.2% per annum	A national average rate of growth between 2001 and 2009 is 6.2%.
Urban Development	Proportion of impervious areas (weighted average):(i)17.7% in 2010 (ii) 29.0% in 2030, and (iii) 38.7% in 2050	For each subcatchment area, change in the proportion between pervious and impervious areas was projected.

GDP = Gross domestic product, KCC= Khulna city corporation
(Source: ADB 2010)

4.3 Integration of Scenarios

Integrated scenarios were developed for socio-economic change and climate for the years 2030 and 2050. To account for the uncertainties involved three integrated scenarios were developed: a trend scenario (called the realistic scenario, assuming business-as-usual), an optimistic scenario and a pessimistic scenario. With the help of the modelling framework and using the trend scenario, various adaptation options were studied and the impacts of the options on evaluation criteria were analysed. The adaptation strategies developed for this study were discussed with the stakeholders to receive feedback and to assess the level of public acceptance.

5. CLIMATE CHANGE IMPACT ON WATER SUPPLY AND DRAINAGE

To assess the impact of climate change hydraulic models were used. The models take into account the output from the global climate models and use them to assess the impact on the water resources of an area. Three mathematical models were developed and/or calibrated to assess the impacts of climate change. Roles of three models, namely South West Regional Hydrodynamic Model (SWRHM), Salinity Model (SM), and Urban Drainage Model (UDM), are shown in Figure 2.

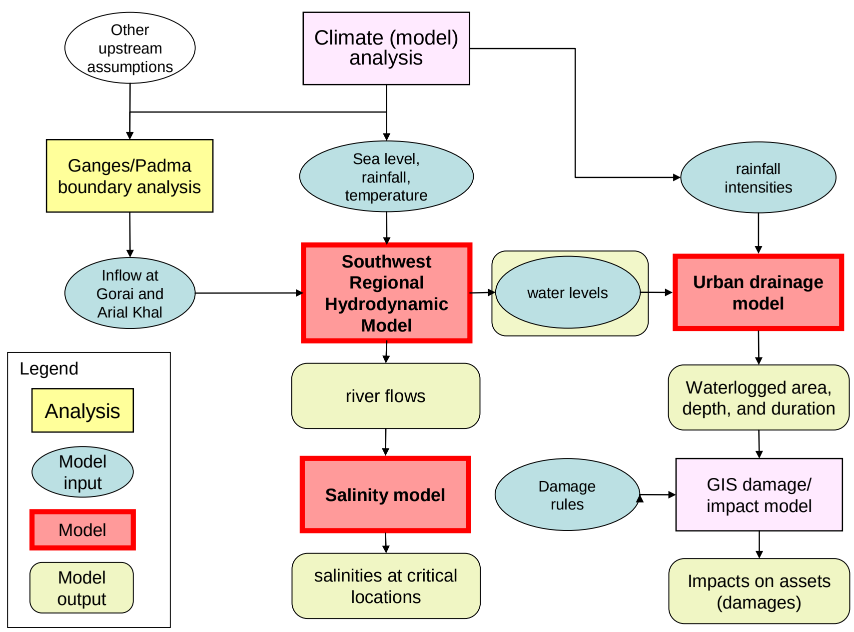

Figure 2: Model Framework

5.1 Impact on Water Supply

The impacts of climate change on the Khulna water supply were assessed in terms of changes in (i) dependable river flow (water availability) from January to June when the water flow will be less, and (ii) maximum salinity levels and duration of river Chloride levels above 600 mg/l and 1000mg/l (GOB 1997). These impacts were estimated at Haridashpur, Mollarhat, Upstream of Phultala, Phultala, Daulatpur, Labanchara which are potential locations to build surface water treatment plants (SWTP) to provide water to Khulna.

5.2 Impacts on Water Availability

During the dry season, the river flows in the southwest region are highly dependent on upstream inflows (from the Ganges and Padma Rivers) and also on local inflows from groundwater. In Figure 3, the 95% dependable flows from January to June at the locations of interest are shown for the base and two reference cases (3a and 5a). For all locations there is a marginal decrease in dependable river flows in Runs 3a (Ref. 2030) and 5a (Ref. 2050) compared to Run 1 (Base Case). The decrease in all locations is less than 9% of the 95% dependable flows in Run 1.

The proposed intake flow rate for water supply is estimated by JICA to be 1.27m^3/s in 2020 and 2.93 m^3/s in 2030 (JICA, 2010). In this study, the 2050 intake rate is estimated to be 4.71m^3/s. All three of these intake rates are well below the 95% dependable flows at all the candidate intake locations in the climate change scenarios. At Mollarhat, the proposed intake location, the intake requirements are 4% and 6% of the 95% dependable flows in 2030 and 2050 respectively.

Increased groundwater abstractions and direct river water withdrawals upstream of these 6 locations have not been considered in this study. Therefore these 95% dependable flow estimates for 2030 and 2050 should be considered as optimistic. Nevertheless, it would be reasonable to conclude that climate change in 2030 and 2050 would not significantly affect the water availability of the proposed surface water supply system.

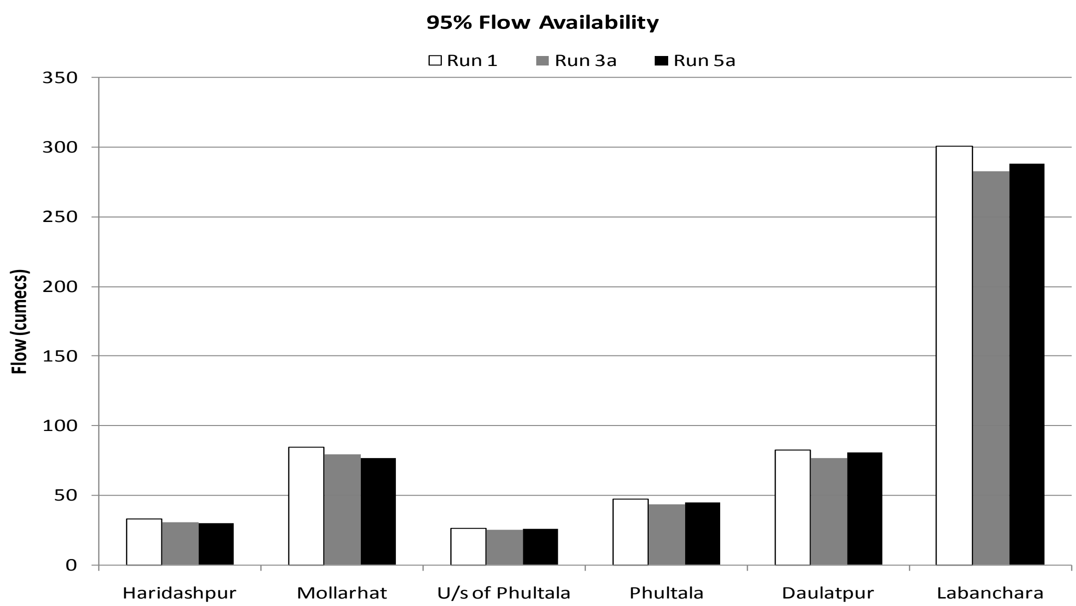

Figure 3: Flow availability (95%) in base and reference scenarios (source: ADB 2010)

5.3 Impacts on River Salinity

Figure 4 presents a probability plot on the number of days that the salinity is exceeded at Mollarhat for the five scenarios. The y-axis is the number of days in a year that the maximum daily Chloride level exceeds 1,000 mg/l. The x-axis is the probability that the number of days the river is too saline in any given year (based on the 20-year simulations) for each scenario. For Run 1, the base run salinity profile crosses the x-axis at around 33% i.e. this is likely to occur in about 1 out of 3 years in current conditions. This means that there is a 33% chance that the salinity at Mollarhat will exceed the 1000mg/l limit for at least 1 day in any given year in the Base conditions. At the other end of the plot, for Run 1, there is a 5% chance (about 1 in 20 years) that the number of saline days at Mollarhat can be 55 days or higher. In Run 3a (2030 climate change with high SLR), the number of saline days is similar with the same exceedance probability (5%). In 2050 the duration increases to 75 days with 5% exceedance probability. In the last 2-3 years, there is an increasing trend in simulated salinity levels at Mollarhat. In Figure 4, the higher duration of salinity levels (> 1000 mg/l Chloride) are from these recent years in Run 1 (Base case). Some measures have to be taken in order to provide water that meets the drinking water quality standard, and such needs will be even greater with climate change.

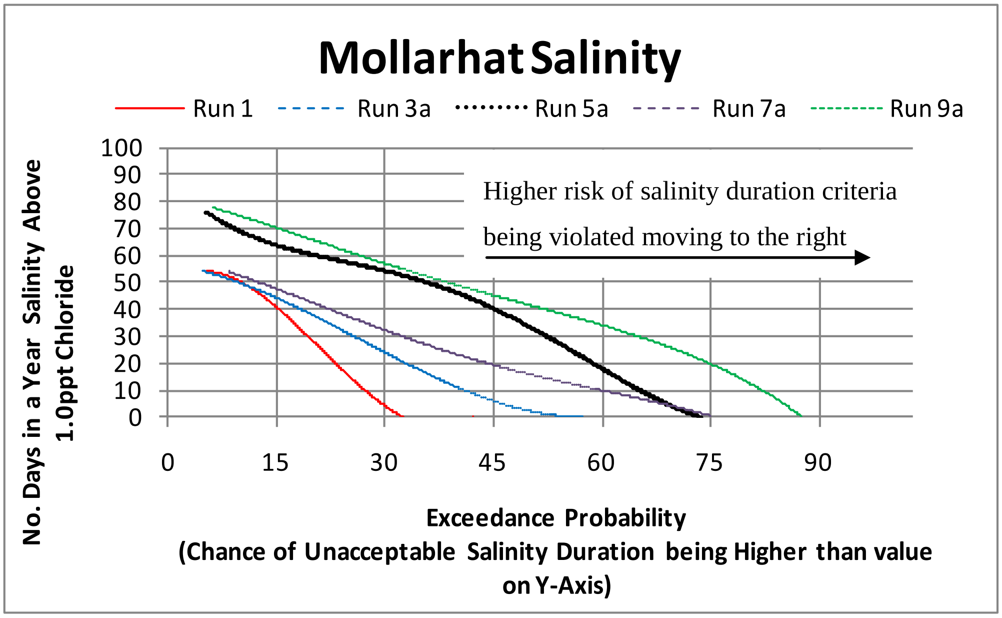

Figure 4: Change in salinity duration curves at Mollarhat due to climate change

Notes: 3a = 2030, A2, High SLR, 5a = 2050, A2, High SLR, 7a = 2030, B1, High SLR, 9a = 2050, B1, High SLR. Durations are for 15% exceedance probability. This means that there is a 15% chance that this duration can be exceeded in any year. Another way to view this is that this duration of salinity can occur once in around every 7 years. (Source: ADB 2010)

6. IMPACT ON URBAN DRAINAGE

Impacts on the drainage conditions were assessed by assuming future socio-economic development and no improvement in the drainage system. Table 3 shows the results. The waterlogged area with damaging water depth (higher than 30 cm) increases from 29 to 34% for 2030 (Run 3a) and to 54% for 2050 (Run 5a) for a 1 in 10 year flood event. Figure 5 & Figure 6 shows the projected flooded area under the 1 in 10 year flood event in 2050 for without- and with-improvement and climate change scenario A2 and high SLR. It has been found from analysis of such flood maps that due to climate change, water logging for 1 in 5 year event in 2030 will be close to 1 in 10 year water logging for the base condition while 1 in 5 year water logging in 2050 will be close to 1 in 10 year water logging in 2030. The average water depth in the waterlogged area will increase from 41 cm in the base case to 49 cm in 2030 and 63 cm in 2050 under the 1-in-10 year flood event. Population exposed to flooding will increase from 24% in the base case to 41% in 2030 and 58% in 2050.

Table 3: Percent of City Area Waterlogged with Climate Change (A2 and High SLR)

Design Event	Measures	FF 0-10 cm	F0 11- 30 cm	F1 31-60 cm	F2 61-90 cm	F3 91-180 cm	Damaging Water Depth F1+F2+F3
Base	None	81	7	11	1	0	11
1-in-5 year	Improvement	92	6	2	0	0	2
Base	None	64	7	26	3	0	29
1-in-10 year	Improvement	87	5	8	0	0	8

Design Event	Measures	FF 0-10 cm	F0 11- 30 cm	F1 31-60 cm	F2 61-90 cm	F3 91-180 cm	Damaging Water Depth F1+F2+F3
2030 1-in-5 year	None	71	8	19	2	0	21
	Improvement	83	8	8	0	0	8
2030 1-in-10 year	None	60	7	28	5	0	33
	Improvement	79	7	13	1	0	14
2050 1-in-5 year	None	58	10	28	4	0	32
	Improvement	68	10	21	1	0	22
2050 1-in-10 year	None	42	3	23	28	3	54
	Improvement	64	8	22	7	0	29

It must be noted that the existing drainage system in Khulna is insufficient even without considering any climate change in the future. Therefore, it is necessary that the drainage system needs to be improved to an acceptable level with current climate conditions. The design standard under the study was set as such that the improvement will make 80% area of each ward free from damaging floods, i.e., flood depth of 30 cm or less. Costs for the improvement, comprising such measures as new and re-construction of drains, river dredging, re-excavation of drains with lining, were estimated to be $7.0 million for a 1 in 5 year return period and $10.7 million for a 1 in 10 year return period.

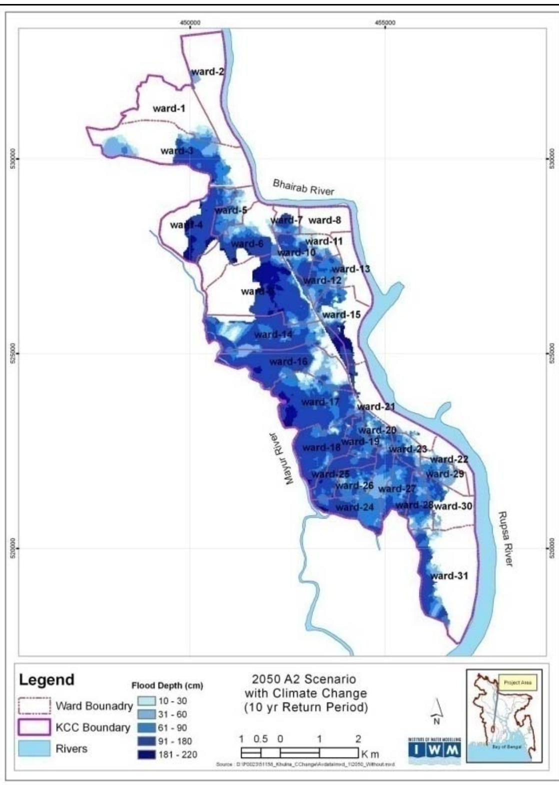

Figure 5: Water logging for 1 in 10 year flood in 2050 under without improvement condition and climate change scenario A2 and high SLR

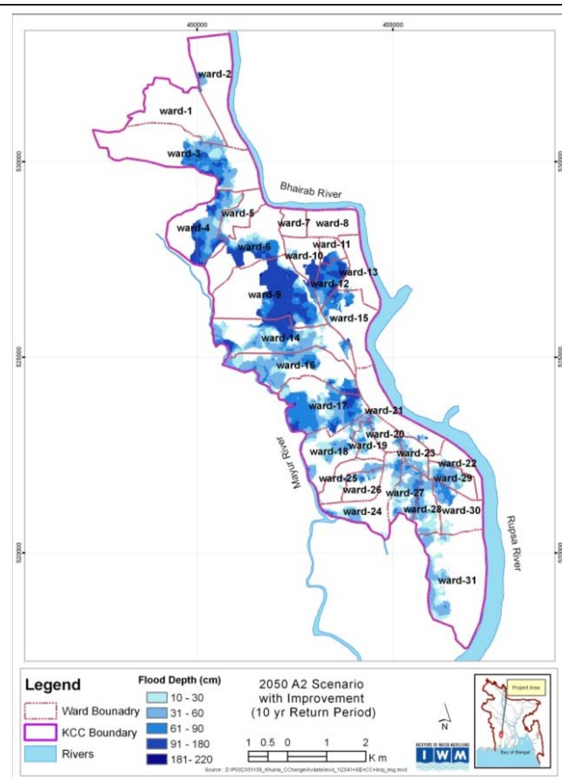

Figure 6: Water logging for 1 in 10 year flood in 2050 under with improvement condition and climate change scenario A2 and high SLR

Table 3 shows how the water logging condition will change in 2030 and 2050 with improvements for 1 in 5 and 1 in 10 year flood events. It shows that with improvements, the water logging area which causes damage would be reduced from 33% to 14% in 2030 and from 54% to 29% in 2050. The average flood depth in the waterlogged area and the population affected by water logging will be reduced from 63 cm to 47 cm, and 58% to 30%, respectively, in 2050 for a 1 in 10 year return period flood. Figure 5 shows the extent of flooding in 2050 under the 1-in-10 year return period with the proposed improvement.

However, both in 2030 and 2050, the design criteria of making 80% area of each ward flood-free cannot be met. Therefore, additional measures, i.e. adaptation measures, will be required to protect the Khulna city from urban flooding.

7. ADAPTATION OPTIONS FOR WATER SUPPLY

Due to increased river salinity levels and longer durations of river salinity, the number of days that river water is not suitable for drinking purposes will increase with climate change. Thus, one core option is to increase the impounding reservoir size so that it can provide alternative supply for a longer period. Another core option is to move the intake point further upstream where the salinity level in the climate change scenario is similar to that in the base case at the original intake point. These two types of core adaptation options have been selected based on practicality and cost-effectiveness.

JICA, in the feasibility study, identified eight intake points, including Phultara, Daulatpur, Labanchara, Mollarhat, and Haridaspur analyzed under this study. By the time of the analyses, it is found that the total investment cost and O&M cost required for each option, and concluded that locating the intake near Khulna city, including at Phultala, Daulatpur, or Labanchara, would not be feasible because of the large size of an impounding reservoir required and availability of suitable land. Desalination was also considered, but rejected because it is too expensive for Khulna's socio-economic condition. Therefore, the study was made on the Madhumati River side (Mollarhat and Haridaspur), focusing on Mollarhat as it was closer to Khulna and would lead to cost saving.

It has been found that the size of impounding reservoir needs to be increased by 12 million cubic meters (MCM) by 2050. On the other hand, moving the intake upstream by 4 km from Mollarhat would also cancel out the effects of climate change.

Evaluation was undertaken to the above two options from economic, social, and environmental viewpoints, and the summary is provided in the Table 4. The least cost analysis was made as the objectives (benefits) of the adaptation options were the same – providing 100% of the population in the KCC area with adequate amount of water that satisfies the drinking water quality standard. This shows that the option 1 costs less and is socially more acceptable.

Table 4: Evaluation of adaptation options

Adaptation options	**Economic**	**Social**	**Environmental**
Option 1 – Relocating the intake point upstream by 4 km by 2050	Investment cost: $8.39 million O&M cost: $24,000 per year	16 households affected, no resettlement	Construction impacts (noise, vibration)
Option 2 - Increase the reservoir size by 12 MCM by 2050	Investment cost: $29.04 million O&M cost: $28,000 per year	More than 20 households affected, including resettlement	Construction impacts (noise, vibration)

(Source: ADB 2010)

In addition to the core options, add-on options were also examined. The add-on options are designed not only to improve the social acceptability of the area but also to make the structure measures more effective and more robust to climate change. Reduction of water leakage, and water demand management were identified as effective add-on options. Implementation of these add-on options may reduce the overall cost of the project in the long run. Two additional issues are worth mentioning at this stage but are excluded in this study. These are: a) separating water supply lines in two parts: one for drinking and cooking purposes with higher price tags per litre, and one for other uses; and b) supply of bottled/jarred water during high saline period instead of expanding the reservoir size. These would reduce the investment cost substantially, while additional management costs would be incurred. Social acceptability would need to be carefully assessed before further consideration of these measures.

8. ADAPTATION OPTIONS FOR DRAINAGE SYSTEM

The adaptation options have been designed for Runs 3a and 5a. As in the case of water supply, two types of adaptation options were considered. The core adaptations contained engineering measures such as additional river dredging, re-excavation of drains with lining, sluice gate improvement, and widening of drains, to address the water logging problem of Khulna in 2030 and 2050 and to ensure that 80% area of each ward remains under flood-free conditions. The permutation and combination of various measures finally provided a set of core adaptation measures. Table 5 shows how the water logging conditions will be reduced with the adaptation measures. The average water logging depth will be reduced to 40 cm, and the population exposed to floods will be reduced to 13%, respectively, in 2050 with a 1 in 10 year flood event. Improved condition of water logging with the adaptation options is presented in Figure 7. However, these are still higher than 33 cm and 6% for the improved drainage system under the current climate. This implies the need for add-on options, which would strengthen the effectiveness of core options. Add-on options identified include good solid waste management to ensure functionality of the drainage system, strict implementation of building codes and land use planning, awareness and education campaigns, and improved prediction and early warning system, which will all contribute to alleviating the flood damage. These options are consistent with the adaptation measures in the context of managing floods provided in the IPCC AR4 (IPCC, 2007) that involve (i) protection against predicted climate change (e.g. structural measures, ensuring functionality of the drainage system), (ii) accommodation to improve resilience (building codes, improved prediction and early warning system), and (iii) retreat to reduce exposure (land use planning).

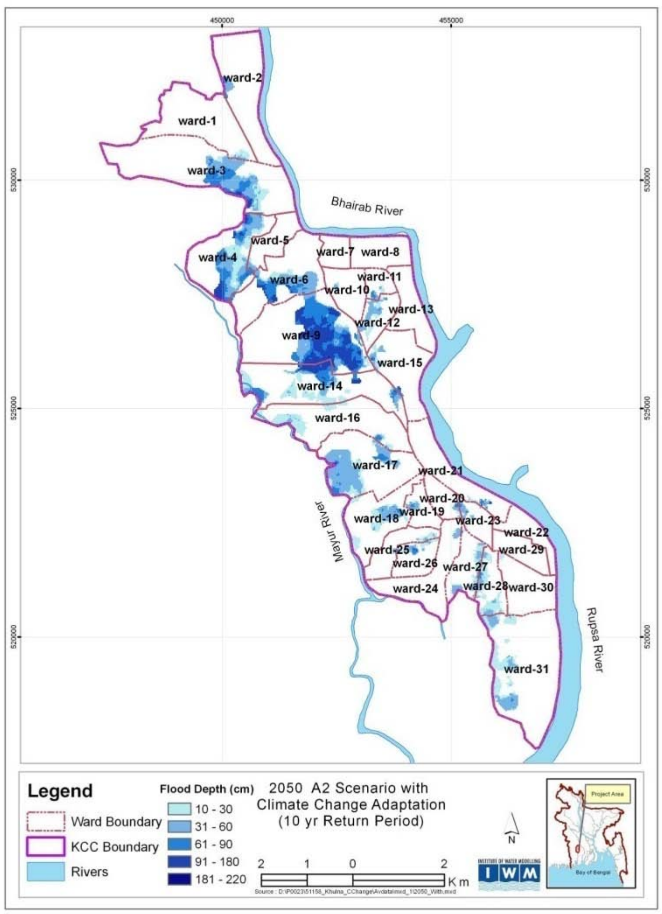

Figure 7: Water logging Map for a 1- in- 10 year Flood Event in 2050 (A2 and High SLR) with Improvement and Adaptation

It was found that the improvements that would be required for the base case were sufficient to address drainage problems for Runs 6a and 8a (B2 with low SLR). So no adaptation measures were considered for these scenarios.

Table 5: Percent of City Area Waterlogged with Climate Change (A2 with High SLR)

Design Event	Measures	FF 0-10 cm	F0 11- 30 cm	F1 31-60 cm	F2 61-90 cm	F3 91-180 cm	Damaging Water Depth F1+F2+F3
2030 1-in-5 year	Improvement and adaptation measures	86	7	7	0	0	7
2030 1-in-10 year	Improvement and adaptation measures	82	7	10	1	0	11
2050 1-in-5 year	Improvement and adaptation measures	80	10	10	0	0	10
2050 1-in-10 year	Improvement and adaptation measures	80	6	11	3	0	14

(Source: ADB 2010)

Evaluation was undertaken to the above adaptation measures. The summary of cost of adaptation measures is provided in Table 6. Cost benefit analysis was made as the benefits (damage costs avoided) of the adaptation measures were different from one set of adaptation options to another. Damages costs under different scenarios are provided in Table 8. Using a 40-years cash flow (from 2010 to 2050) and a 10% discount rate, the benefit-cost ratio is 2.89 for 5-year return period and 4.97 for 10 year return period. Economic internal rate of return (EIRR) is quite high at 34.2% and 111.0% respectively. These figures, however, should be considered indicative only, as significant uncertainties are involved in estimating both costs and benefits as described earlier.

Table 6: Cost of Drainage System Improvements for Different Scenarios

Design Event	*5 Year Return Period*				*10 Year Return Period*			
	Investment Cost		O&M Cost		Investment Cost		O&M Cost	
	Tk (Mil)	USD (Mil)	Tk (Mil)/yr	USD (Mil)/yr	Tk (Mil)	USD (Mil)	Tk (Mil)/yr	USD (Mil)/yr
Base Improvement	493	7			751	11		
2030 Adaptation	64	0.9	24	0.4	39	0.6	15	0.2
2050 Adaptation	1,312	19	101	1.4	1,167	17	89	1.3

Note: 1.Adaptation investment cost for climate change is on top of drainage system improvement cost. Adaptation measures ensure that 80% area of each ward is free from damage causing water logging conditions.

2. Adaptation O&M cost is on top of annual costs without climate change.

(Source: ADB 2010)

Table 7 presents the summary of the cost of climate change for 2050. It shows that the improved drainage system is designed to ensure that under no climate change scenario the 2050 case is equivalent of the base case with respect to damages. However, with climate change there will be significant rise in average water logging depth as well as in the damages. In this case, water logging depth increases to 63 cm while the damages increases to include 58% of the population. If, however, improved drainage system is ensured than both the water logging depth and the damages will be reduced but they remain above the reference case scenario of without climate change. Under a Realistic Scenario, the water logging depth in 2050 would have increased from 41 cm to 63cm if no improvements are made to the current drainage system. However, it is possible to reduce the average water logging depth from 63cm to 47cm with improvement in the drainage system. Furthermore, with climate proofing, the average water logging depth will be reduced from 47 cm to 40 cm (still higher than 33cm). However, given the socio-economic changes assumed under realistic scenario, it is further observed that even with climate proofing, the number of people to be affected will rise from 6% to 13%. In all likelihood, it can be concluded that from the point of view of reducing impact on the people, the climate proofing strategy is not fully effective. Under this condition, it is suggested that effective soft policies are also designed as a part of climate proofing to ensure that the vulnerability is reduced further.

Table 7: Damage Costs and Adaptation Benefits in 1 in 10 Year (A2 and High SLR)

1 in 10 year return period flood Event	**Water logging depth**	**Household**	**Industry**	**Manufacturing**	**Commercial & Others**	**Agriculture**	**Roads**	**Total**
	(cm)	**(Million Taka)**						
Base Case damage	41	5	33	14	564	3	535	1,155
Base Case damage with improvement	33	1	6	3	109	1	165	285
Damage in 2050 Case with climate change	63	48	13,745	30,665	59,548	21	4,651	108,679
Damage in 2050 with Climate change and improvement	47	25	4,964	11,075	21,507	7	1,680	39,259
Damage in 2050 with Climate change, improvement and adaptation measures	40	10	2,157	4,813	9,345	3	730	17,059
Damage in 2030 Case with climate change	49	17	2,300	2,374	13,266	11	1,494	19,461
Damage in 2030 with Climate change and improvement	41	6	623	643	3,593	3	405	5,272
Damage in 2030 with Climate change, improvement and adaptation measures	39	3	435	449	2,506	2	282	3,677
Benefit (reduced damages) of Base improvement	(8)	4	27	11	455	3	371	870
Benefits (reduced damages) of adaptation measures for 2030	(1)	2	188	194	1,087	1	122	1,595
Benefits (reduced damages) of adaptation measures for 2050	(7)	15	2,807	6,263	12,162	4	950	22,201

(Source: ADB 2010)

9. CONCLUSION & RECOMMENDATIONS

The study indicates that the city of Khulna would be vulnerable to climate change, as established by available data and various analyses during the study. Use of sophisticated tools like mathematical and climate modelling are essential for assessing the impact of climate change and effectiveness of adaptation options. Three models have been used in this study – hydrodynamic, urban drainage and salinity models. For developing climate scenarios use has been made of the latest versions of climate models and bias correction techniques.

Appropriate adaptation measures (options) should be selected to combat the impact of climate change, based on a detailed analysis. The socio-economic scenarios, public health issues and urban planning analyses should be carried out to support the selection of the options. These options, should then be evaluated with respect to economic analysis including cost-benefit ratio, social acceptability, resettlement requirement, tariff (water price) in case of the water supply project and drainage tax in case of the urban drainage, land acquisition, overall health impact, environmental aspects such as pollution, etc. Some add-on options have been recommended, which include reduction of leakage in the (pipe) distribution system, improved water demand management, and the possibility of dual reticulation (two-pipe system).

Core adaptation options recommended for the urban drainage are: widen/deepen drainage channels in the problem areas, lay new drains to divert water to new outlets and increase regulator size. The last mentioned option needs sincere cooperation among stakeholder agencies. Add-on options such as solid waste management, awareness and education campaigns, introduction of drainage tax, strict implementation of fines, improved prediction and early warning system for (urban) floods may also be considered.

As Bangladesh in general is vulnerable to the impact of climate change, the study may be replicated to other urban areas of the country.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

QUALITY OF TREATED WATER IN WTP AT KUET CAMPUS

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ABSTRACT

The raw water quality in Bangladesh varies significantly. KUET, situated at the coastal belt of Bangladesh, suffers from the scarcity of sweet water and treatment of saline water is very costly. However, blending of surface water, groundwater, and rainwater from available sources in low-cost treatment process might have promising application for such areas. Keeping this in mind, a water treatment plant (WTP) was established in 2000 to ensure adequate supply of potable water for drinking and domestic purposes. Water for drinking and food preparation needs to be free from pathogenic micro-organisms, minerals and organic substances which have adverse effect on human health. Besides this, water must be aesthetically acceptable. For example, it should be free from turbidity, apparent color, any objectionable taste, and salinity. In this study, the quality of raw water sources and clear water in KUET campus is observed. The physico-chemical water quality and micro-biological water quality was tested and the test results were used to make a seasonal variation relationship. For change of every parameter, some reconditions were given to solve the problems. To minimize the salinity problem a surface water source was suggested. The use of shallow tubewel water as a source is suggested to eliminate.

Keywords: *Quality of treated water, treatment plant, seasonal variation, Physico-chemical quality*

1. INTRODUCTION

Water is a precious commodity (SWTPI 2011) and pure water is the demand of human being. Khulna, situated at the coastal belt of Bangladesh, suffers from salinity problem. To solve this problem, a decision was taken to use surface water, but surface water is polluted by turbidity, color, dissolved solids, and various microbiological elements. To solve these problems, a WTP was established. But lack of monitoring made some problems in the quality of the WTP water.

The problems associated with the WTP may be divided into two main sections, one is physico-chemical water quality problems, and another is the micro-biological (Peavy *et al*., 1995) water quality problems. The main problem of physico-chemical water quality at WTP is the chloride problem. To solve this problem blending of surface water source was suggested. The water of the pond beside the teachers' club was collected to test in the laboratory and was compared with the standard value (Ahmed and Rahman, 2000) of raw water quality and thus the surface water ratio was increased.

The shallow tubwels, another raw water source of this treatment plant was observed throughout projected time and a excessive amount of chloride (AWWA 2005) problem was detected in most of the shallow tubwels as well as failure of layer in summer season.

The micro-biological water quality was also a concern in this study because it's very important for human health (WHO 1984). The total coliform and fecal coliform was tested in laboratory and the test result was used to make seasonal variation. The water from clean water reservoir was collected and tested in the laboratory. From this test, the efficiency of micro-biological water treatment was observed. The process of clorination was observed carefully and some guidelines were suggested to increase the rate of clorination.

The main objective of this study is to investigate the seasonal variation of water quality parameter of WTP, the raw water quality of the pond near the teachers' club and to give some sustainable guidelines to solve the problems associated with the water quality parameters.

2. METHODOLOGY

The work started by studying the map of KUET campus, the layout of WTP for better understanding of the location of water source and supply point of treated water. Then the field investigation was done to get the real picture of the WTP in KUET campus. In the field investigation, a number of operational and maintenance problems were identified in the WTP. A heap of garbage was found at close proximity, which might cause pollution in rainy season. The used sand and sludge were not treated properly. Nevertheless, domestic wastes were found to be disposed on the bank of the source-water pond. As a result, the pond beside the teachers' club has become unusable for high organic matters and bacteriological contamination.

In field investigation, the sampling points were fixed. From the raw water sources and clear water reservoir, the samples were collected. Then the collected samples were tested in laboratory with some necessary parameters. The tests were done in every month of projected time to observe the seasonal variation. The results of the tests were compared with the standard value of drinking water in Bangladesh. The test included the physico-chemical water quality and the micro-biological water quality. Chloride, hardness, turbidity, color were the concern of the physico-chemical water quality and the total and fecal coliform was the concern of the micro-biological water quality parameter.

The test results were analyzed and plotted in graph where seasonal variations were compared with the standard value of Bangladesh. From these results, some discussions were made with respect to each result. The problems of the water quality parameter were identified and the solutions were suggested.

Figure 1 Test in laboratory

Two types of water sources exist in KUET campus from which water is used as raw water in the WTP. The source of surface water is pond water and the source of ground water is deep tubewel and shallow tubewel. Figure 2 shows the basic treatment processes and operations for the removal of the major impurities from pond and tubewel water. Although the water of the protected pond usually does not need further coagulation, flocculation, and sedimentation treatment, this treatment would be required due to seasonal increase of pond water turbidity. Chlorination is done in the clear water reservoir to avoid the micro-biological contamination.

The pond besides the teachers' club was a concern of this study because the blending of surface water is a process of saline-free water. This pond was used in the first design of WTP at very early beginning. But it became unusable after some times. The sample water was collected and tested in laboratory. The BOD and COD result was not satisfactory compared with the standard value of raw water sources. But some repairing can make it usable. The TC & FC can be controlled by changing the unhealthy waste dumping behaviour.

Flow diagram of water sources and treatment process of WTP at KUET campus-

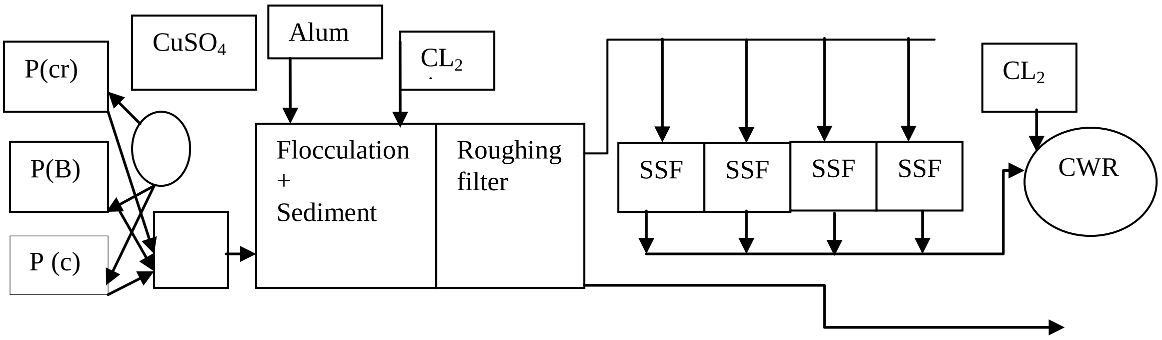

Figure 2 Flow diagram of the water treatment plan at KUET campus

3. RESUTS & DISCUSSION

3.1 Physico-chemical quality

The samples were tested in laboratory and compared with the standard value of Bangladesh. Graphs were plotted according to test results of seasons.

Arsenic: Arsenic is a toxic element which affects human body as a slow poison. It contaminates the shallow tubewels mainly; surface water and deep tubwels are free from this contamination. No arsenic was found in KUET campus throughout this projected time, so there is no need to make a treatment unit to remove arsenic contamination.

Chloride: Chloride is widely distributed in nature, generally in the form of salt. Chloride occurs in natural waters in widely varying concentrations. Upland and mountain water have quite low amount of chlorides, whereas rivers and groundwater usually have a considerable amount. Sea and ocean water represents the residuals resulting from partial evaporation of natural waters that flow into them, and chloride levels are very high. Higher chloride content in inland water usually indicates sewage pollution. As KUET is situated at the coastal belt of Bangladesh, chloride (salinity) is main concern of taste. Source water salinity (Zaman 2009) and clear water salinity is determined and a graphical representation of salinity of clear water is given in Figure 3.

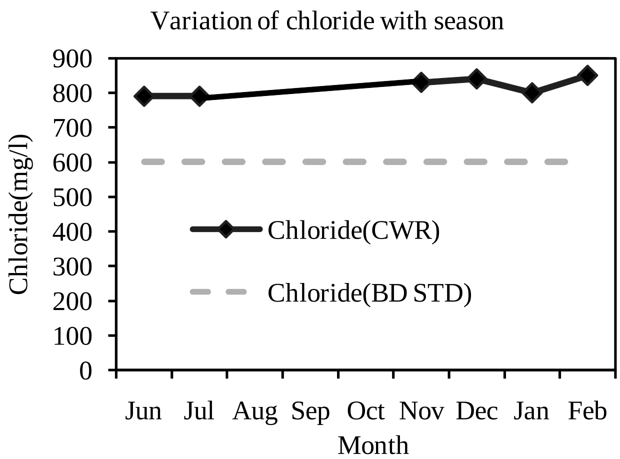

Figure 3 Variation of chloride of CWR of WTP

From the above figure, it is clear that the value of chloride is much higher than the acceptable limit of the standard of Bangladesh and WHO guidelines (600mg/l). During summer, it becomes higher than the rainy season. Therefore, necessary steps should be taken to minimize the salinity of treated water in WTP at KUET campus.

Total dissolved solids: Total dissolved solids comprises of organic salts and small amount of organic matter. The common dissolved mineral salts affect taste, hardness, corrosion, and encrustation. The amount of dissolved solids present in water is an important consideration of its suitability in domestic use. Many dissolved substances are undesirable in water. Dissolved minerals, gasses and organic constituents may produce aesthetically displeasing color, taste and odor. Some chemical may be toxic and some of the dissolved oxygen constituents have been seen to be carcinogenic.The value of total dissolved solid in water treatment plant is varying with season. The result of test is determined and based on test results graph are plotted in Figure 4.

Figure 4 Variation of TDS of CWR with season

From the above figure it is seen that the TDS removal capacity of this WTP is not satisfactory. The TDS value was greater than the standard of Bangladesh (1000mg/l) in all seasons of year.

pH: In water supply pH is important for coagulation, disinfections, water softening and corrosion control. In biological treatment of water, pH is very important. Acceptable limit of pH for drinking water varies from 6.5 to 8.5. (Ahmed & Rahman 2000). pH is a term used universally to express the intensity of the acid or alkaline condition of a solution. pH variations for various stations were determined. On the basis of test results, a graph of pH versus month is plotted as shown in Figure 5.

Figure 5 Variation of pH of water from CWR with time

From the above figure it is viewed that the pH of CWR is absolutely under the standard of Bangladesh and WHO guidelines. So no further unit or repairman is suggested for this WTP.

Color**:** Color results from certain types of dissolved and colloidal organic matters leached from soil or decaying vegetation. Colored water is not aesthetically acceptable to the general public. Highly colored water is unsuitable for laundering, dyeing, papermaking, beverage manufacturing, daily production and other food processing, and textile and plastic production. Aceptable limit of color in the standard of Banladesh and WHO guidlines variation is 15 Pt-Co.Color **v**ariation for different month is given below. On the basis of test results, graph is plotted for color change with season in Figure 6.

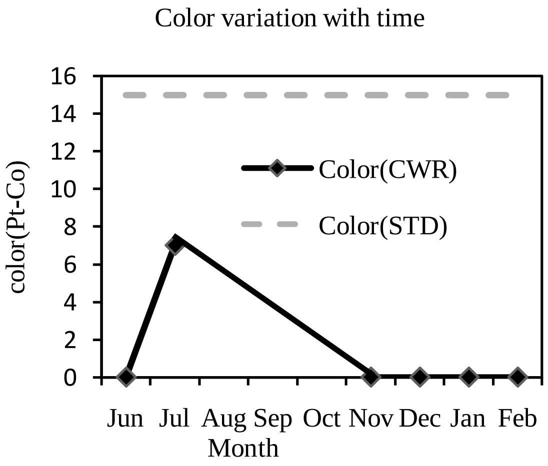

Figure 6 Color variation of CWR with time

From the above figure, it is clear that the Color change of CWR is acceptable compared to the standard of Bangladesh and WHO guidelines. So no repairing is needed for color treatment in WTP of KUET.

Turbidity**:** Turbidity is an expression of certain light scattering and light absorbing properties of water sample and depends in a complex manner on factors such as the number of size, shape and refractive index of particulate present in water. Turbidity occurs in most surface waters due to presence of suspended clay, silt, finely divided organic and inorganic matters, plankton, and microorganisms. Allowable limit of turbidity for drinking is 10 NTU. (Ahmed & Rahman 2000)**.** Turbidity change of CWR with month is also important factor. On the basis of test results, graph is plotted in Figure 7.

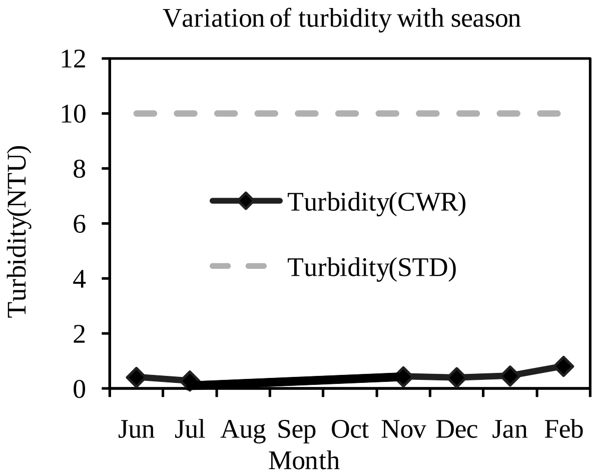

Figure 7 Variation of turbidity of CWR of WTP

From the above figure, it is noticed that the Turbidity is within limit of the standard of Bangladesh and WHO guidelines. So no extra unit is needed to add in WTP at KUET campus.

Hardness: Hard water refers to water that requires considerable amount of soap on latter and that also produces scale in hot pipes, heaters, boilers and other units in which the temperature of water is increased materially. Hardness of CWR also varies with season. On the basis of test results, graph is plotted in Figure 8.

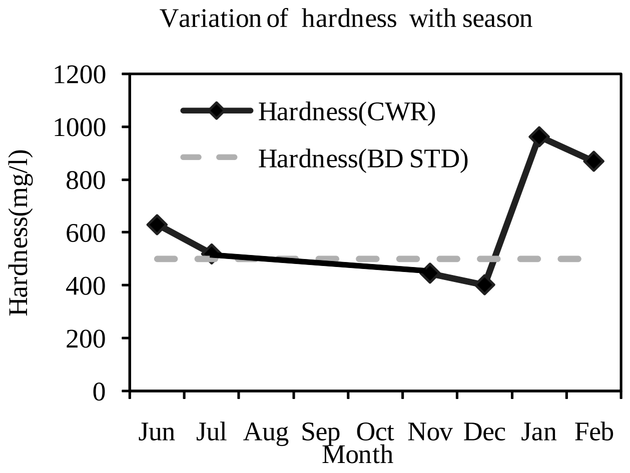

Figure 8 Variation of hardness of CWR of WTP

From the above figure, it is seen that Pond water should be added to minimize hardness of clear water because the Hardness of treated water is exiting the acceptable limit of Bangladesh standard (500mg/l) in dry season. It means the blending of surface water is not in appropriate amount.

3.2 Micro-biological water quality

The main impurities of surface water are bacteria (Wegeline 1916). According to the WHO guidelines and the standard of Bangladesh, there should not be any TC & FC in water. Bacteriological test results for various stations of WTP at different month are given below.
Graphical representation of seasonal variation of TC in raw water:

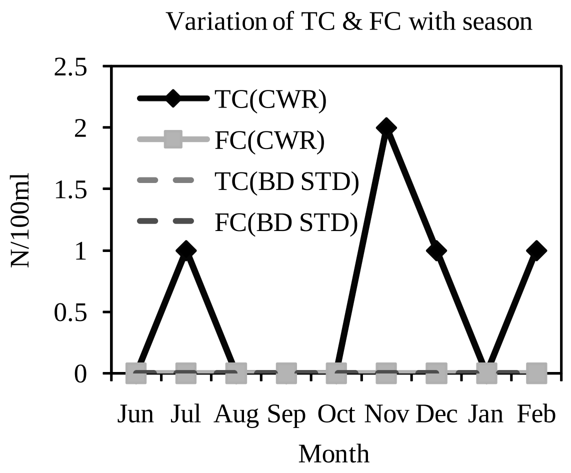

Figure 9 Variation of TC & FC with season

From the micro-biological test, it is clear that some repairing is necessary to avoid TC and FC contamination (IRC 1981). In seasonal variation, it is seen that the TC and FC contamination is higher in rainy season.

3.3 Water quality of pond near the teachers' club

Table 1 Raw water quality of pond besides teachers club (CPCB 2011)

Parameter	Value of the pond	Standard value of raw water	Comments
pH	8.28	6.5-8.5	May be used as a source water in WTP
Turbidity(mg/l)	4.67	50	
Color(Pt-Co)	42	<10	
TDS(mg/l)	129	1000	
Chloride(mg/l)	70	<200	
BOD_5 (ml/l)	3.20	<3	
COD(ml/l)	80	<10	
Hardness(mg/l)	111	1000	

From the above table, it is noticed that the value of BOD and COD is a little bit higher than the acceptable value of raw water because the pond side is used as a domestic waste dumping place. In rainy season, the waste sludge runs into the water and is polluted. But the hardness and salinity is low compared to other sources, so it should be used as a source of raw water after some repairing.

3.3.1 Water quality of shallow tubewel in KUET campus

Table 2 Physico-chemical results of shallow tubewels used as source in WTP

Parameter	Name of the tubewels
Layer failures in summer season	T3,T4,T5,T6,T7,T8,T9,T10
High salinity above BD standard 600mg/l	T1,T4,T6,T7,T8,T12

It might be an economic decision to eliminate tubewel from the WTP because in dry season it cannot supply enough water and most of the tubewels are over saline. So, by eliminating the tubewels, the pumps being used for tubewel, may be used for pond water.

4. CONCLUSION

The objective was to address the inspection of the field survey and to collect samples for laboratory test. From the laboratory test, the prevailing water quality was observed and graphs were plotted for each water quality parameter. From the graphs, it is clearly observed that the raw and clear water quality changed significantly with season. Turbidity was higher in rainy season than summer but salinity was less in rainy season than in summer. Bacterial contamination in CWR also increases in rainy season. By analyzing the individual raw water quality, it was recommended to eliminate the tubwels for the problems of high salinity and layer failure, so a further searching of alternative source of raw water was done. Pond beside the teachers' club was also tested to use as raw water and it was recommended to include in WTP with some repairing.

ACKNOWLEDGEMENT

At first, the authors express gratitude to the almighty Allah that the project has been accomplished perfectly and successfully within fixed time. The authors like to express thanks to Mr. Sardar Saidul Alam, Lab Assistant and Mr. Sayed Ahsan Ali, Assistant Technician officer of Environmental Engineering Laboratory, Department of Civil Engineering, KUET. Mr. Kartiq and all employees of WTP at KUET campus were helpful to fulfil this project.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

A CASE STUDY ON ARSENIC PROBLEM AT HARIARAMPUR VILLAGE, BAGHA, RAJSHAHI

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ABSTRACT

Harirumpur village, Bagha, Rajshahi is a typical Bangladeshi village having majority of people surviving below poverty line has been chosen as the study case. Noticeably, the contamination of arsenic in drinking water of the village has been detected since the time of first identification of this pollutant in drinking water in our country. In the Harirumpur village, the problem of arsenic has been minimized into a considerable limit be introducing the village's new water supply system by community participation which has become a role model for such type of villages which are affected by the pollutant. The community participation for adopting a new water supply system has become very effective since the rate of contamination has been appreciably decreased and there are very few amounts of people in the final stage of contamination. Instead of that, the others who are affected and were in initial or second stage are showing improvement of their health. Even though, the community participation for having a new water supply system was really embraceable but still the sanitation condition of the village is too pore which be the most common problem in every house of the village. The present condition of sanitation has been surveyed which shows really a poor picture. In this study, the present condition of WSS in the Harirumpur village has been studied and a feasible sanitation system for the village people has been suggested.

Keywords: *Arsenic contamination, drinking water, health, poverty line*.

1. INTRODUCTION

The Arsenic poisoning, termed the biggest environmental disaster and a major public health issue in recent times in Bangladesh (Mahmood et al, 2004). It is estimated that 35-57 million of the approximately 138 million people of Bangladesh are at risk of drinking arsenic contaminated water (BGS and DPHE, 2001). During the last 6-7 years many arsenic mitigation technologies have been developed in Bangladesh. Arsenic can be removed or reduced by many method e.g. Co-precipitation, Absorption, Lime treatment, Ion exchange, Membrane technique etc. DPHE constructed ARP (Arsenic Removal Plant) in many water supply system. In some area IRP (Iron Removal Plant) have been converted to ARP (Arsenic Removal Plant). The detailed dosing and chemical use depends upon the quality of ground water of that area.

However, arsenic was first detected in tube well water (at Barogharia union of Chapai Nawabganj district) by the Department of Public Health Engineering (DPHE) (Ahmed, 2000). The greatest concentration of affected wells is in the south and east of Bangladesh, and in the 12 worst affected districts (Chandpur, Faridpur, Munshiganj, Gopalganj, Lakshmipur, Noakhali, Bagerhat, Shariatpur, Comilla, Faridpur, Shatkhira and Meherpur). Roughly 60% or more of the wells in each of the worst-affected districts contain arsenic concentrations exceeding the Bangladesh standard and occasionally exceeded 1000μg/l (Kinniburgh and Kosmus, 2002). An estimated 50 million people in Bangladesh are drinking water with arsenic concentrations exceeding the current Bangladesh standard (Mondal et al., 2006).

On the basis of simulating data of Samta village of Jessore district, Curry et al. (2000) predicted that after 30 years, more than 22% of the villagers were likely to be attacked by arsenicosis and 5.5% of them were likely to die, if the inhabitants continued to drink from the same tube wells. Moreover, the DPHE of Bangladesh indicated that about 75 million people belonging to 59 districts out of the country's 64 districts are estimated to be clinically and sub- clinically arsenicosis victims (Jakariya, 2000). Nasreen (2002) also reports the onset of social problems such as social instability, superstition, ostracism, marital problems, and discrimination against women, increased poverty, diminished working ability, and death. People are reluctant to develop marital relationships with families whose members suffer from arsenicosis. This has caused serious anxiety for parents of unmarried adult children. Children of arsenic patients are not allowed to attend social or religious functions or even school (Hassan et al., 2005). Therefore, the study has investigated about the arsenic problem at Hariarampur village of Rajshahi district.

2. METHODOLOGY

2.1 Location

Harirampur is one of the poorest villages under Bagha Upazila, Rajshahi. The main problem of this village is scarcity of pure drinking water. Here the ground water is greatly contaminated with Arsenic. After drinking this water peoples are affected by arsenicosis. So the supply of pure drinking water becomes the basic needs for the peoples of this village. To mitigate this problem a non government organization named SAMATA NARI KALLAYAN SANSTHA come forward. They made a water supply plant to supply the pure drinking water to the villagers.

2.2 Data Collection

Oral interview has been performed among the peoples of Harirampur village, Bagha about their satisfaction level of present water supply and its cost, arsenic problem, their concept about the disease and their understanding about the community participation. The data collected from the people especially from women has been analyzed and a brief overview of the condition of Harirampur village has been presented.

2.3 Samata Nari Kallayan Sangstha

Samata Nari Kallayan Sangstha is one of the non-governmental, non-profitable and non-political organization. They started a project “SDC-WSP Rural Piped Water Supply Pilot Scheme” in 2000. At first 140 houses were under this scheme. This system is controlled by Village Development Committee (VDC). At the beginning stage of this organization, its working areas were limited within the Bagha pail but due to the implement the program successfully its working areas were expanded with the financial and technical assistance of govt. of Bangladesh and foreign NGOs. At present the organization is working in 12 Upazilas under 5 districts.

2.4 Construction of water supply plant by SNKS

The water supply tank which was constructed by the SNKS in HARIRAMPUR village is a successful project. It supplies water to the villagers since 2001. Though it has some limitation but it serves a tremendous role to the people who are affected by Arsenic. This project becomes successful for the community participation. Peoples of this village gave the land for this project and contribute the development of this project. Without community participation it cannot be done successfully. The design capacity of the water supply plant is 400 houses but SNKS has started their project with the 140 house but a present connected house is only 65 nos. However, the Figure 1 shows the water supply plant by SNKS at Hariarampur village.

Figure 1: Water Supply Plant

3. RESULTS AND DISCUSSION

3.1 Use of Technology

The peoples of the village of Hariarampur, Bagha at Rajshahi are not used the technology for safe water supply. Approximately, 83.75% respondents are not used while 16.25% are used technology for getting safe drinking water (Figure 3). Moreover, the Figure 2 shows the supply system of this technology from removal plant to household. Sometimes beneficial are not getting the water supply due to arise various problem for distributing and consumption of water from tank like as electricity problem, cost of water and alternative uses.

Figure 2: Water supply to household

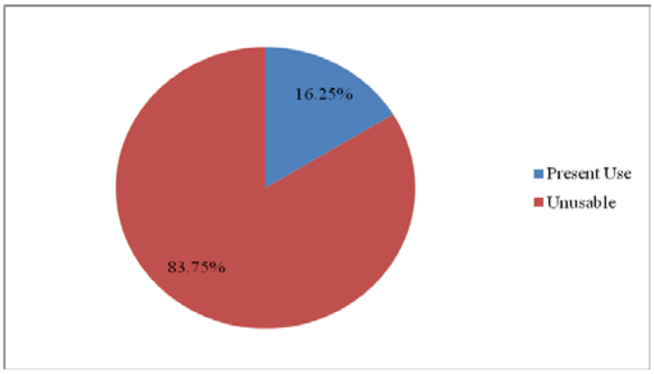

Figure 3: Percentage use of water supply technology

3.2 Satisfy of Water Supply

The peoples of the Hariarampur village are almost satisfied for the proper water supply. Approximately, 85% respondents are satisfied while 15% are not satisfied for the availability of water source in that village (Figure 4).

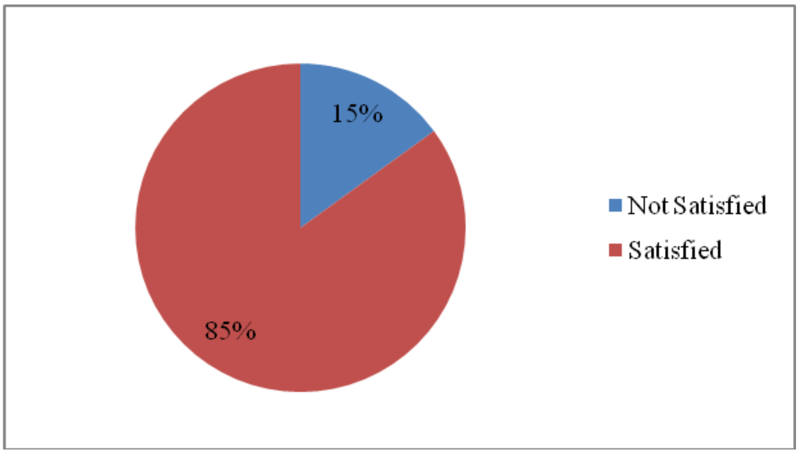

Figure 4: Satisfaction of water supply at Hariarampur Village

3.3 Cost of Water Supply

The peoples of the Hariarampur village are not satisfied for the cost of water supply. SNKS has taken money for the supply of water from the beneficial. Approximately, 40% respondents are not satisfied while 60% are satisfied for the availability of water supply from tank to their house (Figure 5).

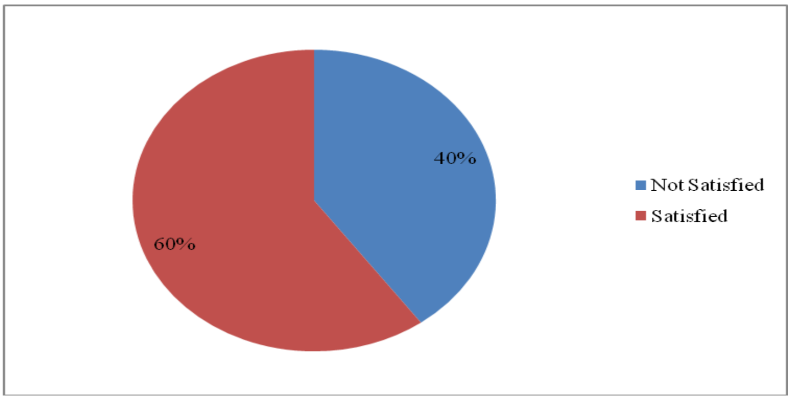

Figure 5: Satisfaction of cost of water supply

3.4 Arsenic Contamination People

The peoples of the Hariarampur village at Rajshahi district are attacked by the arsenic contamination and study shows that people are back from it. Approximately, 82% of peoples are facing problem by arsenic in before project while only 18% of peoples are facing arsenic problem in after the project (Figure 6).

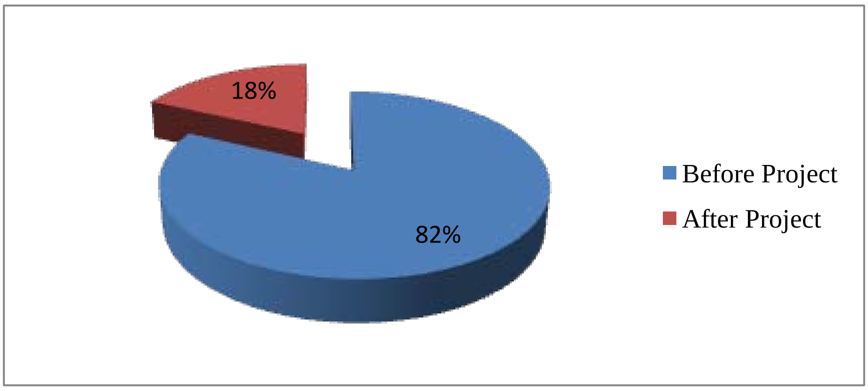

Figure 6: Arsenic contaminated people at Hariarampur village

3.5 Awareness about Arsenic Disease

Awareness about arsenic disease of the Hariarampur village is in the worst condition. Therefore, arsenicosis diseases occur in that area. However, in case of analyzing awareness about arsenic disease the study shows that 15% people are not aware while 85%people are aware the disease by arsenic. So it is observed that the most of the people are aware about the arsenic disease (Figure 7).

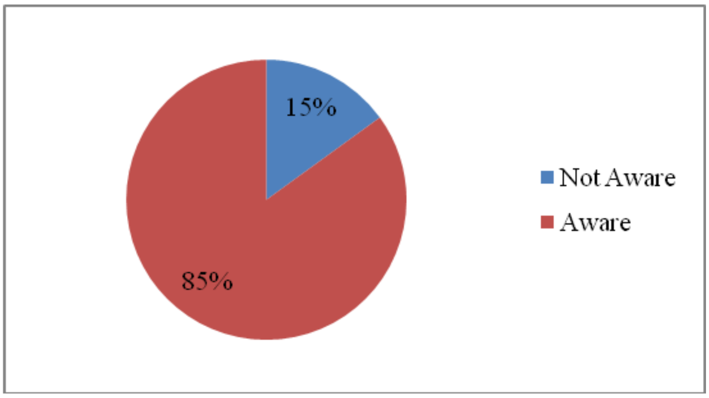

Figure 7: Awareness about arsenic disease at Hariarampur village

3.6 Awareness about Community Participation

Figure 8 shows the awareness about community participation. Approximately, 65% of peoples are aware while 35% of peoples are not aware about the arsenic by the community participation (Figure 6).

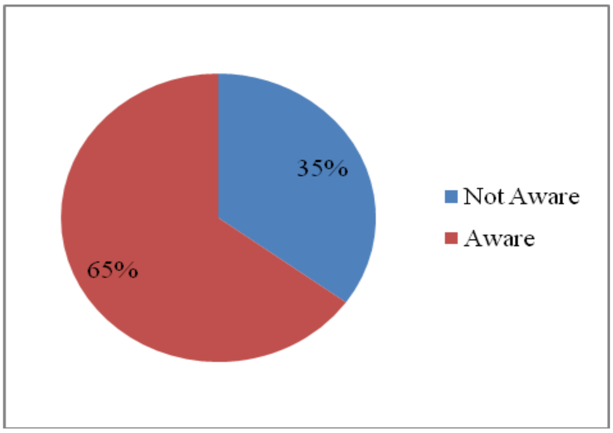

Figure 8: Awareness about community participation

4. CONCLUSIONS

The peoples of the Hariarampur village are almost satisfied for the proper water supply. Approximately, 85% respondents are satisfied while 15% are not satisfied for the availability of water source in that village whatever 83.75% respondents are not used while 16.25% are used technology for getting safe drinking water. Moreover, Awareness about arsenic disease of the Hariarampur village is in the worst condition whatever 82% of peoples are facing problem by arsenic in before project while only 18% of peoples are facing arsenic problem in after the project. Finally, it is clear that the technologies introduced to supply Arsenic free safe drinking water are only short term emergency solutions for areas severely affected by Arsenic contamination. Coordination is needed among the different stakeholders not only to avoid repetition of activities but also to find mitigation packages that are acceptable to the community. The government must cooperate with academic as well as research institutes to assess the causes and impact of arsenic poisoning and take up remedial measures whenever and wherever necessary.

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COMPLETE DECOLOURIZATION OF REACTIVE DYE BY COMBINATION OF COAGULATION-ADSORPTION-MEMBRANE FILTRATION: A HYBRID STUDY

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ABSTRACT

Increasing attention is being paid to health and environmental risk as a result of the presence of dye contaminants in the effluent discharged textile plants. Even the presence of very low concentrations of dyes in the effluent is highly visible and is considered aesthetically undesirable. It must be removed from wastewater completely. As there is no single process capable of adequate treatment mainly due to the complex nature of textile dye effluents, the present paper focuses on assessment of removal of these dye contaminants using a hybrid process with combined coagulation–adsorption–membrane treatment system to overcome the disadvantage of individual unit processes. The experiments showed that the combination of alum coagulation, PAC adsorption and microfiltration could be effective for removal of reactive dyes from water. The used adsorbent was PAC and the membrane was prepared in our lab. The dye removal efficiency by alum coagulation was highly dependent on dye concentration and solution pH. The optimum pH and dose of alum was 4.5 and 0.200 gm. /L respectively. The dye removal efficiency in the membrane equipped batch continuously stirred tank reactor was also analyzed. The data was treated with Pseudo 1st & 2nd order model and was fitted quite well in 2nd order model.

Keywords: *Reactive dye; Adsorption; Coagulation; Hybrid process; microfiltration; Alum.*

1. INTRODUCTION

The introduction of waste products in the environment is a world wide problem that has been highlighted by various environmentalist groups. Ever increasing industrial activity is the main culprit behind these environmental pollution problems and ecosystem damage, coming from the accumulation of pollutants such as toxic synthetic dyes. Synthetic reactive dyes being the major part of this pollutant have been extensively used in many industries such as textile, leather tanning, paper production food technology, photo electrochemical cells, hair colorings etc. (Robinson et al.2003, Forgacs et al.2003, Kim et al.2003, Nakagawa et al.2003, Papic et al.2003) . The dyes are invariably left as the major waste in these industries. Due to their chemical structures, dyes are resistant to fading on exposure to light, water and many chemicals and, therefore, are difficult to be decolourised once released into the aquatic environment (Sharma et al,2008). Many of the organic dyes are hazardous and may affect aquatic life and even the food chain (sheikh et al, 2009). Release of these dyes in water stream is aesthetically undesirable and has serious environmental impact. Due to intense color they reduce sunlight transmission into water hence affecting aquatic plants, which ultimately disturb aquatic ecosystem; in addition they are toxic to humans also. However, these reactive dyes are the most problematic compared to other forms of dyes and removal of these dyes from wastewater is a major environmental challenge. Hence, there is a constant need to have an effective process that can efficiently remove these dyes. Also, tightening government legislation is enforcing to treat textile wastewater to an increasingly high standard.

Methods for treating textile dye wastewaters consist of various chemical, physical and biological processes. These include: adsorption (Bilal et al., 2004), nanofiltration (Chakraborty et al., 2003; Capara et al., 2007), colloidal gas aphrons (Roy et al., 1992), ultrasonic decomposition (Jiantuan et al., 2003), electro coagulation (Alinsafi et al., 2004), coagulation and precipitation (Liu et al., 2003), advanced chemical oxidation (Arslan et al., 2000), electrochemical oxidation (Torres et al. 2010), photo-oxidation (Patricia et al., 2003), predispersed solvent extraction(Mahmoud et al., 2007), ozonation(Mohammed et al., 2007), supported liquid membrane(Mahmoud et al., 2007) and liquid-liquid extraction (Venkateswaran et al.,2005) .But almost all the above described processes can not meet the required water quality demand with individual disadvantages. This leads the requirement of a combined process to overcome the disadvantages of individual unit operations and enhance the overall treatment performance .However, there is only limited research on the coagulation of reactive dyes and the combined treatment process of adsorption, membrane filtration and coagulation.

In this paper, a combined process is applied consisting of alum coagulation, carbon adsorption and membrane filtration for complete decolorization of synthetic reactive dye (Reactive Red 6BL) from model wastewater. The whole procedure was carried out at laboratory scale with PAC as adsorbent and membrane was prepared from nylon-6 pellet. Here, an additional treatment stage using membrane at the final step is provided to obtain reusable quality treated water with retaining the adsorbents in the CSTR so as to obtain the maximum utilization of adsorbent, although complete destruction of dye is achieved by coagulation–adsorption system. The use of low-pressure membranes processes, as microfiltration (MF) have better priority in this respect compared to reverse osmosis and Nano-filtration, which demands considerably high operational cost because of high-energy requirement. The effects of dosage of alum (*Aluminium sulphate*) and pH on dye removal on coagulation, the PAC adsorption isotherm were analysed. Also the pseudo 1st and 2nd order model has been used to assess the mechanisms of mass transfer of dye adsorption to PAC in MF-PAC stirred cell.

2. EXPERIMENTAL MATERIALS AND METHODS

2.1 Experimental materials

Preparation of Reactive Red 6BL solutions: All chemicals used in this study were of analytical-grade. Reactive Red 6BL, a reactive dye used as the model adsorbate in the present study. The dye used was of analytical grade so it was used without further purification. A stock solution of 1000 mg L-1was prepared by dissolving an appropriate quantity of Reactive Red 6BL in a liter of deionized water. The working solutions were prepared by diluting the stock solution with deionized water to give the appropriate concentration of the working solutions.

Adsorbent: Charcoal was collected locally and powdered by a blender.

Monomer for microfiltration membrane (MF): The monomer was Nylon-6, pellets were of analytical grade. Structure is given in Figure: 1.

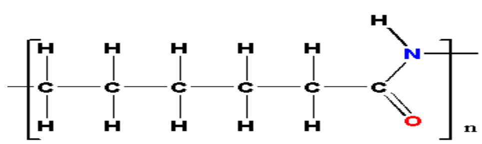

Figure 1: Nylon 6 monomer structure.

Alum: A commercial grade Alum (*Aluminium sulphate*) Collected from local market.

H_2SO_4: MERK,Germany

NaOH: MERK,Germany

2.2 Experimental method

2.2.1 Preparation of membrane:

The membranes were prepared by wet phase inversion method from the monomer Nylon-6 pellets. The procedure consists of weighting of 15 gm. of Nylon-6 pellets in balance followed by adding it with 100mL of Formic acid and stirred uniformly in magnetic stirrer for 24 hours at room temperature. Then stretched uniformly over a flat glass plate by glass rod and kept for 10 minute followed by immerging in distilled water for completion of phase inversion. Then it was stored in distilled water for further use. For each run a new membrane was used. The physical and chemical properties of membrane are listed in Table 1.

Table 1 : Characteristics of prepared membrane.

Monomer	Nylon-6
Type	Microfiltration
Preparation method	Wet-Dry phase inversion technique
Permeability(with respect to 100 ppm iron solution)	3×10^{-5} (m^3/m^2.min.psi)
Module type	Plate & Frame
Surface area	19.635 cm^2
Thickness	0.2mm
Solvent for membrane preparation	Formic acid

2.2.2 Construction of batch CSTR:

A poly acrylic jar with a capacity of 0.600 L is used as the reactor vessel. It was equipped with a 12V DC motor is used as the rotator. A DC-12V (350mA) adaptor with voltage (so speed) controller was used to run the motor. An agitator was attached with the DC-12V motor with the axis of rotation of the motor, axis of reactor and the agitator is aligned in the same line. The agitator is used to mix the supernatant solution inside the CSTR thoroughly with PAC and to maintain the uniform concentration within the CSTR. A circular membrane piece was attached with the support distributor plate (made of Nylon-6) with epoxy resin. A pipe attached with the permeate side of Membrane module was used as the outflow pipe. The outflow was connected with a pump as shown to collect samples for analysis and recirculation of permeate into the reactor. The CSTR volume V can be considered to be constant by the action of pump & recirculation pipe. The residual dye solution of coagulation unit (supernatant) is fed to the CSTR through this hole continuously.

2.2.3 Coagulation Experiments:

Coagulation studies were conducted in duplicate using Jar-test Apparatus with A 500ml. dye solutions in six beakers with different doses of Alum solutions was added. The samples were stirred for three minute at 250-300 rpm followed by 60 minutes slow mixing of 50 to 70 rpm followed by settling for one hour. Then the supernatant is with drawn, filtered and was used for dye removal analysis using UV spectrophotometer (Shimadzu UV-1650 PC, China) at 541 nm after centrifuging for 3 minutes .All the experiments were conducted at room temperature of 27 ± 3^0C.

Percentage of dye removal was calculated by Eq. (1).

$$percent\ removal\,(\%) = \frac{C_r - C_t}{C_t} \times 100 \text{……..} \quad (1)$$

Where, C_r and C_t are the concentration of the dye in raw and treated solutions, respectively.

2.2.4 Adsorption studies:

Flask adsorption experiments were performed to assess the removal capacity of powdered activated carbon for Reactive Red 6BL. In these experiments, 200 mL background solutions with various concentrations was taken in flasks and treated with fixed amounts of PAC .The adsorption experiments were conducted in a flask shaker under conditions of 450 rpm oscillation speed and 25 ^{0}C for 24 hours enabling equilibrium to be reached. The results of adsorption experiments were used to determine equilibrium isotherms for dye. The adsorption capacity (q) of PAC was determined from the following expression:

$$q = \frac{(C_o - C)V}{W} \text{……………………..} \quad (2)$$

Where, C_0 and C are the initial and equilibrium (or residual) liquid-phase dye concentrations (kg/m^3), respectively, V is the volume of solution (m^3), and W is the weight of dry activated carbon (kg).

2.2.5 Coagulation–adsorption–membrane hybrid process :

The schematic of the combined coagulation–adsorption–membrane hybrid system is shown in Fig. 2. Coagulation and sedimentation pretreatment was carried out prior to adsorption–membrane operation. Adsorption and membrane separation were carried out in a single tank .The filtration has been carried out in constant flux mode with the permeate drawn through the membrane by the pump and recirculated to the open stirred cell. The cell has an effective volume of 600mL and a certain amount of PAC was directly added into the cell during the experiments. The samples were taken from the permeate side at a certain time intervals as required. The membrane used in the filtration experiments was a submerged-type plate-frame MF prepared (in laboratory) as described in section: 2.2.1and a new membrane was used for each experiment. The physical and chemical properties of the membrane are given in Table 1. The overall efficiency of the hybrid system was determined based on the difference in dye concentration (based on UV absorbance) between the initial and final treated effluent. lask adsorption experiments were performed to assess the removal capacity of powdered activated carbon for Reactive Red 6BL.

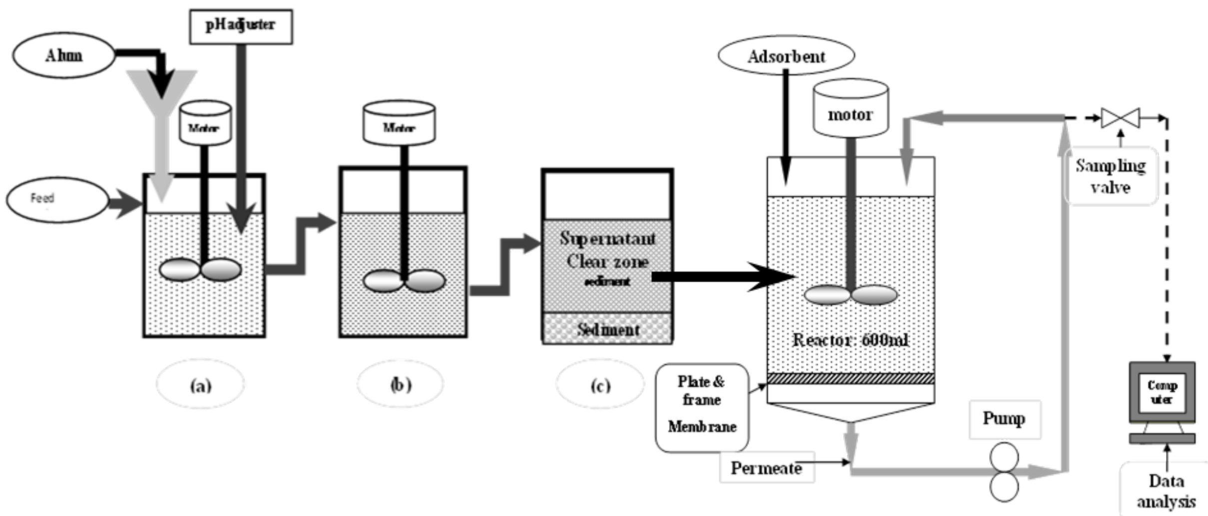

Figure 2: Experimental set-up

(1=motor, 2=pH adjuster, 3=mixer, 4=rotor, 5=plate & frame membrane module, 6=sampling valve, 7=permeate**).**

3. THEORY

3.1 Adsorption Isotherm models

The isotherm models of Langmuir (Langmuir et al.) and Freundlich (Freundlich et al.) were fitted to describe the equilibrium adsorption.

$$\frac{1}{q_e} = \frac{1}{q_\infty} + \frac{1}{q_\infty K C_e} \qquad (3)$$

Where, q_e is the amount of dye adsorbed per unit mass of adsorbent (mg/g) and C_e is the concentration of the dye solution at equilibrium (mg/L). q_∞ and K are Langmuir constants related to the capacity and energy of sorption respectively. The values q_∞ and K shown in Table 1 were determined from slopes and intercepts of the plot of $1 / q_e$ Vs $1/C_e$ (Fig.6)

The linear form of the Freundlich equation is as follows:

$$Log(q_e) = Log K + \frac{1}{n}.Log C_e \qquad (4)$$

Where, log K is a measure of the adsorption capacity and n is an indicator of adsorption intensity The Freundlich coefficient, 1/n, which should have values in the range of 0 to 1 for favorable adsorption. A plot of log qe Vs log Ce gives a slope of 1/n and intercept of log K shown in Fig. 7.

3.2 Adsorption Kinetics

3.2.1 Pseudo 1st order Model

Lagergren's rate equation is most widely used (Lagergren et al.) for the adsorption of adsorbate from a solution.
The first order Lagergren's rate equation expressed as follows:
Where q_e and q_t (both in mg/g) are the solute amounts adsorbed per unit mass of adsorbent at equilibrium and at time t (min), respectively, K_1 (min^{-1}) is the first-order rate constant. The values of K_1 and q_e were calculated from the slope and the intercept of the plots $\ln(q_e - q_t)$ versus t, respectively for different operating condition.Calculated values of K_1 and q_e are summarized in Table 2.

3.2.2 Pseudo 2nd Order Model

Pseudo second-order equation based on equilibrium adsorption is expressed as:

$$\frac{t}{q_t} = \frac{1}{K_2 \cdot q_e^2} + \frac{t}{q_e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(5)}$$

Where, K2 (g/mg min) is the rates constant of second-order adsorption. If second-order kinetics is applicable, the plot of t/q versus t should show a linear relationship. qe and K_2 can be determined from the slope and intercept of the plot.

4. RESULT AND DISCUSSION

4.1 Coagulation Study

4.1.1 Effect of Alum Dose

To study the effect of coagulant dosage on dye removal efficiency, different amounts of alum were dosed into the dye-containing solutions. Dye concentration was kept constant at 100 mg/l. The variations of the dye removal with coagulants dosage are shown in Figure: 3.According to the results, with the increase of the coagulants dosage, the removal efficiencies increased at first (from 0 to 200 mg/L). For the dosages more than 400 mg/L, the efficiency decreased rapidly which was probably due to re-stabilization phenomenon.

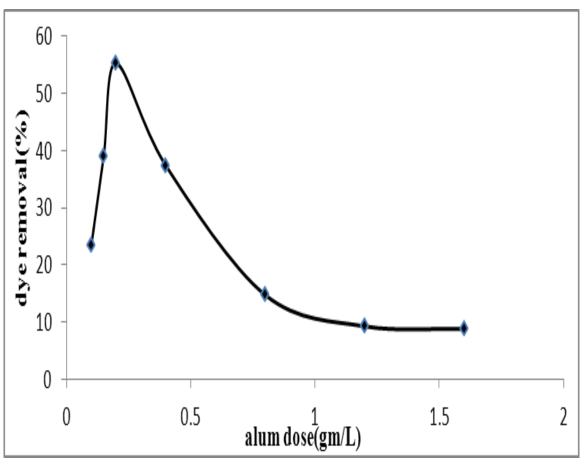

Figure 3: *Effect of Alum dose on dye removal.*

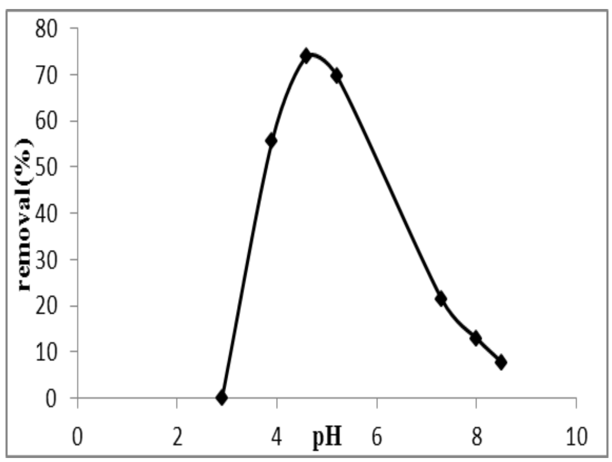

Figure 4: *effect of pH on dye removal.*

4.1.2 Effect of pH on the removal

pH plays an important role in coagulation/flocculation process using inorganic coagulants. Charge on hydrolysis products and precipitation of metal hydroxides are both controlled by pH variations. Thus, pH must be controlled to establish optimum conditions for coagulation (Li and Gregory, 1991).
To study the effect of pH on Reactive Red 6BL dye removal efficiency, dosages of Alum and dye concentration were kept constant at 0.200mg/L and 100 mg/L respectively, while varying pH of the samples using H2SO4 and NaOH. Dye concentration was 100 mg/l for all solutions during the experiment. As shown in Fig. 4, removal of Reactive Red 6BL dye was absolutely dependent on the pH variations. The optimum pH, at which the maximum removal occurred, was about 4.5 for Alum.

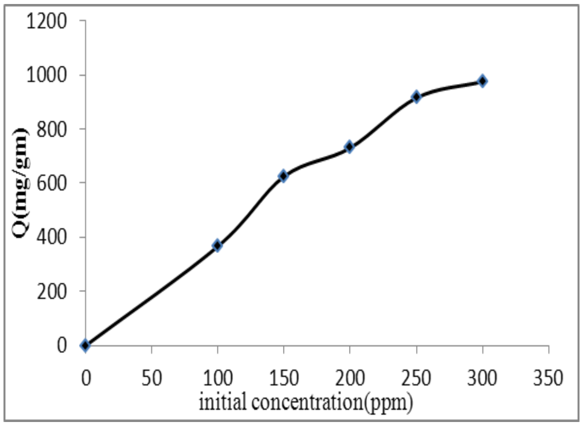

Figure 5 : effect of initial dye concentration removal

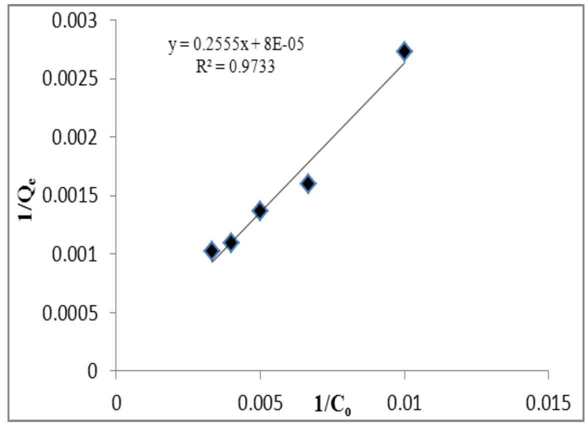

Figure 6 : Langmuir sorption model for Alum coagulation(C_o ane Q_e are initial concentration and dye remova per unit alum at equilibrium respectively).

4.1.3 Effect of initial dye concentration

This step was performed to determine the influence of initial dye concentration on dye removal efficiency, using a optimum condition at room temp. In addition, the variations of the amount of the removed dye per unit mass of coagulant (Q), versus the initial dye concentration are presented. The results are illustrated in Fig. 5.
For alum, with the increase of initial dye concentration from 100 mg/L to 300 mg/L.The highest value of Q was 0.970 mg dye/ mg alum for the dye concentration of 300 mg/L. Here, the curves relatively showed similar changing trends.
The data were treated with Langmuir's sorption model in figure :4 for the approximation of the measurement points in the case of Reactive Red 6BL.

$$q_e = \frac{Q_\alpha . K \; . C_0}{1 + K \; . C_0} \text{..................} \quad (6)$$

Linearization of this equation gives:

$$\frac{1}{q_e} = \frac{1}{q_\alpha} + \frac{1}{q_\alpha . K} . \frac{1}{C_0} \text{.........} \quad (7)$$

Where:
q_e, K and q_α are the dye removed per unit weight of Alum (mg/mg Alum),the maximum mass of dye removed per unit weight of Alum (mg/mg Alum) and constant in Langmuir's equation (dm3/mg) respectively. The Langmuir sorption model (eq. 8) is plotted in Figre.6 and the parameters are found as: q_e = 12.5 gm/gm alum and K= 0.313 m^3/gm .

4.1.4 Effect of initial dye concentration on COD

The removal of Chemical Oxygen Demand with various initial dye concentrations is shown in above figure.:7 showing up to 87.33% COD removal.

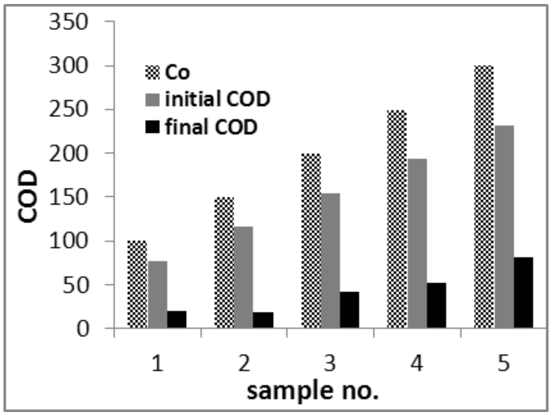

Figure 7 : Effect of initial dye concentration on COD

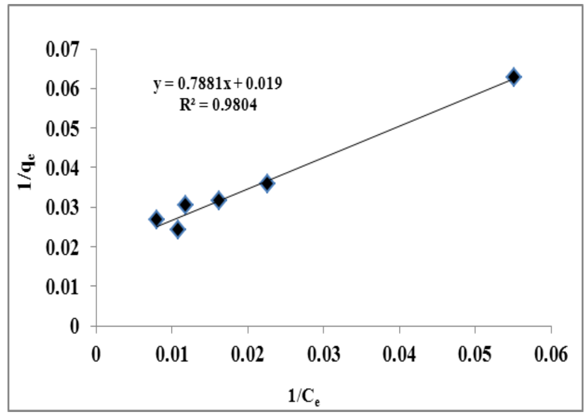

Figure 8 : Langmuir isotherm model(C_e and q_e are batch equilibrium concentration and equlibrium dye uptake respectively)

4.2 Adsorption study

4.2.1 Batch Adsorption study

Adsorption isotherms are the most important information for analyzing and designing an adsorption process (Lee et al.) The adsorption capacity depends on the properties of adsorbate and adsorbent. The plot of $1/q_e$ versus $1/C_e$ is plotted in figure: 8 and from it the Langmuir parameters are determined as: 0.05263158 (kg/kg) and 24.1124 (m3/kg) respectively for q_α and K respectively also the Freundlich parameters (by figure 7) are K=1.797 and n =5.10.

4.3 Combined process study

The membrane separation of dye (by a binary mixer of dye with water) gave a very poor dye removal (after 3 hours of operation) with severe fouling and damaging of membrane.The advantages of combined process is much greater then any of the analysed individual process.

4.3.1 Effect of Contact Time and Initial Dye Concentrations

During the process of secondary removal of Reactive Red 6BL dye after coagulation in PAC-MF coupling unit, the effect of contact time on dye removal was studied and was found to be an important parameter. At the higher PAC doses the removal of dye was faster in initial stages and higher as shown in Figure ;9,10 and 11 due to large surface area available of adsorbent. After this adsorption, the rate of dye uptake is controlled by the rate of dye transported from the exterior to the interior sites of the adsorbent particles, and hence in decreasing pattern and became constant after equilibrium was achieved as shown in these figures. The removals of dye are also presented in form of ratio of present concentration to initial concentrations(C/C_o). It is clear that % of it decreased from 99.99% to 75.48% decreasing contact time from 120 to 30 min. for using 5g GAC in various supernatant concentrations feed to this unit. The % removal was83% to 55.6 by decreasing contact time from 120 to 30min. for 10mg/l using 2.5g GAC and from 63% to27% for 50 mg/l. It was found that the % removal increased with decreasing the initial dye concentration using different adsorbent doses.

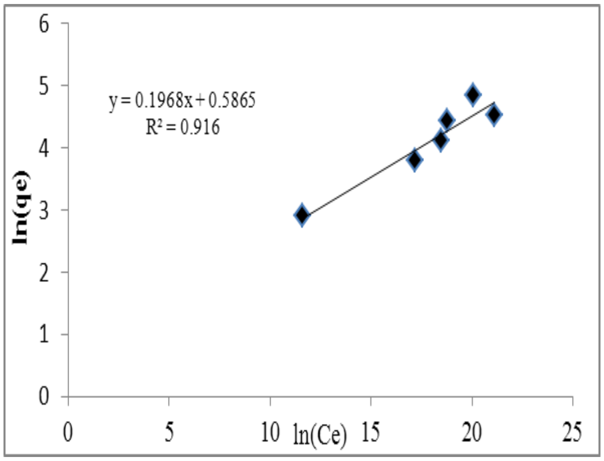

Figure 9 : Freundlich isotherm model(Ce and qe are batch equilibrium concentration and equlibrium dye uptake respectively)

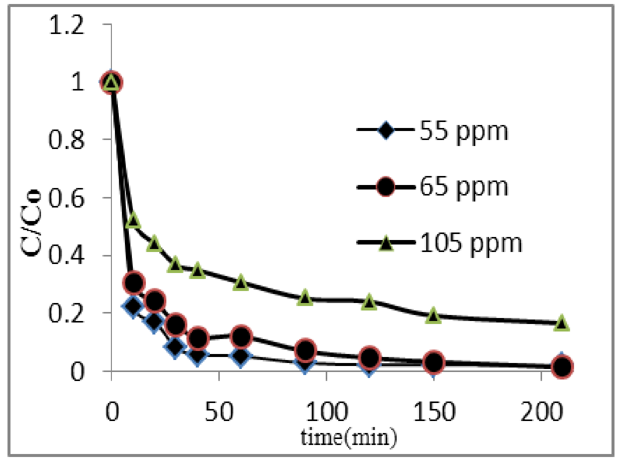

Figure 10 : dye removal with different initial concentration with PAC dose 5gm/600mL. (pH 7.8)

4.3.2 *Effect of Carbon Dosage on Adsorption Process*

Adsorbent dosage plays a vital role for such process because this factor determines the capacity of an adsorbent for a given initial concentration of the adsorbate. The effect of adsorbent dosage was studied at three different doses as shown in the Figure : 10,11 and 12 on reactive red 6BL dye removal. The result showed that as the adsorbent concentration increased, the amount adsorbed per unit mass of the adsorbent decreased (q_e) and also the % of color removal increased. Considerably as shown here,at the equilibrium for different doses of adsorbent with same dye concentrations, maximum removal was found to be 99.99%, 86.90% using 5gm., 3gm. GAC respectively.

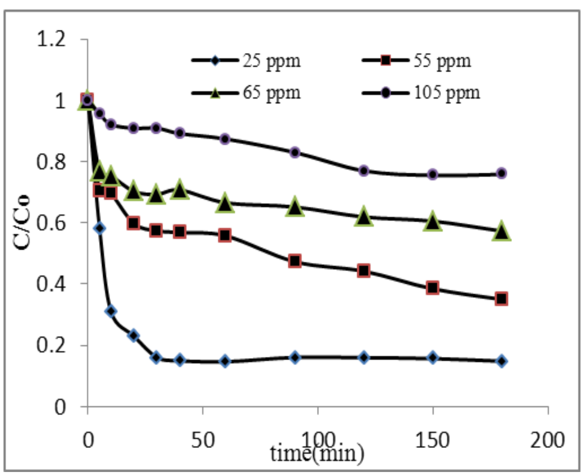

Figure 11 :dye removal with different initial concentration with PAC dose 1.5gm/600mL (pH 7.8)

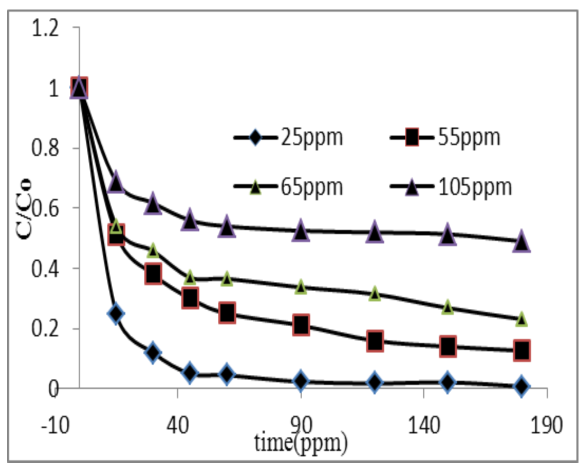

Figure 12 : dye removal with different initial concentration with PAC dose 3gm/600mL (pH 7.8)

4.4 Effect of pH on dye removal in MF-PAC system

Effect of pH on performance of the system was analyzed at pH 4 & pH 7.8.The results are plotted bellow. Here it is clear that for present system lower pH favors very rapid initial adsorption rate and also early reaching of equilibrium.

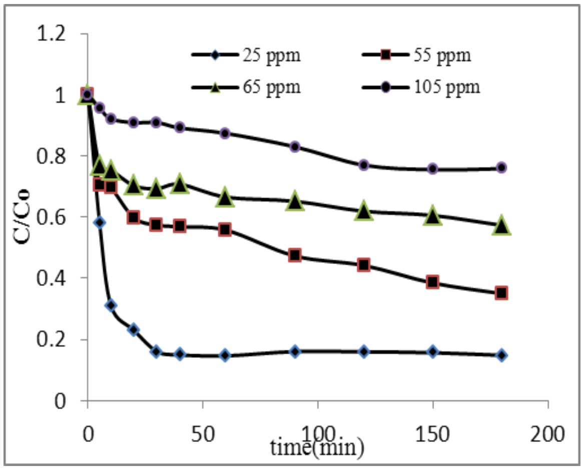

Figure 13 : dye removal with different initial concentration with PAC dose 1.5gm/600mL at pH: 7.8.

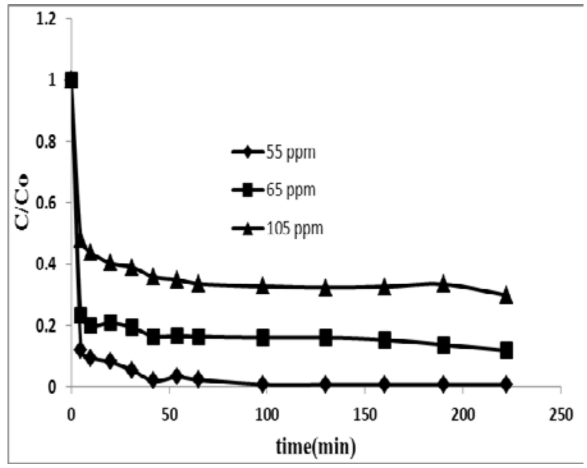

Figure 14 : *dye removal with different initial concentration with PAC dose 1.5gm/600mL at pH:4.0*

4.5 Adsorption kinetics study result

The kinetics are carried out by analyzing permeate sample concentrations of the CSTR unit. The linear plots of t/qt versus t show a good agreement between experimental and calculated qe values are presented in Table 1. The correlation coefficients for the second order kinetic model are higher. The data's are plotted in various plots as shown bellow:

4.5.1 Pseudo 2nd order plot for MF-PAC unit (with different PAC loading, initial dye concentration an of different operating pH)

The pseudo 2nd order equation was applied for all kinetics data obtained from MF-PAC system. The obtained plots for various suparnatant concentration ,adsorbent dose are shown in figure 15-17. The obtained plots for different operating pH are shown in Figure 18 & 20.

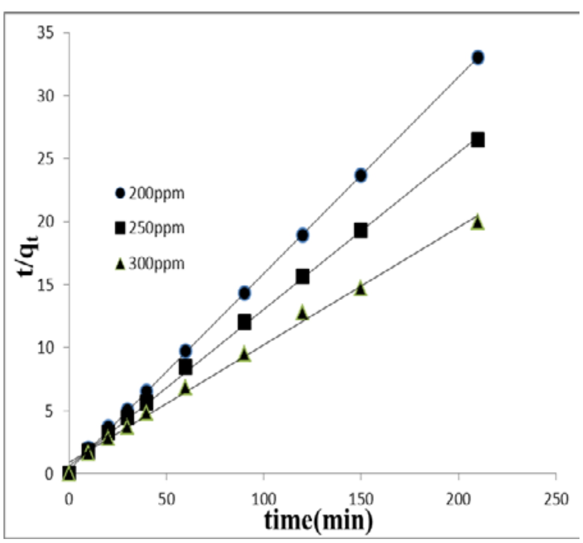

Figure 15 : pseudo 2nd order model for PAC dose 5 gm/600L at pH: 7.8

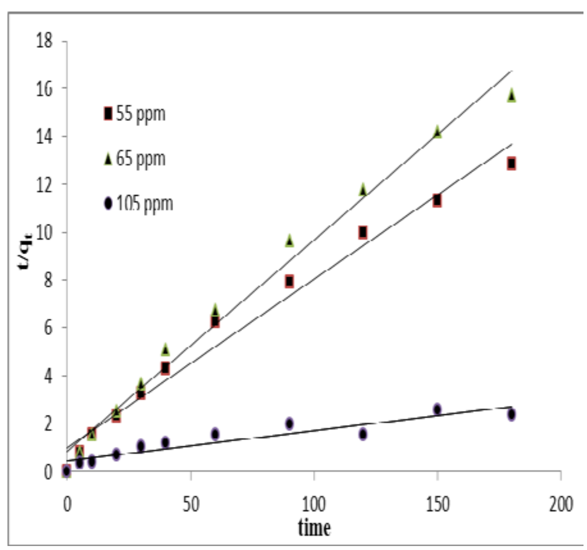

Figure 16 : pseudo 2nd order model for PAC dose 1.5 gm/600L at pH: 7.8

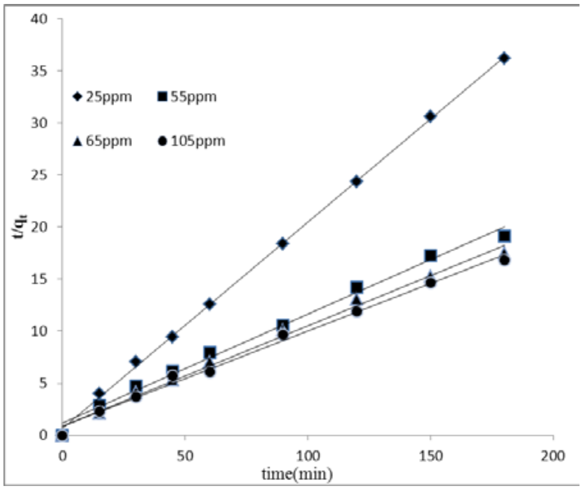

Figure 17 : pseudo 2nd order model for PAC dose 3gm./0.600L at pH: 7.8

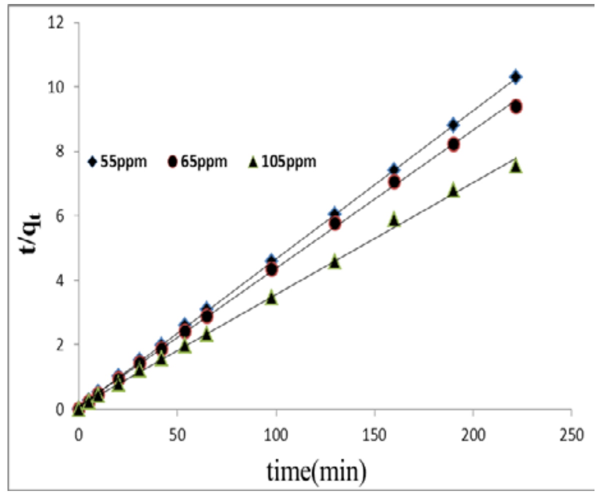

Figure 18 : pseudo 2nd order model at pH: 4 for PAC dose 1.5 gm./.3L.

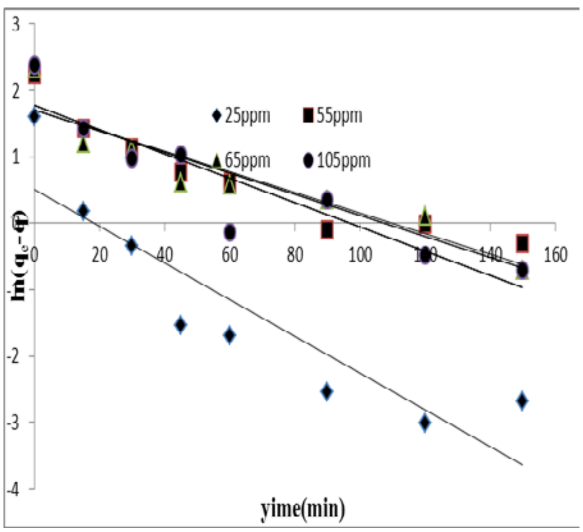

Figure 19 : pseudo 2nd order model for PAC dose 3 gm/.6L at pH: 7.8

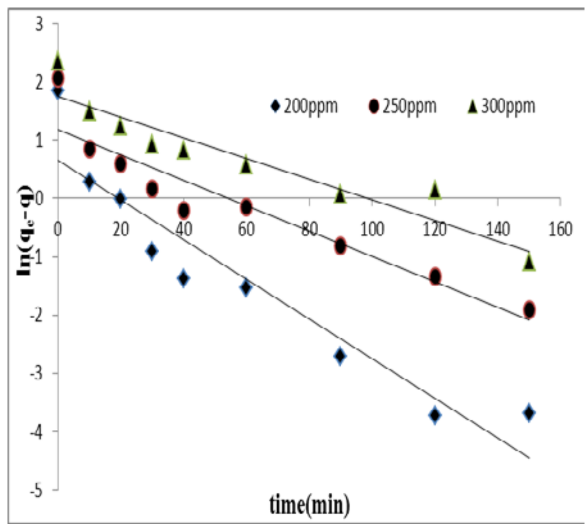

Figure 20 : pseudo 1st order plot for 5gm/600ml at pH: 7.8

4.5.2 *Pseudo 1st order plots for MF-PAC unit (with different PAC loading, initial dye concentration an of different operating pH)*

According to the 1st order equation shown in modeling section, is plotted in Figure 20-22 for various operating condition.

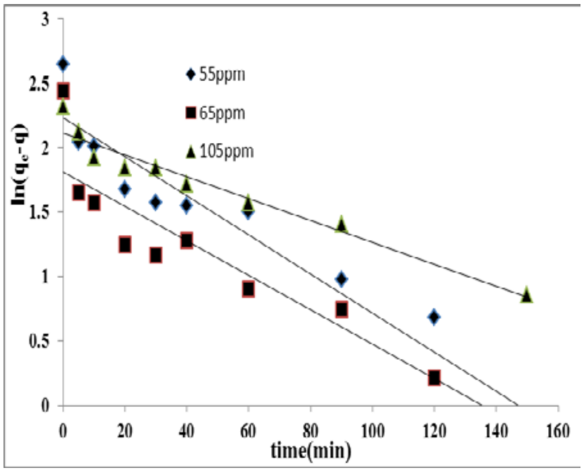

Figure:21 : pseudo 1st order plot for 1.5gm/600ml. at pH7.8.

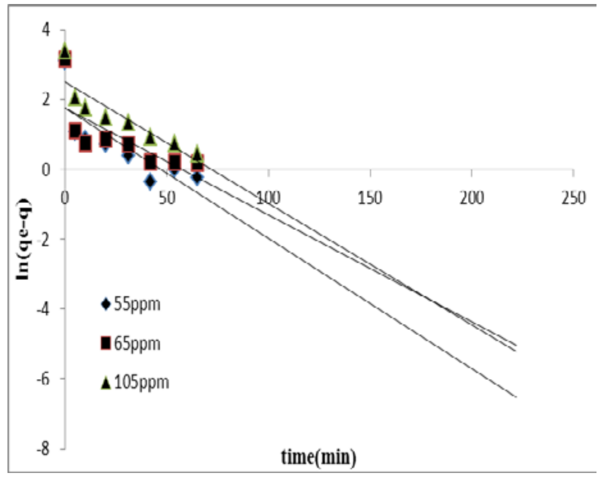

Figure 22 : pseudo 1st order plot for 1.5gm/600ml PAC at pH :4.0

4.6 Pseudo 1st order and 2nd order model parameters for the adsorption of Reactive Red 6BL

Pseudo 1st order and 2nd order model parameter are tabulated in Table 2.

Table 2 : pseudo 1st and 2nd order constants for adsorption of Reactive Red 6BL**.**

pH	**parameter**	**1st order kinetic model**			**Second order kinetic model**		
	Co(ppm)	K_1	q_e	R^2	K_2(kg.kg.min)	q_e	R^2
7.8	25	.027	1.6603	.810	.00972	6.45	.999
7.8	55	.015	5.48	.871	.01214	9.61	.990
7.8	65	.015	5.52	.866	.00817	10.20	.992
7.8	105	.018	5.84	.839	.013	10.96	.991
4.o	55	.030	5.84	.64	.036	21.74	.999
4.0	65	.037	5.70	.59	.022	23.81	.999
4.0	105	.034	12.17	.799	.0133	29.41	.998

5 Conclusion

This study shows that combination of coagulation with alum followed by adsorption with PAC –MF hybrid system for reactive red 6BL dye as an example of wastewater condition was more efficient than adsorption or membrane filtration alone. To overcome the high cost of PAC or membrane, settling for 60 min after complete coagulation and then adsorption-filtration was done to overcome or minimize sludge formation that makes a clogging of filters in industrial treatment operation. The optimum experimental conditions were found in pH 4.5 for alum coagulation for 60 min (equilibrium time at highly % reduction obtained) at room temperature (25 ±2^{o}C).

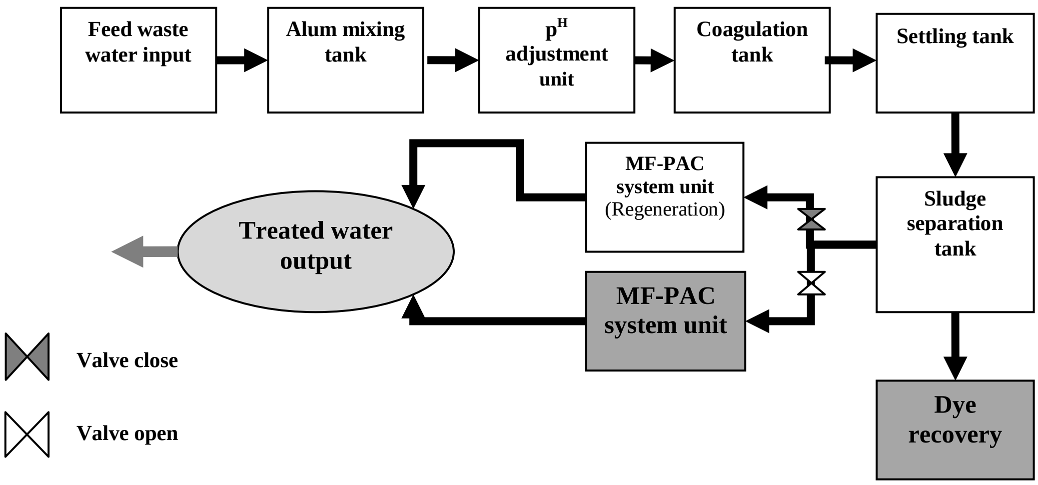

Figure 23 : A flow diagram of a proposed system for textile wastewater treatment and reuse.

A flow diagram of a proposed system for textile wastewater treatment and reused was investigated as shown in Figure 23. Reused effluent may use for cooling or agriculture purposes. The pseudo 2^{nd} order adsorption isotherm model was used for the description of the adsorption equilibrium of Reactive Red 6BL dye onto PAC at different conditions. The data were in good agreement with pseudo 2^{nd} order model.

Here it can be concluded that the combined coagulation–adsorption–membrane process has a great potential application for complete reactive dye removal. It has the capacity for the production of high-quality treated water and allows the reduction in the use of coagulant and adsorbent, which helps in the reduction of amount of sludge produced. Also, the treated water may be reused because of high water quality.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF WATER QUALITY FROM DIFFERENT SOURCES IN KHULNA DIVISION

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ABSTRACT

Intrusion of salinity with gradual increasing rate in coastal areas of Bangladesh leads to critical condition of water supply systems and scarcity of potable water. ***Arsenic contamination in tubewell water is another threat. In those regions pond water is available and comparatively less saline but turbid, colored and contaminated by pathogenic micro-organisms. During cyclone or flood disaster, sea water enters into the ponds that are used for Pond Sand Filters (PSF) and damage the whole systems. Tubewells also get contaminated with high saline water. People cannot use the water without chlorination. The aim of this paper is to present drinking water quality of Dighalia, Koyra, and Paikgassha upazilla of Khulna Division in terms of Chloride (Cl^-), Iron (Fe) and Arsenic (As) content of groundwater. Finally the possible solution of drinking water problem in coastal area was suggested.*

Keywords: *Coastal area, scarcity, salinity, drinking water, GIS*

1. INTRODUCTION

According to 2001 population census, Bangladesh has a population of about 13 crore living on 147,570 square kilometer of land. The Ganges, the Brahmaputra and the Meghna that constitute one of the largest river systems in the world, drain through the Bangladesh into the Bay of Bengal. The country has a coastline of 710 km along the Bay of Bengal. The coast of Bangladesh is known as a zone of vulnerabilities as well as opportunities. It is prone to natural disasters like cyclone, storm surge and flood.

Due to a number of environmental factors, the coastal soils are slightly to moderately saline on the surface and highly saline in sub-surface layers and substrata. The process of accumulation of salts in the soil is called salinisation. There are a number of factors responsible for the salinisation of an area, particularly a coastal area, depending on its situation. The land relief and degree of flooding have mainly affected the formation of coastal saline soils of Bangladesh. The other factors are: i) the nature of the soil, ii) precipitation, iii) tidal action, iv) the effect of the river system and their discharges, v) depth of the groundwater table and salt deposits, and vi) the slope of the ground and the proximity to drainage channels.

Bangladesh has about 2.8 million hectare of land affected by salinity and poor water quality. The total area includes deltaic floodplains and also offshore islands. This comes to about one fifth of the total areas of Bangladesh and lies around the northern apex of the Bay of Bengal. The saline soils are mainly found in Khulna, Barisal, Patuakhali, Noakhali and Chittagong districts of the coastal and offshore lands. This study focuses particularly on three upazila i.e. Dighalia, Koyra, and Paikgassha upazilla of Khulna Division in the context of salinity

2. METHODOLOGY

2.1 Selection of Study Area

Khulna Division lies in the coastal belt of Bangladesh. In ground water of coastal area salinity is a major problem and in shallow aquifer arsenic is also a severe problem. There is acute scarcity of suitable drinking water in Khulna Division. Moreover during any natural calamity like flood, cyclone etc. the amount of Chloride ion increases tremendously. So, to present the current situation of drinking water quality in terms of Chloride, Iron and Arsenic, three upazilla of Khulna Division i.e. Dighalia, Koyra and Paikgacha was selected for the study.

2.2 Collection of Field Data

In order to assess ground water quality parameter, a wide range of homogeneously distributed data is required. In this respect, for collection of field data Dighalia, Koyra and Paikgacha upazilla were further subdivided into union level. Dighalia upazilla was subdivided into Dighalia, Senhati, Gazirhat and Barakpur union. Similarly, Koyra upazilla was subdivided into Koyra, Uttar Bedkashi, Dakshin Bedkashi, Maharajpur union and Paikgacha upazilla was subdivided into Raruli, Haridhali, Kopilmoni and Gadaipur union. Department of Public Health and Engineering (DPHE) install new tubewells every year considering local people request. After installation of tubewells ground water quality are tested by DPHE. In the current study, ground water quality field data of tubewells in terms salinity (Cl^-), hardness (Fe) and Arsenic (As) were collected from the secondary source DPHE, having a sample size of 318 at union level during the year 2006-2008.

Table 1: Summary of Collected Data

Upazilla	Year 2006				Year 2007				Year 2008			
	Number of Tube well	Cl^- (mg/l)	Fe (mg/l)	As (ppb)	Number of Tube well	Cl^- (mg/l)	Fe (mg/l)	As (ppb)	Number of Tube well	Cl^- (mg/l)	Fe (mg/l)	As (ppb)
Dighalia	43	298	0.62	<.01	84	485	0.61	2.02	23	564	2.10	2.52
Koyra	33	507	.62	<.01	33	590	1.54	2.24	34	517	0.97	4.88
Paickghacha	-	-	-	-	38	726	2.42	4.84	30	441	3.31	16

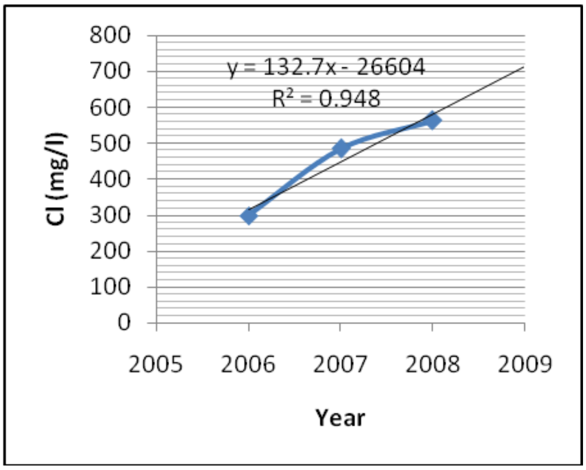
2.3 GIS Application

Salinity intrusion analysis was performed using ArcGIS 9.2 software. For displaying and subsequent processing and enhancement of the salinity map, ArcGIS 9.2 is a proven instrument. Dighalia, Koyra and Paikgachha upazilla was carved out from the whole Khulna Division using the union boundary map of Khulna Division through ArcGIS 9.2. For figuring out the year wise salinity maps at union level of the three upazilla using GIS, following procedure was maintained sequentially

- Khulna Division map was digitized using the union shape file of Bangladesh.
- Dighalia, Koyra and Paikgachha Upazila were located in the Khulna Division shape file.
- Tube well salinity data for the selected upazillas were inputted in the shape file as attributes.
- Salinity condition of the three upazillas being considered for this study was represented by color shades in a map on year basis.
- Potable water supply condition in the selected areas was identified.

3. ANALYSIS OF COLLECTED DATA

Drinking water quality of Dighalia, Koyra and Paikgassha upazilla of Khulna Division in terms of Chloride (Cl^-) Iron (Fe) and Arsenic (As) content were analyzed year wise. Average value of Chloride (Cl^-), Iron (Fe) and Arsenic (As) were plotted against year.



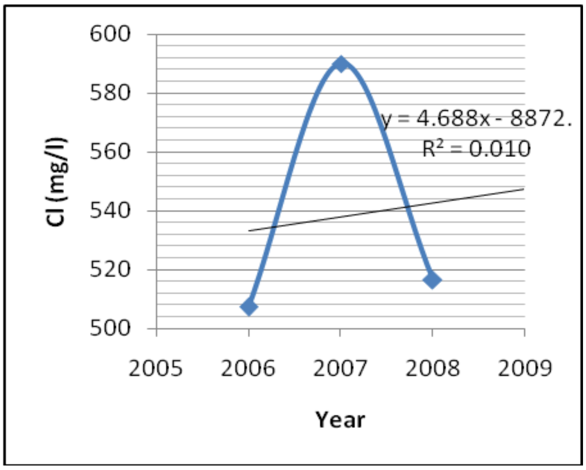
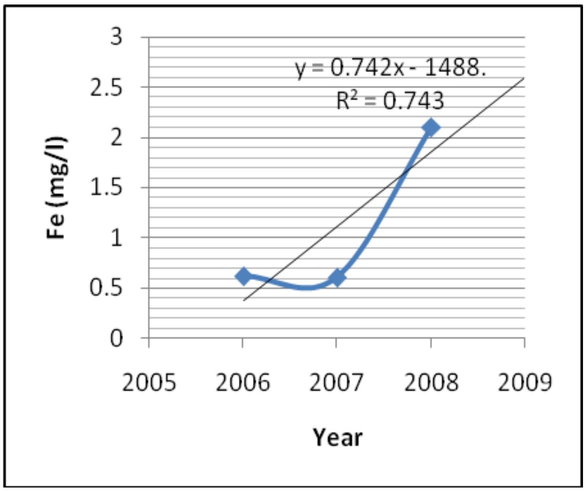


Figure 1: Chloride content (mg/l) with respect to time

Figure 2: Iron
(year) at Digha



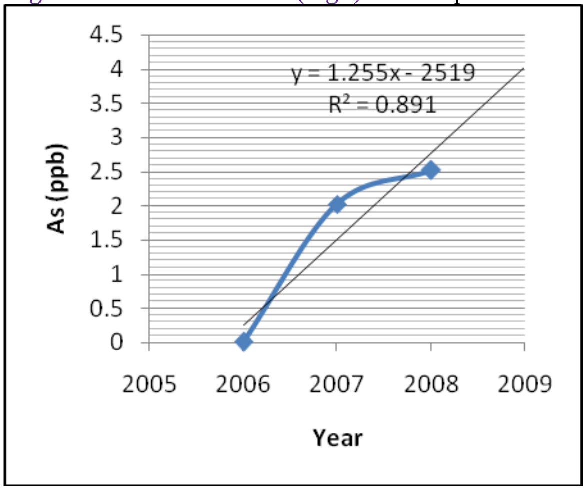
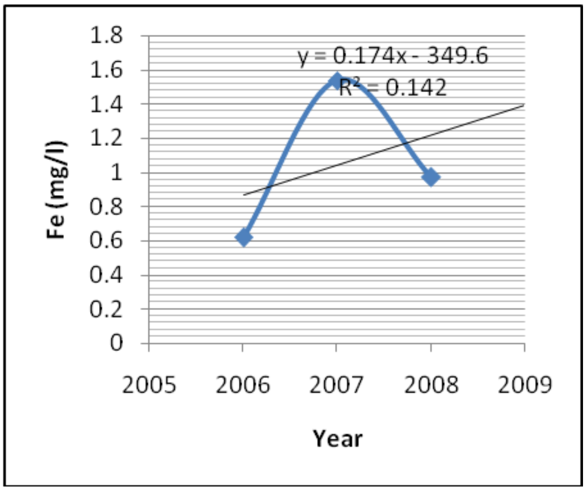


Figure 3: Arsenic content (ppb) with respect to time (year) at Dighalia Upazila

Figure 4: Chloride content (mg/l) with respect to time (year) at Koyra Upazila



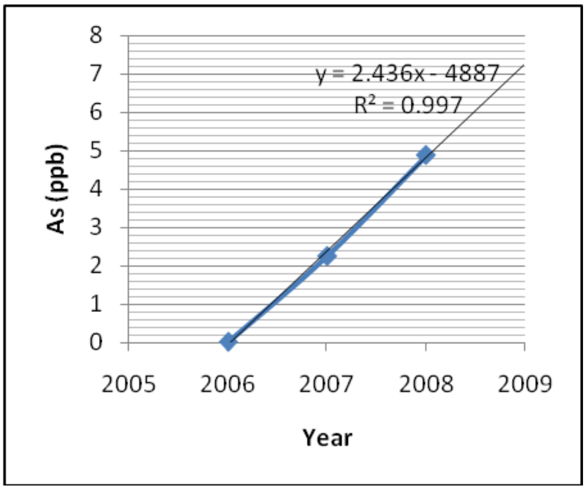


Figure 5: Iron content (mg/l) with respect to time (year) at Koyra Upazila

Figure 6: Arseni
(year) at Koyra U

In Dighalia upazilla, it was observed that, Chloride (Cl-) content in ground water is increasing day bay day (figure-1). Though Iron (Fe) content dropped in the year 2007 (figure-2), even than it has an increasing pattern of growth rate. In the base year (2006) Arsenic (As) content was below 0.01ppb (acceptable limit of drinking water quality in Bangladesh), but after that Arsenic content crossed the limit of drinking water quality with a trend of increasing growth rate (figure-3).

In Koyra upazilla, Chloride (Cl-) and Iron (Fe) content showed similar characteristics. Both Chloride and Iron content of ground water increased rapidly during the year 2007 (from the base year 2006) and then dropped in the year 2008 (figure- 4 & 5 respectively). It can be noted that, among the three upazilla being considered for this study, Koyra upazilla is mostly exposed to the Bay of Bengal. Abrupt rise of Chloride and Iron content during the year 2007 in this upazilla may have the indication related to after effect of cyclone Sidr. Though there was a dropping characteristic observed in the year 2008 for Chlorine and Iron, it can be said that these two water quality parameter is increasing day by day as the trend of growth rate is increasing. On the other hand, Arsenic (As) content was again below 0.01ppb in the base year which shows the same increasing growth trend in the following years (figure-6). Found

In Paikgassha upazilla, only two year data was available (year 2007 & 2008). It was observed that, from the year 2007 to 2008, Chloride (Cl^-) content decreased in this upazilla, whether Iron (Fe) content was increased. The most alarming situation observed in this upazilla was the tremendous increment of Arsenic (As) content in 2008. Average value of ground water Arsenic (As) was 4.41 ppb in 2007, which becomes 16 ppb in 2008.

4. YEAR WISE SALINITY MAP

After performing salinity intrusion analysis using ArcGIS 9.2 software, salinity condition based on tube well data of Dighalia, Koyra and Paikgachha upazilla was represented by color shades in the upazilla map with respect to year.

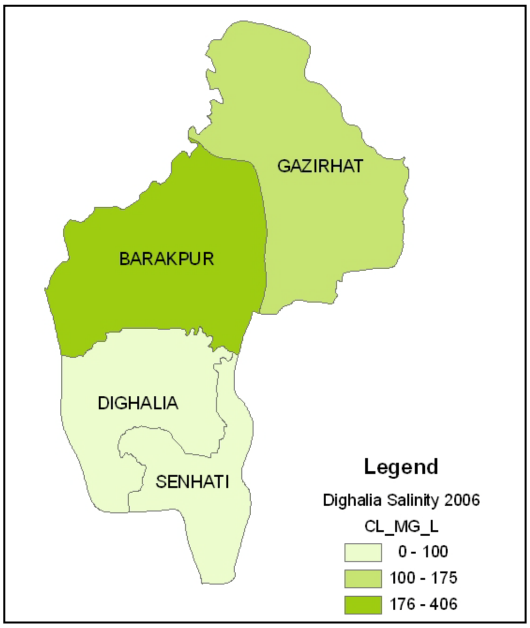


Figure 7: Variation of salinity at different unions of Dighalia upazilla (year 2006)

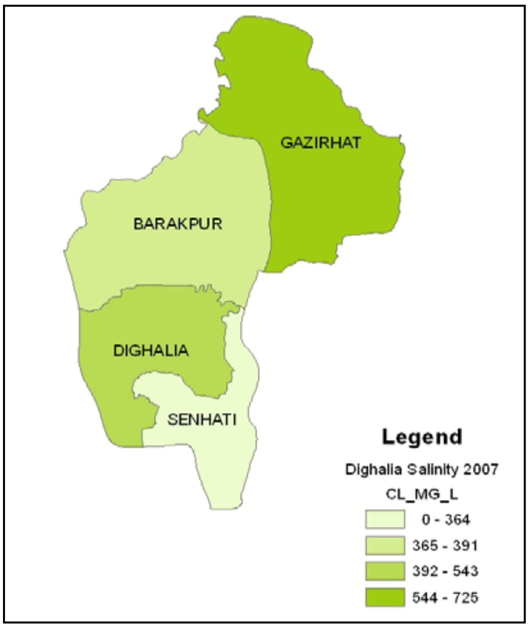


Figure 8: Variation of salinity at different unions of Dighalia upazilla (year 2007)

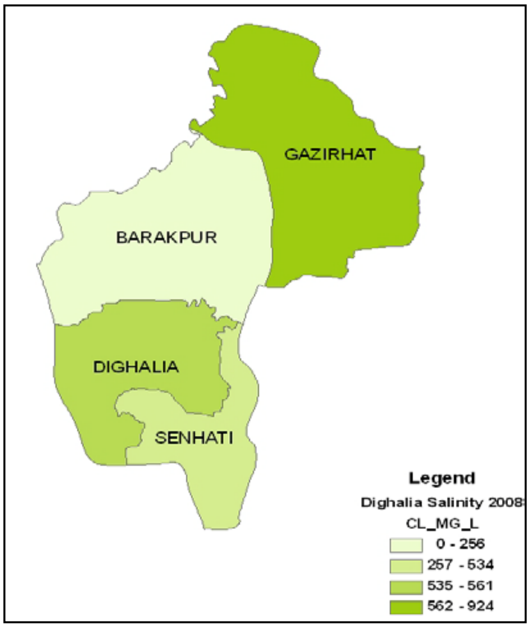


Figure 9: Variation of salinity at different unions of Dighalia upazilla (year 2008)

From the GIS map, it has been seen that the salinity is increasing from year to year. Therefore it can be said that saline intrusion is continuous in Dighalia upazila. Tube well depth for the collected data varies from 130-300m. But there is no correlation found between tube well depth and salinity of tube well water.

Variation of salinity based on deep tube well data at different unions of Koyra upazila (2006, 2007 and 2008) are shown in the following figures

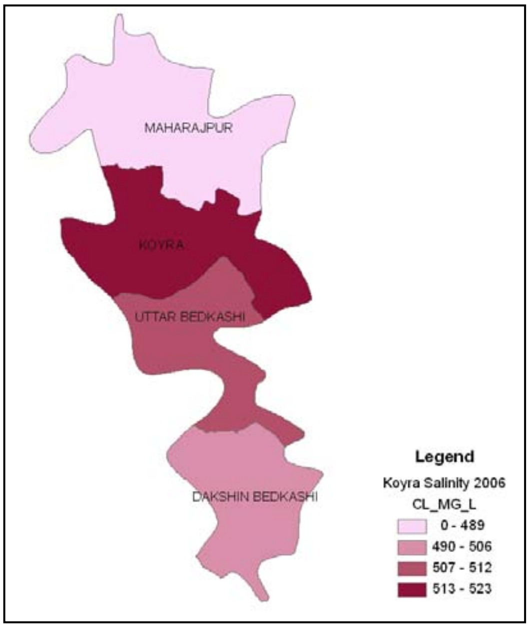


Figure 10: Variation of salinity at different unions of Koyra upazilla (year 2006)

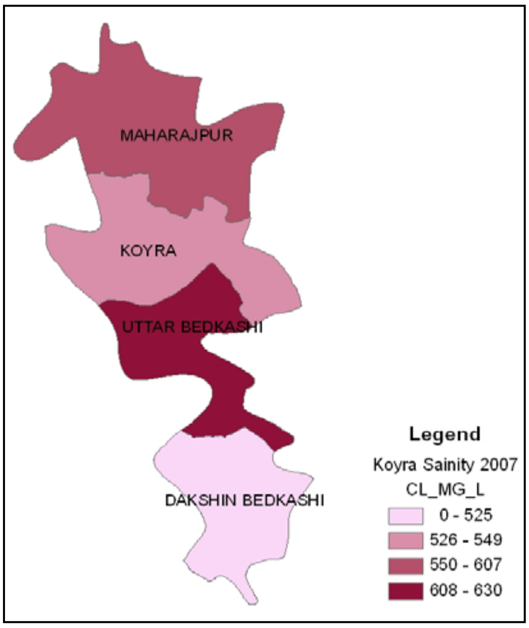


Figure 11: Variation of salinity at different unions of Koyra upazilla (year 2007)

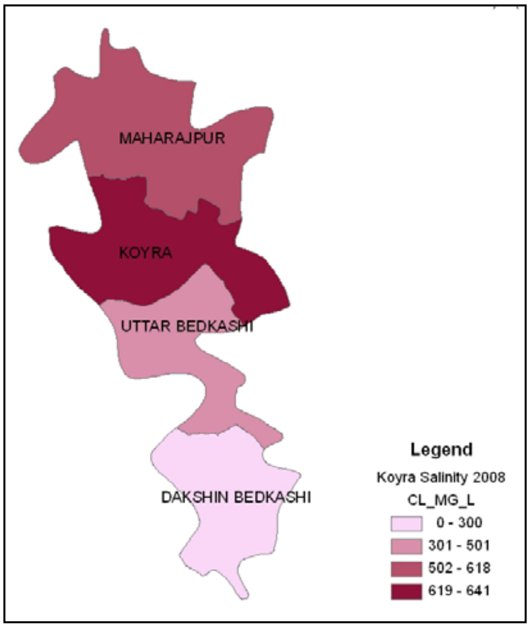


Figure 12: Variation of salinity at different unions of Koyra upazilla (year 2008)

In Koyra upazila, saline water also enters continuously from year to year like Dighalia upazila, and similarly, there is no correlation between tube well depth and salinity of tube well water.

In Paikgachha upazila, only Shallow Shrouded Tubewells (SST) was installed during study period. Variation of salinity based on SST at different unions of Paikgachha upazila (2007 and 2008) are are shown in the following figures

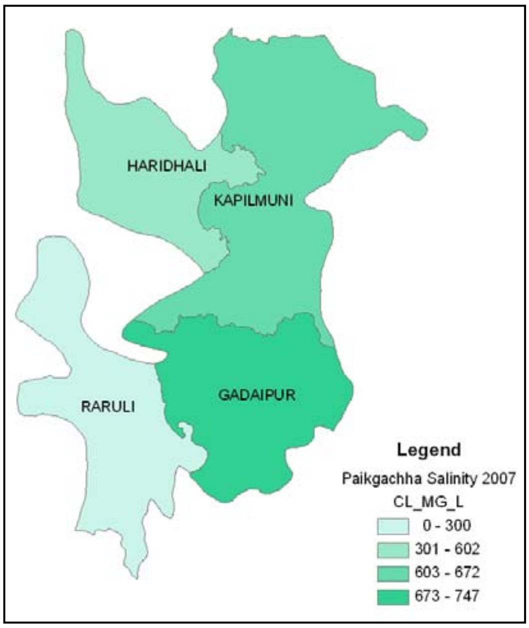


Figure 13: Variation of salinity at different unions of Paikgachha upazilla (year 2007)

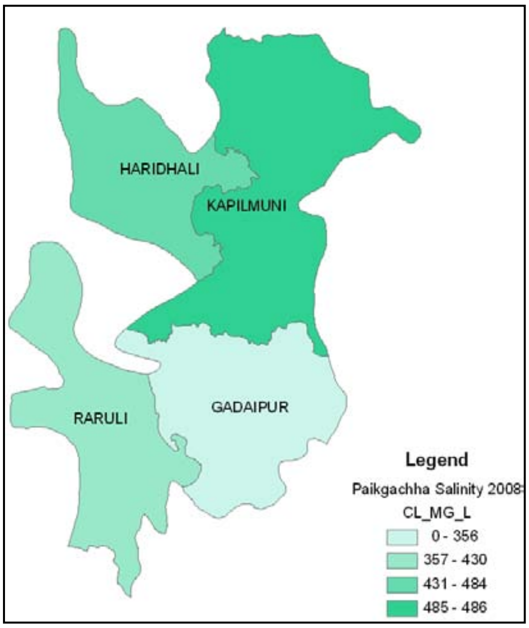


Figure 14: Variation of salinity at different unions of Paikgachha upazilla (year 2008)

The maximum depth of installed SST in Paikgachha upazilla was 18 m. Therefore, intrusion of salinity in deep aquifers was not assessed. Moreover, in this upazilla, deep tubewells installed previously before the study period up to 300m depth, were found unsuccessful. After analysis, it was observed that, salinity of SST varies from 350-750mg/l, which are in acceptable range for drinking water in the coastal belt (1000 mg/l as per Bangladesh Standard). As deep tubewells were unsuccessful in this upazilla, DPHE installed some Pond Sand Filter (PSF) and Rain Water Harvesting (RWH) on experimental basis. After installation and careful observation, it was found that these PSF and RWH may be potential source of drinking water in Paiksachha upazilla.

5. RECOMMENDATIONS

i) In Dighalia and Koyra upazilla, average salinity is 600 mg/l which is within acceptable range according to Bangladesh Standard for coastal belt regions. At present deep tubewells are best solution for drinking purpose of these two upazilla, but as saline enters continuously, so for future, other alternative options like PSF and RWH can be actively considered and exercised.

ii) In Paikgachha upazilla, deep tubewells up to depth of 300 m were found unsuccessful option for collecting drinking water. Increasing the tubewell depth beyond 300m can be examined for getting potable drinking water. Again PSF and RWH were successful in some areas of Paikgachha upazila. So for future, these two options can be potential source of drinking water in Paikgachha upazilla.

iii) The boundary of the ponds that are used as PSF should be protected in such a way that saline water cannot wash the pond. Earthen dam can be made at the boundary of the ponds. So that it can give pure drinking water at the time of disaster.

iv) Rainwater harvesting is a potential water supply option in the acute arsenic and saline affected areas of Bangladesh. Rainwater collection in Bangladesh has been practiced for a long time on a limited scale. So it is recommended that rain water is the good solution for drinking purpose in these areas.

6. CONCLUSIONS

The collected data shows that many deep Tube well of these coastal area contain high salinity. Due to high salinity this water is not suitable for drinking purpose .SST also contains high salinity which does not satisfy the good water quality Standard maximum time. The People of this area also use surface water by PSF. But during flood PSF gets thoroughly corrupted and becomes unsuitable for drinking. On the basis of the data analysis some recommendations are made which can be followed. Pond Sand Filter (PSF) and Rain Water Harvesting (RWH) can be treated potential source of drinking water.

ACKNOWLEDGEMENT

Authors deeply convey their heartiest gratitude to Department of Public Health Engineering (DPHE), Khulna Circle, Khulna

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SUSTAINABILITY OF ECOSAN TOILETS IN WATERLOGGED AREAS: A RURAL PERSPECTIVE IN BANGLADESH

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ABSTRACT

This study aims at investigating the sustainability of Ecosan toilets in waterlogged areas of Bangladesh with regards to ensuring safe disposal of human excreta. Government of Bangladesh (GoB) has set a goal for 100% sanitation by 2013. But, recurrent floods and prolonged water logging make it harder to adapt appropriate sanitation for the displaced and those who are living with flood and water logging conditions. It is very difficult for them to defecate into specific toilet in waterlogged area due to lack of proper sanitation system. It is found that they were extremely demanded of improved sanitation options (toilets) for safe defecation. During the last flood incident in 2011, the inundation depths of water surrounding the Ecosan toilets were recorded to be varied from 3 inch to 15 inch from the ground level. However, no water got entered into the feces chamber. A recent survey in the inundation period identified that around 20% more people in the Ecosan project area, South Banshbaria under Keshabpur Upazila of Jessore district were free from diarrhoeal and skin diseases compared to nearby Dharmapur village where Ecosan toilet is completely absent. Thus, the Ecosan toilet would be a sustainable technique of human excreta management in waterlogged areas of rural Bangladesh.

Keyword: *Ecosan toilet, rural Bangladesh, sustainability, waterlogged areas*

1. INTRODUCTION

Bangladesh is known as one of the most susceptible countries across the globe under natural calamity. The southern, south-western and coastal areas of Bangladesh remain submerged for long periods every year, especially during the monsoon season due to the vulnerable geographical setting and climate change. Both human interruptions upon nature and climate change are responsible for the water logging problem in South-West Bangladesh. Hence, the water inundated the riverside areas. People in these areas have been coping with waterlogged condition for generations. Water logging has been disrupting livelihoods of about one million people in Bangladesh during past two decades. Every year on an average 20% area of Bangladesh are inundated due to annual flood and this is common and normal phenomenon. In Bangladesh, every year, a huge number of sanitation facilities are damaged or destroyed due to natural disaster like flood and cyclone. There is a huge demand for flood resistant, appropriate, socially and culturally accepted technological options for safe disposal of human excreta in the flood prone areas. However, there are limited technological options for sanitation for flood prone areas (Morshed & Sobhan, 2009).

The present study has been conducted in South Banshbaria village in Keshabpur Upazila of Jessore District due to its high vulnerability to water logging for last seven years. From geographical perspective, the village is located in the catchments of river “Kobadak”. Historically, the river has been affected by regional and political decision which was, in most cases, detrimental to the hydrological condition of the region. Due to the siltation of the river, three unions such as Trimohini, Sagardari and Bidyanandakathi in the western part of Keshabpur are affected. As a result, the river has lost its flowing capacity due to sedimentation. Every year the river spreads over the adjacent area of these villages due to heavy rainfall during the monsoon season. Almost eight months in a year most of the land area which is monotonously flat and low elevated is inundated. As a result, the impact of waterlogged condition had smashed the management and safe disposal of human excreta. For defecation, people use the pit latrines with different types of super structure, open spaces, direct pit pour flash toilets, hanging toilets etc. which were very unhygienic and responsible for pollution of sub-surface soil and ground water in rural areas. And it is very difficult for them to defecate into these types of toilet in waterlogged area due to lack of their sustainability. Three years ago, sanitation was very poor in the study area. People of the study area had pit latrines but those latrines were not suitable to use because pits were sunk by water. For this condition, they had been suffering by water borne diseases like diarrhea, dysentery, skin diseases with fever etc. It was found that they were extremely demanded of improved sanitation options (toilets) for safe disposal of human excreta. Ecological sanitation (Ecosan) system is an approach to avoid the disadvantages of conventional wastewater systems which are based on water as transport medium for collection and transport of human excreta via a sewer system. So, non-government organizations had been establishing the Ecosan toilets in the waterlogged area implementing successively by year through foreign project since 2009. In order to be sustainable, a sanitation system has to be not only economically viable, socially acceptable and technically/institutionally appropriate, but also should protect the environment and natural resources (Langergraber and Weissenbacher, 2010). The benefits of ecological sanitation are well-known: it does not need water to function; it protects the environment; and allows the nutrients in human Feces and urine to be returned to the soil as fertilizer (Winblad and Simpson-Hébert, 2004). Ecosan is increasingly recognized as a realistic alternative to provide safe sanitation and thereby reduce the health risks associated with poor sanitation.

In the study, it has been intended at investigating the performance of sustainable Ecosan toilets in waterlogged and/or flood prone areas of Bangladesh with regards to ensuring safe disposal of human excreta with good management.

2. MATERIALS AND METHODS

2.1 Background

The perception towards the sustainability of Ecosan toilets in rural waterlogged areas of Keshabpur under Jessore district was studied using household interview and observation. The survey was carried out in two villages of Sagardari Union in Keshabpur upazila under Jessore district. One of the villages is South Banshbaria named Ecosan village and another is Dharmapur named non-ecosan village. Both of the villages are low lying area and located in the bank of river "Kobadak" showing in figure 1. The survey was based on a sample of 250 respondents of which 125 were representatives of each village. Most of the residents in the villages are poor, illiterate, farmer and low income earners. The field survey was carried out for a period of four weeks from late August to end of September, 2011. Data collection was commenced by conducting interviews.

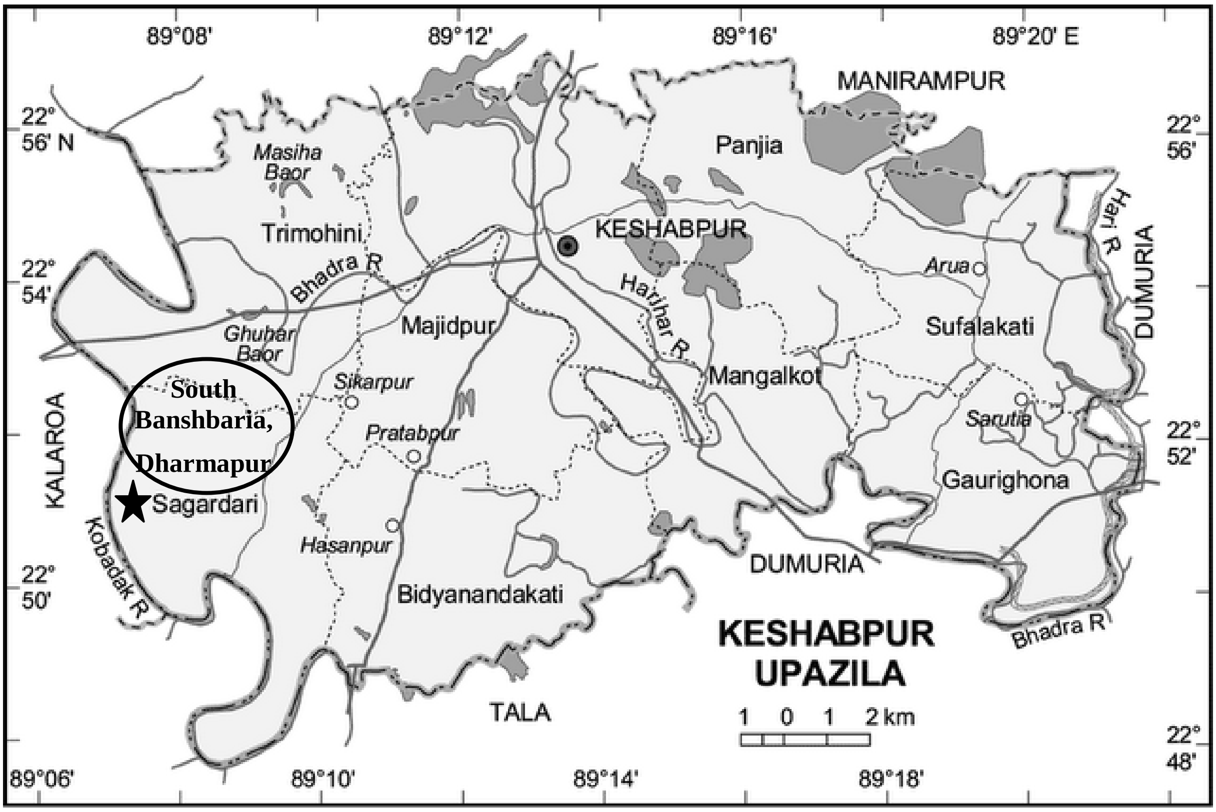

Figure 1: Location of study area under Keshabpur Upazila of Jessore District (Source: http://bdmaps.blogspot.com/2011/12/keshabpur-upazila.html)

2.1.1. Site selection

The first one was also selected because a three year Ecosan pilot project through JADE was first implemented there. It was sponsored by Toto limited, Japan; Mitsui Limited, Japan and Australian High Commission. The project was implemented in this area to improve the ecological sanitation among the villagers. The toilets were given out on first come first serve basis to residents who requested for the toilet through the project coordinator. The beneficiaries paid a small fee because the toilets were subsidized by the project. The second one was selected for comparative analysis of sustainable Ecosan toilets. No improvement of sanitation facilities in this village was occurred.

2.1.2. Household interviews and selection of respondents

Interviews were used to collect data, since it is commonly known and the interviewer is able to get clarifications by probing more on certain issues which may not clearly be answered (Bolt and Cairncross, 2004). It may also generate interesting discoveries (Boot and Cairncross, 1993). Interviews function well when respondents are interested in being interviewed *(*Krantz 2005).

A sample of 125 households in each village was selected. Household interviews were conducted by visiting the households in the community. This was done with assistance of a guide from the study area to locate the household respondents' homes. In cases where the household head or landlord was not present, any adult in the sampled household was interviewed. Data was collected using a structured questionnaire comprising both open-ended and close-ended questions. Questions from the questionnaire were read out to the respondent. The respondent's responses to the questions were then recorded in spaces provided in the questionnaire. For open ended questions, the respondent was at liberty to give multiple answers and also to bring to discussion other issues he/she thought are relevant to the questions. While close ended questions necessitated the respondents to rank, accept by indicating either yes or no answers. Each household interview took about 30 minutes, including an introduction of the study and the researcher, interview of the respondent, and visit to the toilet facility. The interview method took more time because there was communication problem in both areas due to water logging. The questions were prepared in English language but translated into Bengali language in case the respondents felt more comfortable than speaking in English. However, some respondents were not willing to give information or be interviewed because they were busy at their work place while others were simply not interested because they have been interviewed several times by other researchers and that they have not seen any changes as a result. This necessitated a lot of convincing explanations by the interviewer to let those respondents accept to be interviewed.

2.1.3. Field observation

Field observations were carried out simultaneously with the household interview exercise. It was carried out to see the kind of sanitation facilities used within the community, the design of urine diverting toilets, their management and the general sanitation (solid waste management, drainage) within the areas. This method was made possible during each household's respondent interview as the researcher requested to have access to the sanitation facility in the household. Photographs of the toilet facility present were taken to show the type, design and general situation of the facility. This helped to relate to some information collected in interviews. This method was used because one can get accurate behavioral information which is not easily provided in an interview, as well as physical state of subject matter being studied. It was also time-saving and minimized costs since it was done during household interviews.

2.1.4. Data analysis

Information from the interviews was assigned for the various multiple responses to ease the analysis process. Descriptive statistics (i.e. percentages) were derived for the data analysis.

2.2 Structure of toilet

Ecosan toilet is a structure which was built completely on the ground level. It has two portions, one is lower portion called Feces chamber and another is upper portion called defecation place. Feces chamber is the most important part of the structure. During construction period, the chamber is made water proof carefully so that no water enters in it i.e. there is no scope that human Feces get mixed with water. The Feces chamber is made of bricks and well sealed with cement mortar at the bottom and surrounding corners. The front faced and backside wall of the chamber is typically 2.50 ft and 2.0 high from the ground level. An impermeable base is constructed underneath the Feces chamber. In this process, a compacted sand layer is provided on the leveled ground followed by 3 inch brick flat soling and 3 inch cement concrete. Finally, both inside and outside of the chamber are plastered with neat cement finishing (figure 2).

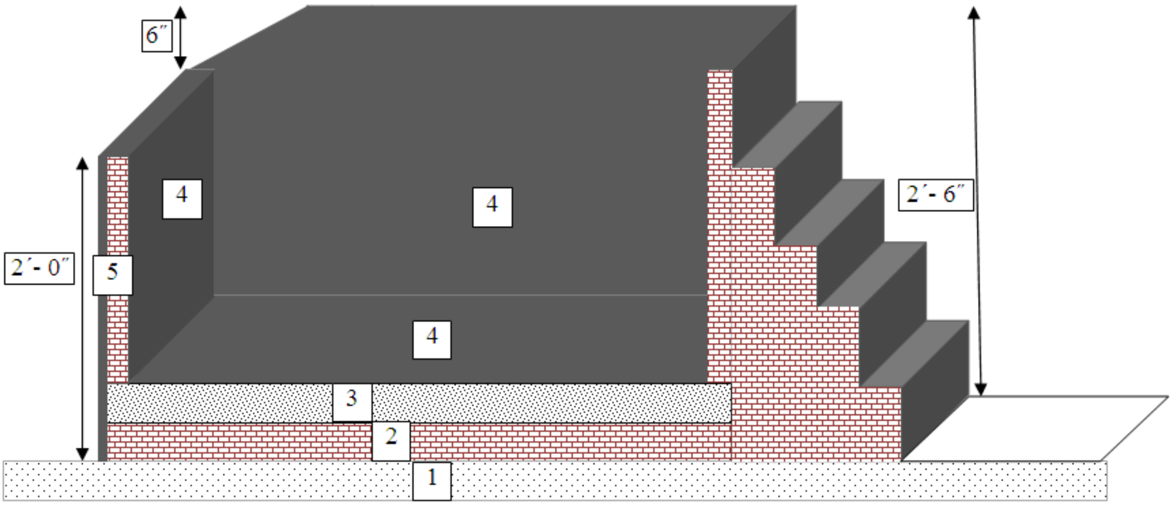

Figure 2: Construction stage of feces chamber

In the figure, steps that are marked for construction are given below:
1: Sand filling with thickness of 3 inch on the plain ground
2: Flat brick soling with thickness of 3 inch
3: 3 inch thick cement concrete casting
4: Inside of the chamber plastered with cement grouting
5: outside of the chamber plastered with neat cement finishing

3. RESULT AND DISCUSSION

3.1 Ecosan toilets in waterlogged area

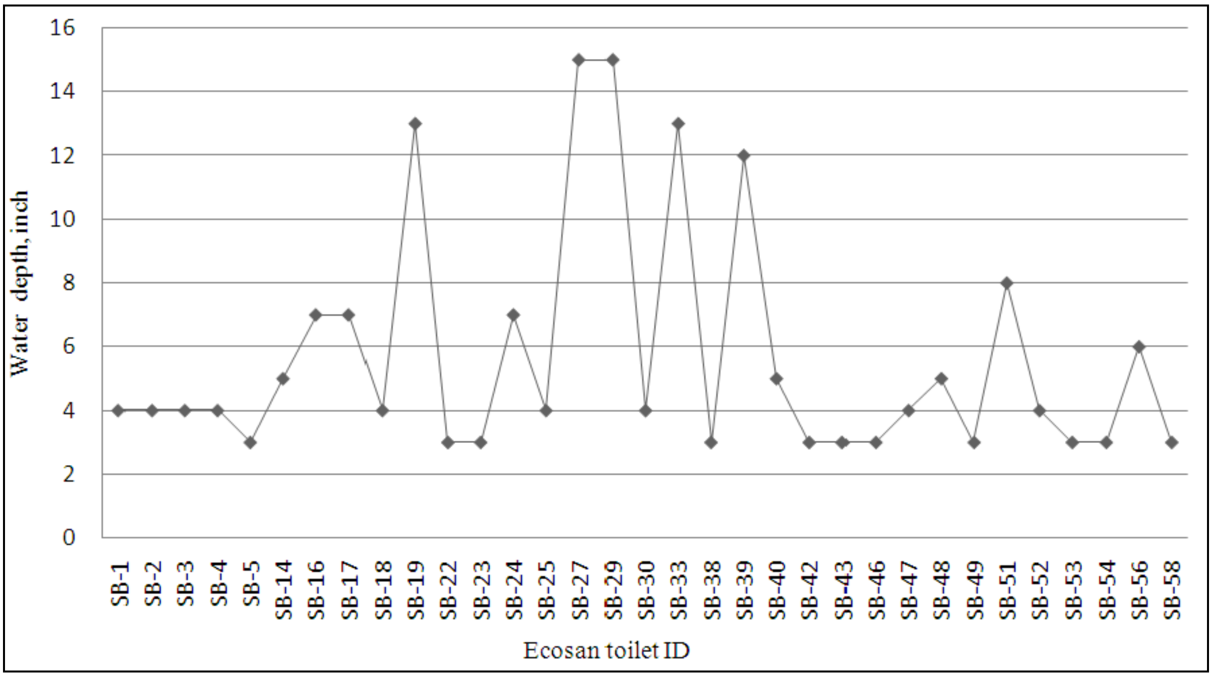

Figure 3: Water depth surrounding the Ecosan toilet during flood, 2011

The village had been inundated in last August, 2011. Shelters, toilets (both Ecosan and non-ecosan), yards, cowsheds, agricultural farmlands of the inhabitants of the village were waterlogged for more than three months. The people who lived in the area faced a vulnerable situation because water spread over the area at most. Water stayed in varying depth due to tidal effect surrounding the Ecosan toilet in the flood prone area during this period. Water depth fluctuated when flow-tide and ebb-tide had occurred through the nearest river. The water depth from ground level surrounding the toilet along the height of the structure has been showed in figure 3. Water depth and identity mark (ID no.) of toilet, in vertical and horizontal axis respectively have been showed in the figure. ID of toilet has been expressed by name of village (in short) with serial no. For example, SB-1 means toilet no. 1 in South Banshbaria (SB). The toilets, marked in the figure had been construction on low lying land in the village.

3.2 Existing facilities of defecation

Proportional result of existing toilets that were counted between the villages in period of survey in last August, 2011 has been detailed in the following table 1. From the table, it is simplified that the sanitation facilities of Dharmapur village is poorer than the village of South Banshbaria. The result shows the 96 no. of families in Dharmapur village are deeply accustomed with open defecation that is largely reduced to 15 in Banshbaria. But 43 no. of households, comparatively higher than Dharmapur use pit toilet in South Banshbaria. There are two types of toilet such as hanging toilet and direct pit pour flash toilet, alternatively absent in Dharmapur and South Banshabria respectively. After all, it is a great difference with Ecosan toilets between these villages. From three years ago, Ecosan toilets had been going ahead to bring a revolution all over the concept of ecological sanitation in the area. For this, South Banshbaria is called Ecosan village mentioned in art. 2.1. They are using Ecosan toilets through which human excreta such as feces and urine is being become organic fertilizer as soil conditioner. A good management of human excreta with safe disposal has brought an approached sanitation throughout the toilets.

Table 1: Comparative result of existing defecation facilities, 2011

Serial no.	**Types of toilet**	**Existing no. of household toilet**	
		South Banshbaria	**Dharmapur**
1	Open defecation	15	96
2	Pit latrine	43	28
3	Hanging toilet	0	1
4	Direct pit pour flash toilet	6	0
5	Ecosan toilet	61	0
Total no. of toilet		125	125

As many in other developing countries, sanitation remains a major challenge in Bangladesh. In the following figure 4, it is showed that a huge amount of open defecations (approximately 80%), large no. of pit latrines (about 20%), presence of a hanging toilet and completely absent of direct pit pour flash and Ecosan toilets (0%) has lead a poor sanitation facility to the people of Dharmapur village. The low sanitation coverage poses a serious public health concern. Sanitation method aims to decrease spreading of diseases by adequate waste water and human excreta with proper treatment, proper handling of water and food and by restricting the occurrence of water borne diseases such as diarrhoea, dysentery, skin diseases etc. On the other hand, South Banshbaria village represents good sanitation with large amount of Ecosan toilets (around 50%). Although there are also pit latrines (about 35%), open defecation (12%) and a little no. of direct pit pour flash toilet, sunk by water in the village. After all, People were well practiced with the Ecosan toilets during the inundation period and thus leading sanitized life. But, human health had been affected by the water borne diseases at lower rate than other areas where Ecosan toilets are completely absent.

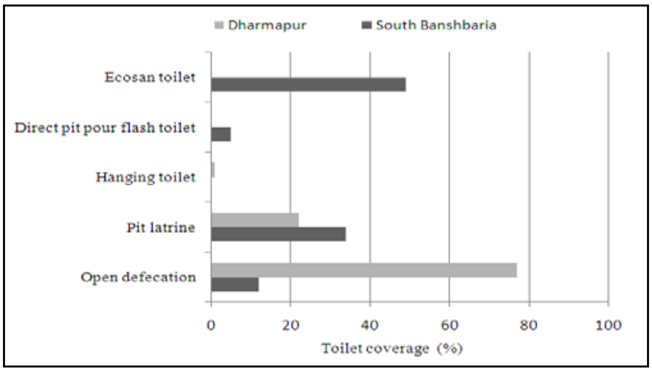

Figure 4: Progress on sanitation facilities

3.3 Impact of water borne diseases

From the above discussion, the villages of Dharmapur and South Banshabria are different from the ecological sanitation point of view. During the last flood period in 2011, a preventive rate of water borne diseases carried out through survey work. Flood had occurred due to water level in the river raised for heavy rainfall for which the land areas of villages were washed out. In the circumstances, water borne diseases spread all over the villages. From child to old, most of the inhabitants were suffering from diarrhoea, dysentery, skin diseases, fever etc. which were found in survey work. In villages, open defecation and pit latrines are jointly featured but variation in existing no. of facilities. On the other hand, people whose toilets were sunk by water defecated on open space in non-ecosan village. The contamination rate of diseases transmission was varying from village to village which showed in table 2. From the table, it is mentioned that the rate of contamination in Dharmapur is higher than South Banshbaria due to largely variation of open defecation rate discussed in art. 3.2.

Table 2: Comparative results of diseases prevalence

Serial no.	Name of diseases	No. of patient		Percentage (%) of patients	
		South Banshbaria	Dharmapur	South Banshbaria	Dharmapur
1	Diarrhoea	12	120	2.4	24
2	Dysentery	17	36	3.4	7.2
3	Skin diseases	82	178	16.4	35.6
4	Fever	107	160	21.4	32

From the interviewing of respondents during period of survey, it was found that they were not conscious about the hygiene practice. As a result, they were suffering in malnutrition and water borne diseases. No. of patients of water borne diseases in Ecosan village is lesser than non-ecosan village (table 2). No. of families are 125 and total population is more or less 500 that is same in both villages. Among these diseases, skin diseases infected a large no. of people in both villages because their body stayed contact with contaminated water for their daily activities. There is no option of safe drinking water in Dharmapur where rate of the contamination of diarrhoea is higher than South Banshbaria where two pond sand filters (PSFs) had been established in 2010 and running still now. People of the village collected the water from the filter units for which they were safer than others. In the inundation period, it was identified that around 20% more people in the Ecosan project area, South Banshbaria under Keshabpur Upazila of Jessore district were free from diarrhoeal and skin diseases compared to nearby Dharmapur village where Ecosan toilet is completely absent (figure 5). It was also found that about 15% more people were infected by more than one disease in Dharmapur. However, patients were suffering in fever, indentified with parallel the water borne disease due to environmental change in these areas.

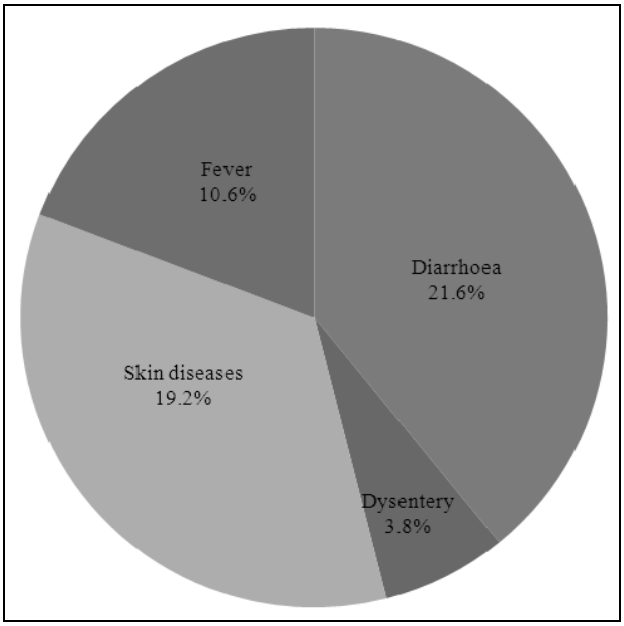

Figure 5: Reduction rate of water-borne diseases and viral fever

3.4 Sustainability of Ecosan toilets

Figure 6: A common feature of pit toilet in waterlogged area

Pit latrine is the most common and low-cost technological option for sanitation. In rural Bangladesh more than 90% of the latrines are pit latrines. Culturally, people of Bangladesh use water to clean themselves after defecation. Therefore, urine, cleaning water and excreta are being accumulated in the single pit, which causes rapid fill up of the pit and subsequent return to open defecation due to lack of replacement space. In flood prone areas, overflowing pit latrines during flooding pose a high health risk. The major problems of sanitation in flood-prone areas are surface water contamination and loss of accessibility to the latrine during flood. Thus surface water gets mixed with a large range of pathogenic organisms of viral, bacterial, parasitic protozoan and helminthes origins may be present in human excreta. The significant lack of technological options particularly in difficult and flood areas means the government target of 100% sanitation by 2013 is likely not to be reached and/ or not sustainable.

A pit latrine which was placed in village of Dharmapur has been located in figure 6. The figure indicates the common feature of pit toilets in flood affected area. It is realized that this type of toilet was absolutely useless and unhygienic. Bad smell, mixing of water and human excreta, risky structure were the problems of pit toilets. The toilet is source of water borne diseases that are spread out through surface water contamination with pathogenic organisms, present in human excreta. In this process, rate of contamination increased in the area and people were suffering from the diseases for a long period.

Figure 7: A photograph of direct pit pour flash toilet in South Banshbaria

A direct pit pour flash toilet, placed in flood prone area has been marked in figure 7. In the flood affected village of South Banshbaria, this type of toilet became unsuitable to use showing in the figure. It is also a source of water borne diseases as pit latrine. People of the area had been suffering for three months with a great sorrow. The pit of the toilet was filled up with water which polluted the environment. Respondents of this toilet used Ecosan toilet of another respondent during the inundation period.

Figure 8: Ecosan toilet in waterlogged area

Two years ago, Ecosan toilets had been constructed in the study area in 2009. After construction, this was first time that faced the flood. So, it was a big challenge about its importance and sustainability in water logging area. Ecosan toilets were used successfully during the last flood period when other types of toilets were completely useless shown in figure 6 and figure 7 because Ecosan, superstructure had been constructed on the ground level. It has no component below the ground level. It has two parts such that lower portion and upper portion. Construction of lower portion, 2 ft high above the ground had been described in art. 2.2 i.e. defecation place is 2 ft 6 inch high above the ground level. Flood level did not exceed the height. On the other hand, toilets were built considering impermeability of chamber. During the flood period, inside of chamber was dry at most because no water entered into it although water remained surrounding the structure showing in figure 8. Most of the inhabitants, both Ecosan and non-ecosan users used this toilet. No water pollution occurred by the toilet because human excreta did not get mixed with water. Inside of the toilet was dry. Human excreta were being deposited safely in the chamber. The Ecosan users were informed that wooden ash will be applied to feces to be dried in the chamber if the water enters. About one year later, the dried feces turn into organic fertilizer. Covering and drying up feces with ash suppresses foul odour and eliminates the generation and infestation of flies and other insects, which can mediate contact between fecal pathogens and human. In addition, people do not need to see the feces deposited by others in chambers by covering feces with ash.

Sanitation systems can be considered as sustainable if they protect and promote human heath, do not contribute to environmental pollution, and are technically and institutionally appropriate, economically viable and socially acceptable. An overall simplified model of sustainable sanitation is shown in the following figure 9 (Chowdhury and Mamun, 2007):

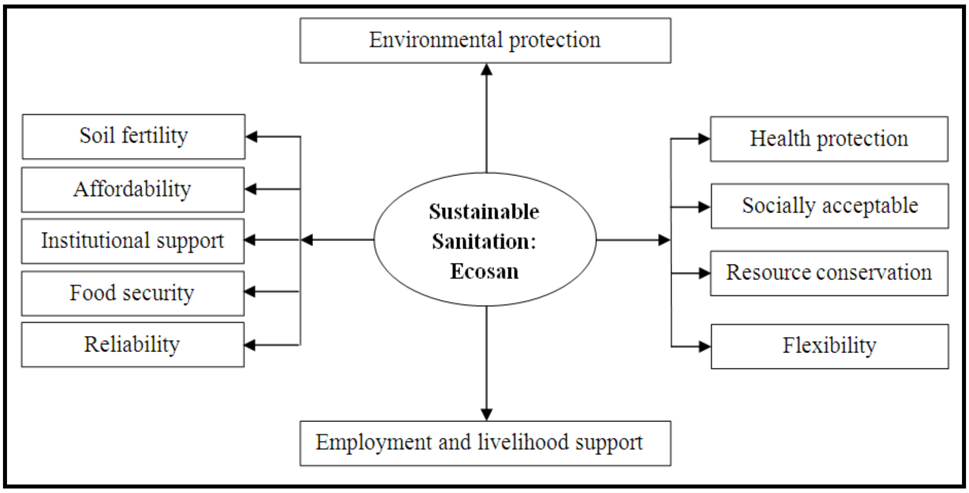

Figure 9: Components of sustainable sanitation

4 CONCLUSION

It is proved that Ecosan, recognized and accepted technology to improve the poor sanitation system can be applied in water logging area of Bangladesh. In people's living environment, sanitation reduces pathogenic bacteria and incidence of parasite-borne diseases, which reduces infant and child morbidity and mortality. In Bangladesh, every year flood visits and stays for several months when people are not able to use toilets. Living environment may be deteriorated when the human excreta in the pit overflow. Improved sanitation against flood causing insanitary situation will lead to continuous health risk mitigation. It is also an approach to promote sanitation facilities and hygiene practice in rural areas of Bangladesh. However, Ecosan can provide flexibility of rural people. Ecosan toilet is essential for the rural people to lead themselves diplomatic life. So Ecosan technology is considered as most expectable alternative option for sanitation facilities in place of pit latrines and open defecation in flood prone areas of rural Bangladesh.

ACKNOWLEDGEMENT

The authors would like to express deep gratitude to Non-profitable International Organization named Japan Association of Drainage and Environment (JADE) that implemented the project, financed by Toto Limited, Japan; Mitsui Limited, Japan and Australian High Commission for the study. Many thanks go in particular to the rural people who helped us to complete the project successfully. Finally, we express our gratefulness to the God.

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PERFLUORINATED CHEMICALS-A NEW CLASS OF GLOBAL POLLUTANTS OF THE WORLD

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ABSTRACT

Recently, perfluorooctane sulfonate (PFOS) and perfluorooctanoic acid (PFOA), which are concerned as a new class of persistent organic pollutants, have been found to be widely distributed in many living organisms. From half a century ago, perfluorinated substances have been being widely used in industrial goods and processes as well as in consumer products. Exposures to PFCs result in potential developmental and other adverse effects in animals. The paper is focuses on the application, production processes, bioaccumulation in living organisms including and toxicity effect of living organmisns of PFOS and PFOA. These compounds were detected globally in the tissues of fish, bird and marine mammals, their concentrations from relatively more industrialized areas were greater than those from the less populated and remote locations. Blood samples of these living organisms in various countries were found to contain PFOS and PFOA which suggested a possibility of atmospheric transport of these compounds. There is still a death of information about the environmental pathways of PFOS and PFOA. These findings indicate different patterns of animal and aquatic organims exposure to PFCs examined and stress the importance of conducting research to identify the environmental sources and pathways of exposure to PFCs.

Keywords: *Perfluorooctane sulfonate(PFOS), perfluorooctanoic acid (PFOA), bioaccumulation, living organisms, toxicity effect.*

1. INTRODUCTION

Perfluorinated compounds such as perfluorooctane sulfonate (PFOS) and perfluorooctane acid (PFOA) have been recognized as emerging environmental pollutants because of their ubiquitous occurrence in the living organisms. PFOS and PFOA are the candidates for new class of persistent organic pollutants which are an organic compound that remain intact in the environment for long periods, extremely resistant to degradation because of their very strong carbon-fluorine bonds which contain both atoms of carbon and fluorine and it is most abundant in halogen family and most reactive to all elements. The carbon-fluorine bond in PFCs is very strong and gives thermal and chemical stability to many PFCs. The stability that makes fluorinated compounds desirable for commercial use also makes them potentially significant environmental contaminants due to their resistance to natural breakdown processes, that is, their persistence (Key et al, 1997).

Research over the past few years has shown that the perfluorooctane sulfonate (PFOS) and perfluorooctane acid (PFOA) is now a widely distributed and accumulated in the fatty tissue of living organisms and are toxic to human and wildlife. This review gives a light knowledge about application, production processes, and contaminations of the water, biota, human and wild animals and toxicological studies.

1.1 Physio-Chemical Properties of PFCs

Water is the most non-biota environmental compartment of concern for PFOS and PFOA because these compounds have moderate water solubility, while atmospheric PFOS and PFOA might be not as such concern because of their involatile characteristics. The characteristics of PFOS and PFOA, shown in Table 1, indicate that they are rather low in vapor pressure and highly soluble in water.

Table 1: Physio-chemical properties of PFOS and PFOA (OECD, 2002).

Properties	PFOA	PFOS
Molecular weight	414	500 (for acid)
Melting point	45-50°C	>= 400° C
Boiling point	189-192 C (736 mm Hg)	Not calculable
Vapor pressure	10 mm Hg (25° C)	3.31 ×10 -4 pa (20° C)
Solubility in pure water	3.4 g/L	570 mg/L
pKa	2.5	NA
pH	2.6	7-8

1.2 Production of PFCs

At first, PFCs have been produced by the 3M Company since 1956. PFCs used in industries were mainly derived from two major classes named perfluoroalky1 sulfonates (PFASs) and perfluorocarboxylate acid (PFCAs), which were produced in processes of electro-chemical fluorination (ECF) and telomerization respectively.

Electro-chemical fluorination (ECF) was developed in 1945 by Dr. Simons in Penn State University, and adopted by 3M Company to produce organofluorine chemicals. In this processes, electric current was passed through dispersed solution of alkyl organics and hydrogen fluoride (HF), causing hydrogen atoms to be replaced by fluorine atoms (3M, 1999). 3M is the only major company known to use the electrochemical fluorination process. Other companies use a different process for the production of PFCs namely the telomerization process. Telomerization has recently become more widely used in the production of many PFCs. Telomerization processes was developed by Dupont, and related products based on telemerization are manufactured by a number of companies, including Dupont (USA), Asahi Glass (Japan), Atofina (France), Clariant (Germany), and Daikin (Japan) (Lange et al., 2006). Telemer was generated by reaction of perfluoroethylene ($CF_2 = CF_2$) and perfluoroalky1 iodides (CF_3 CF_2I). Telemerization processes exclusively generated linear products with even number of carbon atoms. Telemerization processes are still applied in industries to produce PFCs now.

1.3 Application of PFCs

Within the past 50 years, industrial application as well as consumer uses of perfluorinated chemicals (PFCs) such as perfluorooctane sulfonate (PFOS) and perfluorooctanoic acid (PFOA), as well as their related products have increased tremendously. PFOS and PFOA related compounds have been in commercial use since 1960 (Lehmler, 2005) in a wide range of applications in the board categories-surface treatment, paper coatings and performance chemicals. Surface treatment applications provide soil, oil, and water resistance to clothing, carpets, car interiors and furniture. The US EPA reported that 37% of the production of fluorinated surfactants is used for surface treatment applications (Kannan et al, 2002a). For paper protection, perfluorinated chemicals were applied to food packaging (containers, bags, warps). According to Kannan et al the US EPA reported that 42% of sulfonated fluorochemicals were used for coatings on paper products in 2000 (Kannan et al, 2002a) and performances chemical include fire fighting foams, mining and oil well surfactants, photolithography, electronic chemicals, hydraulic fluid additives, floor polishes, photographic film, denture cleaners, shampoos, etc. while PFOA is used as an emulsifier (such as Teflon used for coating of cooking pans) and surfactant (such as soap and shampoos, etc).

2. BIOACCUMULATION IN LIVING ORGANISMS

2.1 Freshwater Invertebrates and Fishes

PFCs are the hazardous chemicals building up in environmental contaminant. PFOS has moderate acute toxicity to freshwater fish and slight acute toxicity to invertebrates. Biological monitoring surveys conducted using tissues of marine organisms reported the occurrence of perfluorooctanesulfonate (PFOS) and related perfluorinated compounds in freshwater invertebrates and fishes.

In Canada, PFOS was found in various samples of fish collected at different locations in the circumpolar region (Martin et al, 2004). A study in Japan analyzed concentrations of PFOS in surface water and fish from Tokyo

Bay (Taniyasu et al, 2003). Based on the concentrations of PFOS in water and in fish livers, bioconcentration factors were calculated to range from 274 to 41600. The PFOS was detected in livers of 13 fish species from Japanese marine and fresh water. Concentrations ranged from <7 to 381 ng/g wet weight, which, it was noted were similar to concentrations found in fish liver from Tokyo Bay in another study. PFOS was also detected in fish eggs at concentrations approximately twice those as in the livers from the same species of fish. PFOS is actively transferred from adult female fish to their eggs. Furthermore, occurrence of PFOS in eggs has implications for early life stage effects.

2.2 Marine Fish & Marine Mammals

PFOS and PFOA are widespread in the global environment and are present in the tissues of marine fish and marine mammals living organisms. In this study PFOS was the predominant compound of all the PFCs that were investigated. Concentration ranges reported for conventional POPs in marine fish samples are varying considerably depending on sampling location and species caught. Marine fish species were approximately ten times lower contaminated compared to the previously described freshwater fish samples.

A study on fish from different coastal regions of Japan investigated concentrations of PFCs in samples of blood and liver (Taniyasu et al, 2003). PFOS was detected in all samples and the concentrations in blood ranged from 1 to 834 ng/ml and in liver from 3 to 7900 ng/g wet weight. Berger et al indicated that PFOS was the predominant PFCs in Nordic countries (Berger et al, 2004). The least contaminated fish originated from the Faeroe Islands. Around Iceland, fish contained unusually high levels of perfluorodecane sulphonate (PFDcS) (median 10 ng/g wet weight) and perfluorohexanoic acid (PFHxA) was present at concentrations of >1ng/g wet weight. It was suggested that the different patterns of contamination indicated country specific contamination with PFCs. Overall; the marine fish that were analysis from Nordic countries in this study were approximately ten times less contaminated compared to freshwater species from Nordic countries. A study on fish collected from the Mediterranean, specifically the Italian coast, investigated bluefin tuna (Thunnus thynnus) and swordfish (Xiphias gladius) (Kannan et al, 2002c). Concentrations of PFOS in blood ranged from 4 to 52 ng/ml and in liver ranged from <1 to 87 ng/g wet weight. Perfluorooctanesulfonamide (FOSA) was detected in all blood samples of fish that were analysis at concentrations of 1.1 to 28 ng/ml.

Another study of Arctic animals measured PFOS in the livers of seven polar bears from the Canadian Arctic (Martin et al, 2004). Concentrations of PFOS ranged from 1700 to >4000 ng/g, mean 3100 ng/g. The study also investigated the levels of other PFCs. In this study, 247 tissue samples from 15 species of marine mammals collected from Florida, California, and Alaskan coastal waters, the northern Baltic Sea, the Arctic (Spitsbergen), and Sable Island in Canada were analyzed for PFOS (Kannan et al, 2001b). PFOS was detected in liver and blood of marine mammals from most locations including those from Arctic waters. The greatest concentrations of PFOS found in liver and blood were 1520 ng/g wet wt in a bottlenose dolphin from Sarasota Bay, FL, and 475 ng/mL in a ringed seal from the Northern Baltic Sea, respectively.

The study also suggested that the feeding ecology of the marine mammals, either in inshore or in offshore waters, was likely to affect the level of PFOS contamination in the animals. For instance, those feeding in inshore coastal waters may be more highly contaminated due to feeding nearer to PFC sources than offshore feeders. In the study, results showed that the offshore feeders, namely, the sperm whale, fin whale and white-sided dolphin, had comparatively lower tissue concentrations of PFOS than coastal.

2.3 Birds

PFOS has been found in birds of entire world like as United States (Kannan et al, 2001a), Canadian Arctic (Martin et al, 2004), Japan (Taniyasu et al, 2003) etc. A study in Japan and Korea determined PFOS and other PFCs in several species of birds from different locations (Kannan et al, 2002b). PFOS was found in bird livers at the several range of ng/g, which suggests the widespread occurrence of this chemical. It is also shown that urbanized and industrialized areas are major sources of exposure to PFOS whilst remote areas are less so.

A study on the presence of PFOS in fish-eating water birds from across the USA was conducted (Kannan et al, 2001a). PFOS was found in most samples of blood and liver tissue from the birds that were analyzed. Concentrations in blood plasma for birds from the Midwestern United States ranged from 13 to 2220 ng/ml, and concentrations in liver tissue for birds from various locations ranged from several ng/g wet weights, the greatest concentration of PFOS is 1780 ng/g wet wt of those areas. PFOS was also found in the sera of albatrosses from the central North Pacific Ocean at concentrations ranging from 3 to 34 ng/mL. And it is also detected in yolk from bird eggs.

In birds, the embryo is exposed to chemicals during the most critical early development phase. Birds, like mammals, are susceptible to alterations in hormone levels and their reproductive cycle is controlled by hormones. In birds, thyroid hormones regulate metabolism, growth, weight, nervous system function, egg hatching, molting, and reproduction. Bird eggs are good contaminant monitoring tools, and also serve as sentinels for human health. Sea birds, such as gulls, have long life spans, low reproductive rates and a delayed onset of reproduction. Population stability depends on a high adult survival rate and even small reductions in the adult survival rate may have large consequences for the overall population.

3. TOXICITY EFFECTS ON LIVING ORGANISMS

There is lot of information on toxicity of PFCs in the literature, mainly for the persistent and bioaccumulative key compounds PFOS and PFOA (DePierre, 2002; Hekster et al, 2002; OECD, 2002; USEPA, 2002; USEPA, 2003). The majority of the information on the toxicity of PFCs (mainly PFOS and PFOA) comes from animal studies and information on adverse health effects in humans is limited to a small number of occupational studies (Calafat et al, 2006). For a brief overview of the toxic effects of PFOS and PFOA (Lehmler, 2005). Despite the limited number of human studies, there are serious health concerns over the widespread exposure of human populations to PFCs, especially considering their highly persistent and bioaccumulative nature and increasing evidence of their potential developmental effects.

In summary, a diversity of adverse effects has been reported in organisms exposed to various PFCs. Although these effects often occur only at levels higher than those expected to be encountered in the environment at present, this is not always the case. Precise details of the mechanisms and potency of toxic effects are still relatively poorly understood for this group, for which the scale extent of environmental contamination has only recently emerged. More ever, what limited information does exist focuses primarily on PFOS and PFOA.

Although there have not been epidemiological effects of PFOS and PFOA on animals and humans, the presence of these compounds in tap water, surface water and animal and human tissues indicates their global contamination and bioaccumulative phenomena in the ecosystems. Regular monitoring of the contamination levels of PFOA and PFOA in the tap water and surface water and in animals and high tissues should be conducted.

3.1 Birds

The reproductive toxicity of PFOS to birds has been investigated in two species, namely the Northern bobwhite quail (Colinus virghinianus) and the mallard (Anas platyrhynchos) (Giesy et al, 2004). The toxicological and potential ecological significance of this effect is unknown. In both species, PFOS was measurable in the yolk of eggs that were laid by exposed females. Based on the results of this reproductive toxicological study, and based on levels of PFOS in wild birds that were measured before the year 2000, the study performed a hazard assessment which suggested that in most circumstances, the concentrations found in bird tissues do not seem to pose a significant threat to the health of wild birds.

3.2 Rodents

PFCs have been found to cause a variety of toxicological effects in rodents. Rodent studies report reduced birth weight, increased postnatal mortality, delayed sexual maturation, hepatotoxicity and haematological (blood) effects in rats (USEPA, 2003; USEPA, 2005). In rats adverse effects have been observed at a serum PFOA level of 370 µg/L (Butenhoff et al, 2004; Environmental Working Group, 2006; USEPA, 2003). The rat of urine excretion was found in rats to increase with decreasing chain length of perfluorocarboxylic acids (Kudo et al, 2001). Biological half-life of PFOA in plasma of a few days for mice and rats and half life of PFOS varies from 7.5 days in rats. Giesy reported on PFOS in mink from the USA (Giesy et al, 2001a). The largest concentration that was found in mink livers was 4800 ng/g wet wt. Mink could be more or less sensitive to PFOS than rats, so it is possible that current environmental levels of PFOS may already be causing adverse effects in such wildlife species.

A study on developmental effects in rats also reported significant increases in liver weight in newborns exposed to PFOS in utero (Lau et al, 2003). It was suggested that the developing liver is a potential target for PFOS action. PFOS and PFOA have been found to cause hepatic (liver) peroxisome proliferation in rats that were given a single injection of these chemicals.

3.3 Monkeys

Today, very little is known about toxicity and ecotoxicity of PFCs related compounds. The toxicity profile of PFOS is similar among rats and monkeys. PFOS have half life in the Cynomolgus monkeys is 200 days (OECD, 2002).

According to OECD, the US Environmental Protection Agency conducted a study on rhesus monkeys which showed that no monkeys survived beyond 3 weeks into treatment with PFOS at a dose of 10 mg/kg/day. At a dose of 4.5 mg/kg/day, no monkeys survived beyond 7 weeks into treatment (OECD, 2002). A study on cynomolgus monkeys showed they died at doses as low as 0.75 mg/kg/day. There were also changes in the livers of the monkeys and significant reductions in blood cholesterol.

3.4 Aquatic Organisms

A review of aquatic toxicity studies, mainly published by industry, reported that PFOS is moderately acutely toxic and slightly chronically toxic to aquatic organisms (Hekster et al, 2003). A study was carried out to test the toxicity of PFOS in various small freshwater organisms. PFOS has been shown to be acutely toxic to fish in short-term studies, at concentrations below the limit of its solubility in freshwater approximately 370 mg/l. PFOS was also acutely toxic to fish in salt water (3M, 2003). With regard to steroid hormones, levels of testosterone were significantly reduced in male and female fish at PFOA water concentrations of greater than 1.0 mg/l. Such reductions in steroid hormones, however, only appeared to have a modest effect on reproduction in the fish. For instance, there was a trend towards reduced egg production. It was suggested that further studies needed to be carried out on reproduction effects to investigate effects in subsequent generations of fish after exposure of the parents.

A diversity of adverse effects has been reported in organisms exposed to various PFCs. Although these effects often occur only at levels higher than those expected to be encountered in the environment at present, this is not always the case. Some species appear to be markedly more sensitive than others. Precise details of the mechanisms and potency of toxic effects are still relatively poorly understood for this group, for which the scale and extent of environmental contamination has only recently emerged

4. CONCLUSIONS

PFOA and PFOS are globally distributed in wildlife and the aquatic environment. Based on their persistence, bioaccumulation potential, toxicological concerns, and the global distribution, voluntary actions of the fluorochemical industry and regulatory measures have taken place or are currently underway. PFCs have been found in the water environment of various countries at the several ng/L levels. The wastewater pathway seems to be a major entrance pathway of PFCs in the aquatic environment. These chemicals in the environment are generally a concern because there is no known route of degradation either in the environment or by living organisms.

Both PFOS and PFOA have been found to be globally distributed in versatile living organisms including humans, birds, fish etc in the range of several mg/l but their contaminations in animals in industrialized areas were more than those from the less populated and remote regions. Although there have not been epidemiological effects of PFOS and PFOA on animals and humans, the presence of these compounds in animal and human tissues indicates their global contamination and bioaccumulative phenomena in the ecosystems. For PFOA, some of the toxicology end points of interest in animal studies include postnatal development, liver to brain weight ratio, body weight change etc. While numerous animal toxicity studies exist, the results are difficult to interpret for the general human population. Until today, very little is known about the probable exposure routes. Because of the ubiquitous nature of PFCs in the environment, continued research into the possible health effects is certainly warranted.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF PHARMACEUTICAL INDUSTRIES IN BANGLADESH - AN EFFECTIVE STEP TOWARDS THE ACHIEVEMENT OF ENVIRONMENTAL SUSTAINABILITY

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ABSTRACT

Bangladesh sustains mainly on its industry business both in socio economically and geographically. Industries mainly pharmaceutical industry is one of those which signify not only for its revenue but also for the safety and proper medication depends on it as long as the health issue is concerned. Now-a-days the importance is focusing on burning issues like environmental sector because public health is in great threat for environmental decaying due to improper procedures of industries. This project was aimed to assess the present conditions of pharmaceutical industries as well as a cluster of industries and tried to evaluate the present scenario and figure out some of the mitigation measures of the existing problems through the concept of Initial Environmental Examination (IEE). A few important environmental parameters like pH, TDS, TSS, BOD, COD were measured for assessing the impacts on environment due to pharmaceutical industries in Bangladesh. This research tried to focus on the adverse impacts as well as the health impacts and a reflection on the environmental conservation rule regarding the issue. The assessment concluded with some rational mitigation measures which would be firmly related to the public and manufacturer to protect the environment from pollution.

Keywords: *EIA, environmental impact, mitigation measures, pharmaceutical industry, pollution.*

1. INTRODUCTION

The government of Bangladesh has already declared pharmaceutical sector of Bangladesh as the thrust sector. According to the information of directorate general of drug administration of Bangladesh 257 allopathic companies are operating at present in Bangladesh. Our locally produced medicines are exported to 83 countries including Europe, America & others. The pharmaceutical sector in Bangladesh is mostly developed in the areas of drug formulation & manufacturing of finished products. Pharmaceuticals were thought to reach the environment primarily through usage or inappropriate disposal. Various Production facilities were found to be sources of much higher environmental concentrations than those caused by the usage of drugs. The Pharmaceuticals plants generate a large amount of wastes during manufacturing, housekeeping and maintenance operations everyday which are being directly discharged into the surrounding channel, agricultural fields, irrigation channels, surface water and mostly enter into Turag and Shitalakkhya River. Different classes of drug have been documented as environmental pollutants such as analgesics, antibiotics, antiepileptic, antihypertensive, antiseptics, beta-blocker heart drugs, contraceptives, hormones, and psychotherapeutics. Worldwide detection of waste pharmaceuticals in the environment causes risks associated with their introduction into wildlife habitats and is becoming a serious issue for both regulators and the pharmaceutical industry. People who live surrounding area of Gazipur utilizing surface water for their house hold washing, bathing, irrigation, fish culture and other necessary works are affected enormously by these pharmaceutical wastes. This assessment provides an overview of pharmaceutical manufacturing processes and operations that generate waste and presents options for minimizing the generation of waste materials through source reduction and recycling in such cases where suitable opportunities exist. Proper analysis is needed to assess the pollution level also for the protection of environment and natural resources. Besides we also did the environmental impact assessment in the industrial zone adjacent to Beximco Pharmaceuticals Ltd. and Healthcare Pharmaceuticals Ltd. on the basis of relevant environmental parameters and made an EIA report based on it.

2. METHODOLOGY

The full methodological flow diagram is shown in figure 1.

2.1 Study Area

1. On-site assessments and interviews with relevant personnel including workers, managers, and local people.
2. Visiting two renowned and international pharmaceuticals industries.
3. Understanding the operational activities of a pharmaceutical industry and its compliance with the environment along with the process flow diagram of the ETP used there.
4. Collecting the samples from both effluent and underground water and analyzing the laboratory experimental data.
5. Preparation of EIA report from the interviews of relevant workers and local people.
6. Literature review of relevant issues and locate findings.

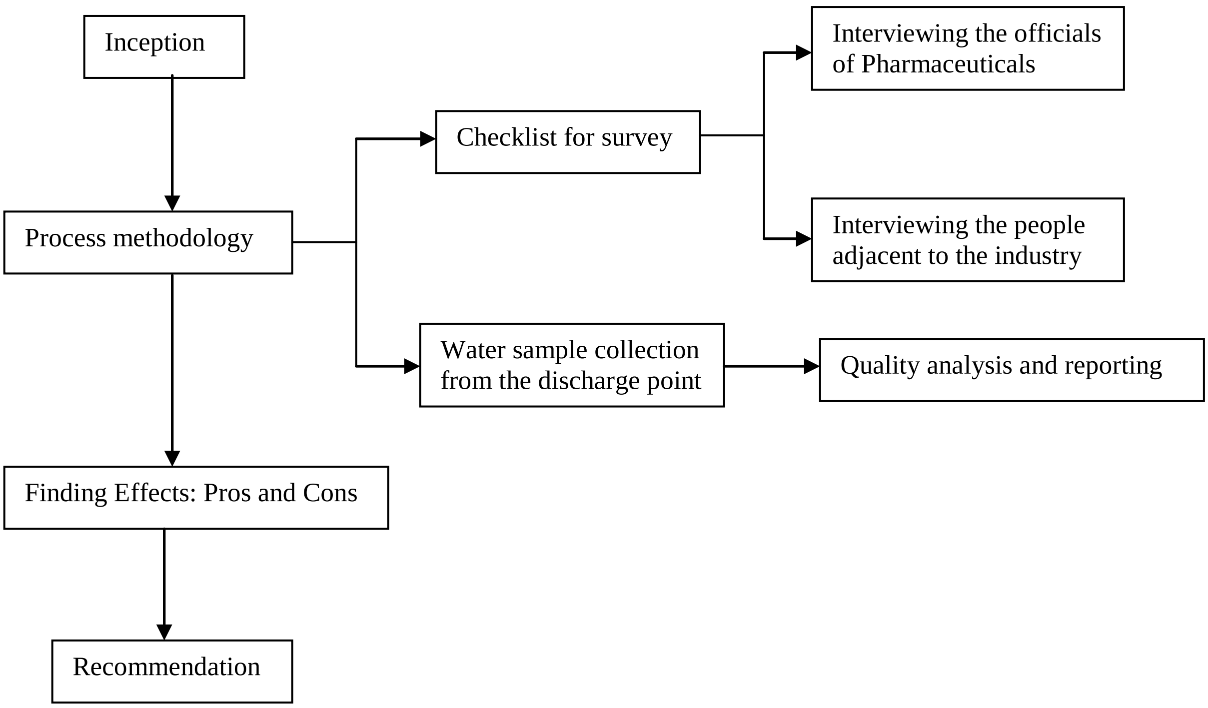

Figure 1: Methodological flow chart

3. CONTRIBUTION OF DIFFERENT PHARMACEUTICAL INDUSTRIES IN BANGLADESH

The modern era of the pharmaceutical industry of isolation and purification of compounds, chemical synthesis, and computer-aided drug design is considered to have begun in the 19th century, thousands of years after intuition and trial and error led humans to believe that plants, animals, and minerals contained medicinal properties. The unification of research in the 20th century in fields such as chemistry and physiology increased the understanding of basic drug-discovery processes. Identifying new drug targets, attaining regulatory approval from government agencies, and refining techniques in drug discovery and development are among the challenges that face the pharmaceutical industry today. The continual evolution and advancement of the pharmaceutical industry is fundamental in the control and elimination of disease around the world.

Among the 257 pharmaceutical industries in Bangladesh some of the industries are well acknowledged due to their contribution in economy and well process structure in industrial processes which are illustrated in table 1 and figure 2 (http://www.beximco-pharma.com/).

In the pharmaceutical industry various types of processes are involved in the manufacture of pharmaceutical products. Due to the diversity of these processes, it provides a general set of waste minimization guidelines that would apply to all drugs manufacturing process. Along with research and development, four methods used in the manufacturing of pharmaceuticals are considered (Salini et. al., 2010):
1) Research and development

2) Chemical synthesis
3) Natural product extraction
4) Formulation.

Table 1: Top ten pharmaceutical industries in Bangladesh

Top 10 Pharma	Sales in $ Mn
Square	$138.7
Incepta	$ 52.1
Beximco	$ 49.0
Acme	$ 36.3
Eskayef	$ 32.1
ACI	$ 31.3
Opsonin	$ 29.7
Renata	$ 29.1
Aristopharma	$ 28.8
Drug International	$ 23.4

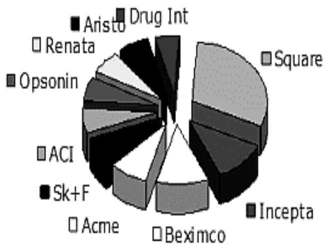

Figure 2: Top ten pharmaceutical industries in Bangladesh

3.1 Research and Development

Research and development (R&D) department in the pharmaceutical industry include chemical research, Microbiological research and pharmacological research. In this wide range of chemical and biological laboratory, wastes are produced. The most common chemical wastes produced from research and development department includes halogenated and non-halogenated solvents, photographic chemicals, radionuclide, bases, and oxidizers.

3.2 Chemical Synthesis

In drug manufacturing plants, reaction vessels and ancillary equipment are often arranged into separate, process units, process functions (i.e., flow rate, pH, and temperature) according to good manufacturing practice protocols. At the end of whole process equipment is thoroughly cleaned. Chemicals used in chemical synthesis operations range include organic and inorganic reactants and catalysts. Manufacturers use a large group of solvents listed as priority pollutants,and these are used for product recovery, purification, and as reaction media.

3.3 Natural Product Extraction

Natural product extraction is the production of pharmaceuticals from natural material sources such as roots, leaves, barks and animal glands. Such pharmaceuticals, which typically exhibit unique pharmacological properties, include allergy relief medicines, insulin, morphine, alkaloids, and papaverine etc. During each process step, the volumes of materials are reduced and final purification may occur on volumes very less than the initial volume. Wastes from natural product extraction include spent raw materials such as leaves and roots, water-soluble solvents, solvent vapors and waste waters. Extraction waste waters typically have low biological oxygen demand (BOD), chemical oxygen demand (COD) and a pH in the range of 6 to 8.

3.4 Formulation

Pharmaceutical formulation is the preparation of various dosage forms such as tablets, capsules, liquids, parenterals, and creams and ointments etc. Tablets account for over 90 percent of total medications taken orally, and types are: plain compressed, coated, and molded. Capsules, in hard or soft form, are the second most widely used oral dosage form for solid drugs. The third type of pharmaceutical formulation is the liquid dosage form prepared for injection or oral use, which includes solutions, syrups, elixirs, suspensions, and tinctures, all of which are usually prepared by mixing the solutes with a selected solvent. Ointments are usually prepared by melting a base, which is typically the petroleum derivative petrolatum. This base is then blended with the drug and the cooled mixture is passed through a colloid or roller mill. Creams are oil-in-water or water-in-oil emulsions.

4. PROCESS DESCRIPTION

The synthesis of medicinal chemicals may be done in a very small facility producing only one chemical or in a large integrated facility producing many chemicals by various processes. Most pharmaceutical manufacturing plants are relatively small. Organic chemicals are used as raw materials and as solvents. Nearly all products are made using batch operations. In addition, several different products or intermediates are likely to be made in the same equipment at different times during the year; these products, then, are made in campaigned equipment. Equipment dedicated to the manufacture of a single product is rare, unless the product is made in large volume. The principal manufacturing steps are: (a) preparation of process intermediates; (b) introduction of functional groups; (c) coupling and asterification; (d) separation process such as washing and stripping; and (e) purification of the final product. The process flow diagram for pharmaceuticals industry is shown in figure 3.

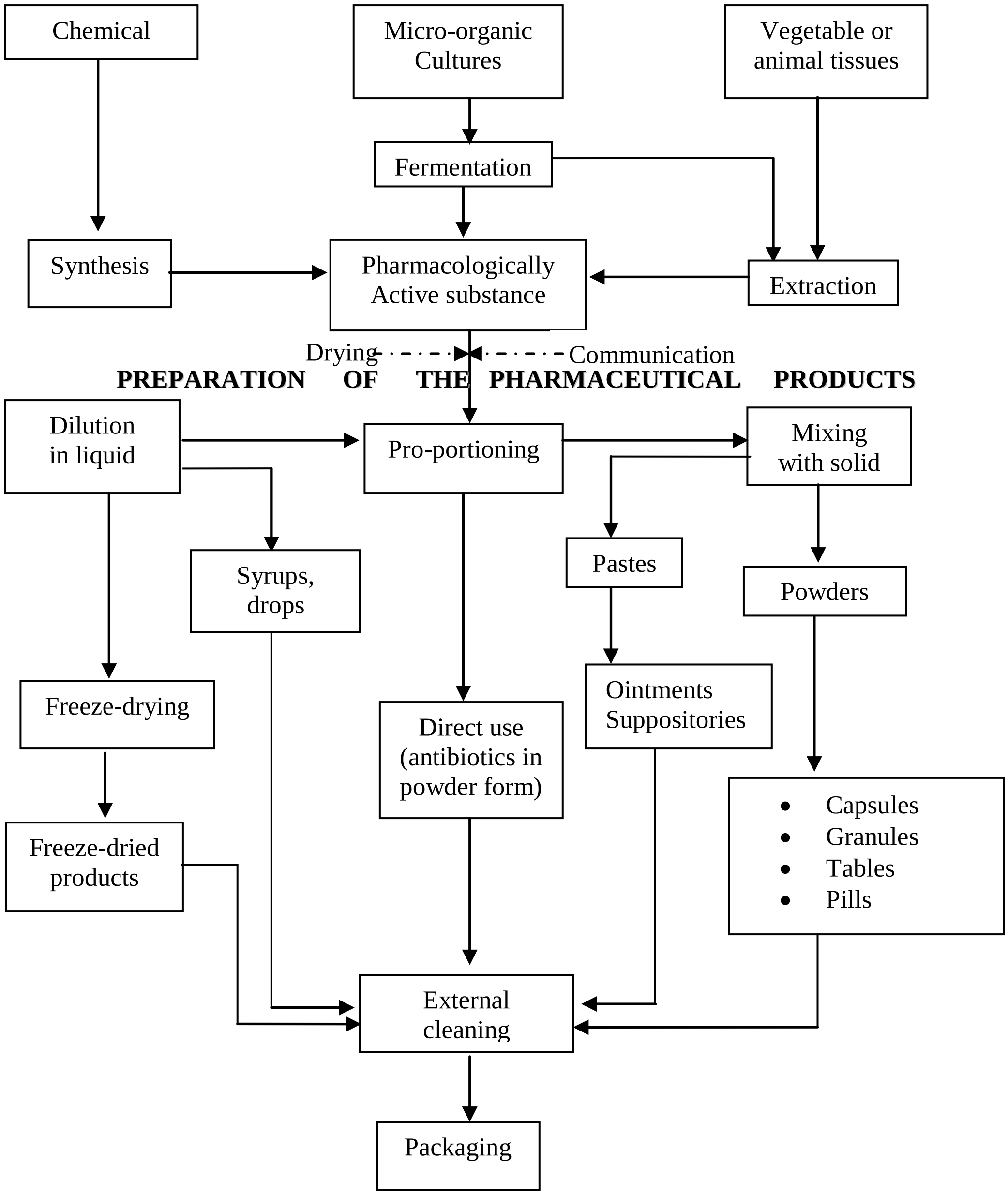

Figure 3: Detailed process diagram of Pharmaceutical industry

Volatile organic compounds, which are the main pollutants, may be emitted from a variety of sources within plants synthesizing pharmaceutical products. The following process components have been identified as VOC sources and will be discussed further: reactors, distillation units, dryers, crystallizers, filters, centrifuges, extractors, and tanks. The manufacture of the following types of pharmaceutical products can generate hazardous wastes:

- Organic medicinal chemicals
- Medicinal from animal glands
- Inorganic medicinal chemicals
- Antibiotics
- Biological products
- Botanicals

- Miscellaneous products

The largest quantities of hazardous waste are from the production of organic medicinal chemicals and antibiotics. Major pollutants are shown in table 2 (Oley, 2001).

Table 2: Major Pollutants from solvent use in Pharmaceutical Production Pollutant

(Solvent) Waste	Ultimate Disposition (%) Air Emissions	Sewer	Incineration	Solid
Acetic anhydride	1	57	-	42
Acetone	14	22	38	719
Amyl alcohol	42	58		
Benzene	29	37	16	810
Carbon tetrachloride	11	7	82	-
Dimethyl formamide	71	3	20	6
Ethanol	10	6	7	176
Ethyl acetate	30	47	20	3
Isopropanol	14	17	17	745
Methanol	31	45	14	64
Methylene chloride	53	5	20	22
Solvent B (hexanes)	29	2	69	-
Toluene	31	14	26	29
Xylene	6	19	70	5

5. ETP PLANTS IN PHARMACEUTICAL INDUSTRIES

The ETP plants are used widely in pharmaceutical industry to remove the effluents from the bulk drugs. During the manufacturing process of drugs, varied effluents and contaminants are produced. The effluent treatment plants are used in the removal of high amount of organics, debris, dirt, grit, pollution, toxic, non toxic materials, polymers etc. from drugs and other medicated stuff. The ETP plants use evaporation and drying methods, and other auxiliary techniques such as centrifuging, filtration, incineration for chemical processing and effluent treatment. The treatment of effluents in pharmaceutical industry is essential to prevent pollution of the receiving water. The effluent water treatment plants are installed to reduce the possibility of pollution, biodegradable organics if left unsolved, the levels of contamination in the process of purification could damage bacterial treatment beds and lead to pollution of controlled waters. Waste water disposal standards are shown in table 3 (DoE, 2008).

6. EVALUATION OF IMPACTS ON PHYSICO-CHEMICAL ENVIRONMENT

The major problem associated with the pharmaceutical industries is the disposal of liquid waste. To obtain laboratory data, effluent water sample was collected by use of a water can from the outlet drain of pharmaceutical industry. All possible efforts were made to minimize the time lag between collection and analysis so that no significant change may occur in the quality of the samples. The concentrations of various physicochemical properties of effluent of the study area were determined through extensive laboratory analysis. For carrying out the production, the standard for air and noise quality of the environment shall be determined in accordance with standard specified in schedule 2 and schedule 4 in the Environment Conservation Rule 1997, complied by DoE, ministry of environment and forest, GoB. Schedule 2 and schedule 4 are presented in the table 4 (ECR, 2006) and table 5 (ECR, 1997). Regarding air quality of Gazipur area, that all the parameters (PM_{10}, SPM, SO_2, NO_X) are shown in table 6 (AECL, 2010). The noise analysis of Gazipur area has been shown in table 7 (AECL, 2010). The levels of pollution of the effluents were determined by comparing the observed values of the various parameters with the inland surface water standard value recommended by DoE. The concentrations of various physicochemical properties of effluent of the study area of Healthcare Pharmaceuticals Ltd. were determined through extensive laboratory analysis and are shown in table 8. Digital pH meter was used for the determination of pH of the samples. Two buffer solutions containing pH 4.0 and 7.0 were used to calibrate the digital pH meter. The concentrations of dissolved species were determined by DR-4000 spectrophotometer. BOD_5 was measured by incubation in the dark at 20^0C for 5 days. Total dissolved solid was dried to a constant weight at 105^0C. In summary, testing for dissolved solid involves weighing a clean beaker to the nearest mg. Fill the beaker with test water and evaporate off the water. Weigh the beaker again with the resulting residue. Then subtract the two results to determine the amount of milligrams of residue per liter of water. The levels of pollution of the effluents were determined by comparing the observed values of the various parameters with the inland surface water standard value recommended by DoE. Our sample source was the treated effluent water and the test results were found within the standard limits of DoE. The water test report shows no vulnerable impacts of water quality for Healthcare Pharmaceuticals Ltd. due to their proper use of ETP.

Table 3: National standard for waste water disposal

Parameter	**Unit**	**Location of final disposal**		
		Inland surface water	**Public sewer**	**Irrigated land**
Ammonia(free ammonia)	mg/l	5	5	15
Ammoniacal nitrogen (as N)	mg/l	50	75	75
Arsenic (As)	mg/l	0.2	0.5	0.2
Bod_5 20°c	mg/l	50	250	100
Boron (B)	mg/l	2	2	2
Cadmium (Cd)	mg/l	0.05	0.5	0.5
Chloride(CR)	mg/l	600	600	600
Chromium (hexavalent Cr)	mg/l	0.1	1.0	1.0
Chromium(total Cr)	mg/l	0.5	1.0	1.0
COD	mg/l	200	400	400
Copper (Cu)	mg/l	0.5	3.0	3.0
Cyanide(CN)	mg/l	0.1	2.0	0.2
Dissolved Oxeyzen (DO)	mg/l	4.5-8	4.5-8	4.5-8
Dissolved phosphorus (P)	mg/l	8	8	10
Electrical Conductivity	µMho/cm	1200	1200	1200
Fluonde (F)	mg/l	7	15	10
Iron (Fe)	mg/l	2	2	2
Lead (Pb)	mg/l	0.1	0.1	0.1
Manganese (Mn)	mg/l	5	5	5
Mercury (Hg)	mg/l	0.01	0.01	0.01
Nickel (Ni)	mg/l	1.0	1.0	1.0
Nitrate (N molecule)	mg/l	10.0	Undetermined	10.0
Oil and Grease	mg/l	10	20	10
PH		6-9	6-9	6-9
Phenol compounds C_6H_6OH	mg/l	1.0	5	1
Radioactive Materials	As determined by Bangladesh atomic energy commission			
Selenium (Se)	mg/l	0.05	0.05	0.05
Sulfide (S)	mg/l	1	2	2
Temperature – summer	°C	40	40	40
Temperature- winter	°C	45	45	45
Total Dissolved Solids (TDS)	mg/l	2100	2100	2100
Total kjeldahl nitrogen (N)	mg/l	100	100	100
Total suspended solids (TSS)	mg/l	150	500	200
Zinc (Zn)	mg/l	5.0	10.0	10.0

Table 4: Bangladesh standards for ambient air

Location	**Unit**	**SPM**	**SO_2**	**NO_X**
Industrial and mix area	µg/m^3	500	120	100
Commercial and mix area	µg/m^3	400	100	100
Residential area	µg/m^3	200	80	80
Sensitive area	µg/m^3	100	30	30

7. ENVIRONMENTAL IMPACT ASSESSMENT (EIA)

The United Nations environment programme defined EIA as a method “To identify, predict and to describe in appropriate terms the pros and cons of a proposed development. To be useful, the assessment needs to be communicated in terms understandable by the community and decision makers and the pros and cons should be identified on the basis of criteria relevant to the countries affected. There are two main objectives of EIA:

- Have two major themes in common that EIA is a planning tool and is concerned with identifying, predicting and assessing.
- Safeguard the environment and for that matter the major aim of EIA is to improve decisions on development by increasing the quality and scope of information on likely impacts presented to the decision makers and the public.

Table 5: Bangladesh standards for noise

Location category	Standards determined (dBa)	
	Day	Night
Silent zone	50	40
Residential area	55	45
Mixed area	60	50
Commercial area	70	60
Industrial area	75	70

Table 6: Ambient air quality test report of Healthcare Pharmaceuticals Ltd.

Sl No	Sample location	Ambient air pollution concentration in µg/m^3			
		PM_{10}	SPM	SO_2	NO_X
01	Project West South Corner	77	287	29.37	57.39
Duration (minutes)		480	480	480	480
DoE Bangladesh standards (for ambient air)		150	500	120	100
Method of analysis		Gravimetric	Gravimetric	West-geake	Jacob and hochhelser

Table 7: Ambient noise quality test report of Healthcare Pharmaceuticals Ltd.

Sl no	Location	Site condition	Concentration present (leq)
			Day time
01	Project west side (inside boundary area)	Running condition	55.01 dBA
02	Project east side (inside boundary area)	Running condition	54.37 dBA
03	Project north side (inside boundary area)	Running condition	53.18 dBA
04	Project south side (inside boundary area)	Running condition	56.21 dBA
DoE Bangladesh standard (commercial area)			70 dBA

Table 8: Water Quality test report of Healthcare Pharmaceuticals Ltd.

Water Quality	Unit	Concentration present before treatment	Concentration present after treatment	DOE Standards (for Inland Surface Water)
pH	-	6.8	7	6-9
TDS	mg/l	480	460	2100
TSS	mg/l	62	54	150
BOD_5	mg/l	64	24	50
COD	mg/l	158	68	200

Considering the situation prevailing in the country, a simple methodology was taken for Environmental Impact Assessment of Pharmaceuticals industry .The methodology used for EIA due to 'Pharmaceuticals industry' is Environmental evaluation system (EES). Relative importance of the parameters is selected based on 'LGED guideline, 1992'. The environmental parameters are grouped into 4 categories which are: Physical resources, Ecological resources, Human use values and Quality of life values. The environmental impact was assessed by Environmental Impact Values (EIVs) which may be defined mathematically as equation (1):

$$\mathrm{EIV} = \sum_{i=1}^{n} Vi\ Wi \qquad (1)$$

Where V_i is the relative change in the value of environmental quality of parameter i with respect to existing situation. W_i is the relative importance or weight of parameter i, and n is the total number of environmental parameter related to the project. The computation of Environmental Impact Value (EIV) of Pharmaceuticals industry needs determination of V_i, the value representing the magnitude of alteration of the environmental parameters, and W_i, the value representing relative weight or importance of the respective parameters. Magnitude of Environmental parameters: The beneficial and adverse changes in environmental parameters resulting from Healthcare Pharmaceuticals Ltd. and industrial zone adjacent to Beximco Pharmaceuticals Ltd., usually expressed in qualitative terms are plotted in a scale to quantify the environmental alterations in table 9 and table 10.

Table 9: Environmental Impact Assessment (EIA) of Healthcare Pharmaceuticals Ltd.

Environmental Parameters	**Relative importance value**	**Degree of impact**	**Relative impact**		**EIV**
			POSITIVE	**NEGETIVE**	
Physical resources					+8
Surface Water	3	+1.4	+4.2		
Air & Sound Pollution	2	+1.2	+2.4		
Soil Quality	5	+0.4	+2		
Climate	3	-0.2		-0.6	
Ecological Resources					+6
Fisheries	10	+0.6	6		
Forest	5	0		0	
Human use value					+9.2
Agriculture	5	+0.4	+2		
Sewage and waste disposal	5	+0.8	+4		
Land use	2	+0.6	+1.2		
Drainage	5	+0.4	+2		
Quality of life values					+16.2
Agricultural land loss	8	-1.2		-9.6	
Socio – economic	8	+1.8	+14.4		
Public health	6	+1	+6		
Resettlement	3	+1.8	+5.4		
Total Environmental Impact Value (EIV)					**+39.40**

Table 10: EIA of industrial zone adjacent to Beximco Pharmaceuticals Ltd.

Environmental Parameters	**Relative importance value**	**Degree of impact**	**Relative impact**		**EIV**
			POSITIVE	**NEGETIVE**	
Physical resources					-13.5
Surface Water	3	-1.2		-3.6	
Air & Sound Pollution	2	-0.8		-1.6	
Soil Quality	5	-0.7		-3.5	
Climate	3	-1.6		-4.8	
Ecological Resources					-10
Fisheries	10	-0.6		-6	
Forest	5	-0.8		-4	
Human use value					-17.7
Agriculture	5	-1.1		-5.5	
Sewage and waste disposal	5	-0.5		-2.5	
Land use	2	-1.1		-2.2	
Drainage	5	-1.5		-7.5	
Quality of life values					-1.8
Agricultural land loss	8	-2.2	+15.2	-17.6	
Socio – economic	8	+1.9	+7.2	-6.6	
Public health	6	-1.1			
Resettlement	3	+2.4			
Total EIV					**-43**

The adverse changes have been given values -1, -2, -3, -4 and -5 to represent very low, low, moderate, high and severe negative impacts respectively. Similarly +1, +2, +3, +4 and +5 represent very low, low, moderate, high and very high positive impacts respectively. A value from the scale representing effect of the project on each parameter was taken to compute the EIV of the pharmaceuticals industry.

At first the values indicating magnitude of environmental changes influenced by the pharmaceutical were placed in the appropriate columns in Table 9 and Table 10, then multiplied them to obtain positive and negative impact of the parameters. Finally all these impacts were summed up to obtain the total EIV of -43 for industrial zone adjacent to Bexmico Pharmaceuticals Limited and +39.40 for Healthcare Pharmaceuticals Limited. As the water test report shows the water quality parameters are within standard limits for Healthcare Pharmaceuticals Limited, the overall EIA report also shows positive impacts upon all environmental sector. But in case of industrial zone adjacent to Bexmico Pharmaceuticals Limited the negetive EIV value represents mainly all the adjacent industries rather than Beximco Pharmaceuticals Limited.

8. RECOMMENDATIONS

The result of EIV shows that from both of the survey there is a visible difference in impacting. Most of the pharmaceutical industries accommodate in cluster and there is a association of those other industries regarding the combined environmental contribution. So for industries like pharmaceutical company, it may run more or less safely but the overall contribution which is so high due to the all industries that the adverse impacts and prevention is very much in need to regulate and reduce the environmental hazards in a moderate level**.** That's why we measured the effects based on the overall impacts of the adjacent area of the industries.

8.1 Adverse Effects and Impacts

i. Due to the bad effluent discharge in the water body the water quality has been decreased for aquatic lives.
ii. The quality and color and odor of water is deteriorated so much that people adjacent to the Turag River are unable to use the water.
iii. Fish production in the river is being severely reduced due to the behavior of industries and after the industrialization
iv. The previous green land is being converted to the industry which caused the greener condition more vulnerable and it destroyed the habitat of various lives of the ecosystem.
v. Also the clearing of Greenland made the environment unhealthier for the purpose of industry and different processes which eventually reduced the required oxygen of the surrounding.
vi. Now very little agriculture is going on due to the high land value and it eventually reduces the total contribution of the green field in to the green environment.
vii. The reduction of agriculture is one of the most important impacts of the industrialization of the area.
viii. People has obviously lost some of the agricultural lands which was their source of harvest previously but now it is being highly demandable due the surrounding land use of the industry .
ix. Drainage congestion and water logging is the major concern of the area and in the rainy season it becomes more adverse than ever.
x. It also causes the spreading of the nuisance and insects which causes the spreading of seasonal diseases.
xi. Also it causes the vehicular as well as transportation problem at rainy time.
xii. Industrial process of different plants causes problem in different ways. The sound is causing a great problem to the neighbor of the industries.
xiii. The temperature of the surrounded environment is being increased due to the different processes.
xiv. The public health of the area is being deteriorated overall after the industrialization because of the increase of seasonal diseases.

8.2 Mitigation Measures

Despite of having some adverse impact of pharmaceutical industry it cannot be stopped. Rather we should find some mitigation measures to minimize the adverse impacts and increase the beneficial impacts. To do so the following steps should be taken:

i. Every industry should have sufficient land to have their process flow safe and sound to prevent any damage.
ii. Every ETP should be run on the regular basis to ensure the proper treatment of water.
iii. For the unprocessed emission of the smoke and gases there should be proper exhaust system.
iv. Needs to use canopy on the generators to reduce sound and other than that should be punished which will enforce the law.
v. To ensure that the regulatory body of DoE should be strengthened and monitored regularly.
vi. The present condition of the water body needs to be normalized by using technology or needs to be waited because it is needed for the people and also for the aquatic life.

vii. Needs to restore alternative habitat for endangered species.
viii. Needs to replace trees along the free spaces.
ix. Needs the proper drainage of the industrial processed water through a dedicated line to the discharge.
x. The drainage of the municipality should be improved for the industry with proper monitoring and maintenance.
xi. The industrial roads should be made in proper estimation to prevent the decay and water clogging on the road.
xii. Agricultural lands shouldn't be given permission for any kind of production.
xiii. Human living should be transferred on to another place distant from industry to keep them safe from the adversary.
xiv. There should be a mandatory plan for the flood management to mitigate the unnecessary water to be a hassle for the production.
xv. There should be an estimation of how much industries should be given permission to open their production on a particular area based on the land available.

9. CONCLUSION

One of the most important industries that ensures the most part of our countries medicine requirement is pharmaceutical industries. This industry is very importantly in need to maintain the environmental safety due to its product vulnerability and health safety. The waste for which we are concerned about is a real subject for our country's perspective where waste minimization is a real challenge. It is clear that pharmaceuticals have a major impact on the nature, particularly capable of affecting host immunology and physiology leading to changes in susceptibility and associated pathological symptoms. Now a real case scenario is a large number of people lives on this industry by their earning the livelihood. So despite this environmental negative result which concerns us a lot, never makes so importantly bad to them as long their standard of life and expectancy is concerned. So considering all this objectives, proper study of the peoples need and also the regularity implication to force the industrial authority to manage the environmental damage made by the industry is now a demand of age and future.

ACKNOWLEDGMENT

The authors would like to thank Md. Halimuzzaman (Executive Director at Healthcare Pharmaceuticals Ltd.), Dr. Kazi Reshad Agaz (Manager, MD at Beximco Pharmaceuticals Ltd.) and local people of Gazipur and Rajendrapur area for their supports.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

NON-DESCRIPT LANDS IN INDIA: AN ALTERNATIVE ECOSYSTEM FOR SEED PRODUCTION

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ABSTRACT

The mass or place of lands that are kept out of conventional use of economic productivity or somehow overlooked for a protractile period, having high level of floral and faunal diversity can be brand as non-descript land. As high has 5-6% land in India are falling under this category. While land- man ratio is sequencing very faster the rate of 1: 0.8 (ha) at national level and 1: 0.04 (ha) in WB, exploration of new opportunities is the utmost task. These goners of lands are representing long history of agro-economic and meteorological characteristics. Non-descriptive lands viz, nayanjuli, bandh, tanr, bill, rooftops, etc are sustaining huge floral and faunal diversity including rich Genetic Diaspora. These huge land masses can be effectively utilized for seed production of several horticultural crops like turmeric, ginger, seed spices, coconut, arecanut, elephants foot yam, colocasia, betel vine, pepper, cashew which require minimal input lesser management. These virgin lands have enough potential to provide a soil rich in plant nutrients along with naturally modified microclimates like partial shade, soil moisture, humus and organic substances. The soil may also be less prone to erosion, absolutely chemical-free and provide the crop all the situations towards sustainable production. The paper entails these nondescript lands for horticultural seed production in detail.

Key Words: *Non-descript land, diaspora, climate chage, global warming, floral diversity and faunal diversity.*

1. INTRODUCTION

Seed is the basic and vital input in agriculture and in all farming systems. The strength and health of an agricultural economy is ensured by the timely availability of quality seed in adequate quantities. Sustained increase in agriculture production and productivity necessarily requires continuous development of quality seed and their efficient system of production and supply to the farmers.

Despite a organized efforts for seed production created by the different sectors, availability of good quality seeds continues to be a problem for the farmers. As a result, they prefer to rely on their own seeds. Seed replacement rate continues to remain in the range of 2-10 percent in certain states for certain crops, which is much below the desired level of 20 percent for most crops. As per ICAR report (1), quality seed production in India has increased from 2.92 lakh quintals to 8.55 lakh quintals, under horticultural crops production of saplings/ planting materials including tissue culture plantlets enhanced from 51.16 lakhs to 285.33 lakhs during 2007- 08. But the expansion in area under different crops which hitherto was responsible for increase in production during last two decades may be hindered due to non availability of cultivable land for further expansion. Hence to achieve four per cent growth in agriculture, the most suitable alternative left was the vertical growth in the production, which is possible only through use of quality seeds of proven varieties/hybrids. The problem of adequate land may also be resolved by searching and utilizing the non-descript land which is generally not under cultivation or not given due attention so far.It has been proved beyond doubt that the quality seed of improved varieties can alone increase the crop yields to the extent of 25-30 percent over the old and obsolete varieties. This is possible by exploring new areas and situations in making the unconventional lands suitable for seed production under continuous management. Thereby the conventional land can only be utilized as the main field for cultivation of crops. The so called non descript land may be very much effective in producing seed for their nutrient rich soil, specialized microclimates, by default organic in nature, etc. It is also well recognized that the production of quality seed meeting the prescribed standards of genetic and physical purity, seed health, vigour, viability and storability are specialized and scientifically based activities requiring focused attention.

2. OBJECTIVES

1) To generate concept on *non-descript lands* in terms of ecological features
2) To examine the *social taxonomy* of this unique piece of land
3) To elucidate the economy and ecology of seed production in *non-descript lands*

3. MATERIALS AND METHODS

The following methods are followed to generate classified information on the non-descript land.

- Focus Group Discussion (FGD)
- Local History
- Ethnography
- Time Line Analysis
- Experts' Triangulation
- Nomenclatures and Social Taxonomy
- Analysis and Extrapolation

4. RESULTS AND DISCUSSION

Classification of Non Descript Land in West Bengal, India

In West Bengal, net sown area is 62.48% of total reported land area(the corresponding) figure for India is 46.06%); 13.48% is under forest; 0.67% is under miscellaneous trees and groves not included in net sown area; and 18.52% is under non-agricultural use. The total land use in the broad categories of barren and unculturable land; permanent pasture and other grazing land; fallow land other than current fallow; and culturable wasteland (which is 4.31% for India as against 0.40% for West Bengal), together add up to 1.02% of the total reported area (2).

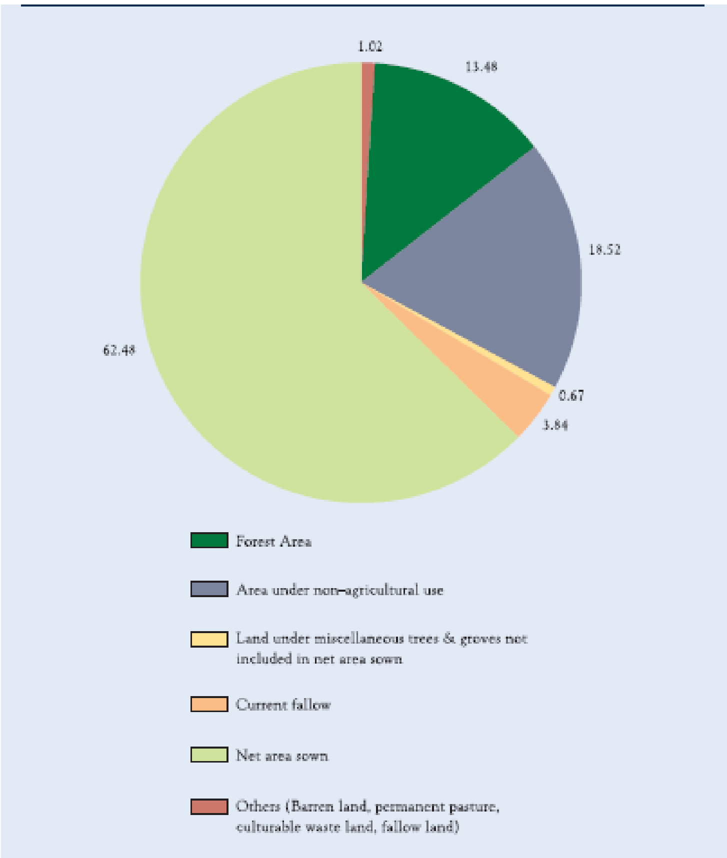

Fig 3: Land use pattern in West Bengal-Percentage Distribution

1. Current Fallow
2. Fallow Land other than Current Fallow
3. Culturable Waste
4. Misc. Tree Crops & Groves
5. Permanent Pasture & Other Grazing Land
6. Barren & unculturable Waste
7. Land put to Non-agricultural Users
8. Forest

Table 1: Classification of Land in West Bengal (In Thousand Hectares)

District	Net Cropped area	Current Fallow	Fallow other than cur fallow	Cultivable waste	Misc Tree Crops & groves	Perm Past & Other Graz lands	Barren & Uncultivable waste	Land Put to Non ag waste	Forest
A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**
West Bengal*	8687.52	333.37	22.12	34.47	57.87	4.69	27.05	1608.99	1171.29
Burdwan	698.74	8.04	3.33	9.84	3.01	0.63	2.38	182.62	22.26
Birbhum	451.12	26.02	2.61	2.29	0.75	0.31	0.40	90.81	15.85
Bankura	688.09	41.12	1.49	3.71	0.88	0.51	2.37	142.19	147.69
Midnapore East	396.59	0.78	0.15	0.06	3.90	0.07	1.74	91.70	0.90
Midnapore West	928.58	29.98	3.82	4.19	6.64	0.65	2.44	146.07	169.69
Howrah	138.68	4.92	1.11	0.13	0.74	0.14	0.76	43.77	....
Hooghly	312.22	0.84	0.14	1.62	2.30	0.11	0.97	80.54	0.53
North 24-parganas	386.52	2.40	....	....	8.71	....	....	117.96	....
South 24-parganas	953.37	13.66	0.21	0.45	2.76	....	0.59	125.95	426.30
Nadia	360.65	3.94	0.04	0.58	2.64	0.10	0.20	74.71	1.22
Murshidabad	532.50	1.82	0.40	0.82	2.00	0.04	2.03	120.28	0.77
Uttar Dinajpur	312.47	4.47	0.62	0.12	3.23	0.09	0.22	30.56	0.58
Dakshin Dinajpur	221.91	1.15	0.09	0.02	0.67	....	0.21	25.61	0.93
Malda	371.05	49.65	0.30	0.09	3.01	....	....	84.06	1.68
Jalpaiguri	622.70	21.46	0.08	0.78	5.07	....	2.41	76.44	179.00
Darjeeling#	325.47	11.15	3.80	1.38	1.94	0.91	4.92	33.79	124.58
Cooch Behar	331.38	1.26	0.24	1.41	8.97	0.48	1.10	56.66	4.26
Purulia	625.48	110.71	3.69	6.36	0.82	0.65	4.31	84.75	

Source: West Bengal Agricultural Marketing Board

Note: #Siliguri Sub- Division only; *Excluding Kolkata; A=Reported area; B=Current Fallow; C=Fallow other than current fallow; D=Cultivable wasteland; E=Miscellaneous tree crops and groves; F=Permanent pasture and other grazing lands; G=Barren and uncultivable wasteland; H=Land put to non agricultural wasteland; I=Forest

The potential crops for seed production:

A. Vegetables: Many common vegetables like brinjal, tomato, chilli, French pean, peas, dolichos bean, basella, palak, elephants foot yam, colocasia, etc can be grown in non-descript land areas for seed production. Besides some unusual vegetable crops like broccoli,Chinese cabbage, asparagus, celery, parsley, leek, baby corn, cherry tomato, etc,can also be tried with suitable management practices. A study conducted by Suman (4) revealed that only 13 per cent farmers produced the seeds of pea, french bean, palak, okra for own use for next season. 16.67 per cent farmers were satisfied from the income of vegetable production.

B. Spices: Spices are emerging crops acclaimed not only in India, but also throughout the world. However, to promote the seed requirement by the farmers due care is to be taken in West Bengal areas for seed production of several spices especially pepper, turmeric, ginger, seed spices like coriander, fenugreek, black cumin, dill, etc .

C. **Plantation Crops:** West Bengal is endowed with a vast area of land having differential land and agro-climatic situations. To grab this unique opportunity several plantation crops can be grown particularly to supplement the huge requirement of seed and quality planting materials to the growers. These plantation crops include coconut, arecanut, cashewnut, betelvine, etc

D. **Other crops:** Several other crops like flowers and ornamentals, medicinal and aromatic crops, forest tree crops etc can also be grown in the non-descript land situations for seed production.

Some typologies of NDL

Sl No	***Local name***	***Type***	***Elements***
1	Nayanjuli	Marginal aquatic body by the roadside bunds	Perennial to seasonal water Availability of local fish species Water weeds, snails and water chestnuts.
2	Smashan (Burning Ghat).	Wasteland generally associated with watering.	Generally raised land with local trees, herbs, shrubs, weeds, away from community settlement.
3	Traditional and cultural water bodies Eg. Dighis and Bunds	Traditional water bodies with raised embankments with huge floral and faunal diversity.	Local icthyofaunal diversity, minor fruits, trees, rare type of waterweeds and birds.
4	Bhusundir Math	Abandoned lands with beliefs, taboos and far-away watching	Occasionally tried with field crops, local pulses and social forestry, rich with NPK status and traditional biotypes.
5	Open roof tops: Khola Chhad	Open terrace 10-100ft above ground level, closer to canopy meteorology 3-4 lakhs ha of open terrace in WB transformed landmass.	Domestic grooming on movable roots, plastic house with rain water harvesting or municipal water supply.
6	Basra	A piece of land not used for cultivation and used only for keeping waste materials of farm	Post harvest operations for value addition, extracting marketable products and spending recess hours
7	Bali khadan	A piece of land or stretch of land used for extraction of sands for construction and other civil works	Commercial extraction of sands, destruction of natural topography, livelihood generation
8	Brick kiln field	The land area where the bricks are manufactured	Conversion of natural lands into hard and non-recyclable masses
9	Char	The area emerged from the river	Rich alluvial soil, remains fertile for up to 6-8 months

Fig 1: The paradigmatic Structure of non-descript land (NDL)

- Economic viability may be impaired by socio-cultural issues.
- Around these lands, there a lot of taboo, prejudices and ritual barrier to invite modern technology in this piece of land.
- Policy barrier: A bane to adopt the land for conventional Agriculture.
- Technology Non-availability:
- Need based research vs. Subjective research
- Alternative Technology vs. General Technology

SUCCESS STORY OF NDL UTILIZATION AT BCKV FARM:

In Bidhan Chandra Krishi Viswavidyalaya, Mohanpur, Nadia, West Bengal, India (State Agricultural University) a sizeable area of land in different farms under Directorate of Farms has been utilized for seed production of various horticultural crops like turmeric, ginger, elephant's foot yam, colocasia, coriander, black pepper etc. It has been found that with minimal inputs and management conditions, a good harvest of optimum quality is possible from the non descript lands. The following table will summarize the out put of seed production during 2010-11:

Sl No	***Crop***	***Botanical name***	***Area under seed production (m^2)***	***Production (Kg)***
1	Turmeric	*Curcuma* sp	1. 1530 2. 754 3. 750	508 260 246
2	Ginger	*Zingiber officinale*	1050	300
3	Elephant's foot yam	*Amorphophalus campanulatus*	700	1083
4	Colocasia	*Colocasia esculenta*	650	260* 230**
5	Coriander	*Coriandrum sativum*	600	
6	Black pepper	*Piper nigrum*	Per plant basis	2

Solakochu, **Loti

The Problems Ahead and Strategies to Overcome:

1. Entitlement is a problem: Ownership to this land is mostly community-based, sometimes private, and other times it goes controversial.
2. Research inputs are very poor on the land utilization technology for this ND lands.
3. The quality of the lands in terms of NPK status, run-off character, marketability, participatory micro planning are not endeavored yet.
4. Social access to the economic production is not customized.
5. Lands are sometimes very difficult to access.
6. Beliefs and taboos related with these piece of land may create some hindrance towards opting for economic activities.
7. Absence of proper State Policy is one of the major problems.

Peri-urban social ecosystem: An emerging trend across India

Of late, peri-urban agriculture is gaining attention for its potential role in increasing food security, employment and income generation, poverty alleviation, community resource development, waste management and environmental sustainability by utilizing the nondescript lands including that of the rooftops. A large part of the world population is living in peri-urban areas. The rich social and economic variability of peri-urban areas offers opportunities for the promotion of farming including seed production and thereby keeping harmony both between society and nature and between groups of people(5).

Strategies and Recommendations:

1. Government should come up with proper policy including land bank creation and entitlement decision.
2. IPCC should incorporate ND lands as a strategic intervention for combating global worming and climate change.
3. University research programmes should be undertaken.
4. Agricultural research should reset its destination by incorporating NDL.

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PHYSICO-CHEMICAL CHARACTERISTICS OF SURFACE WATER IN THE MAGURA MUNICIPALITY AREA, BANGLADESH

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ABSTRACT

A study was conducted to determine the physico-chemical parameters of surface water and to evaluate their suitability for various purposes in the Magura municipality area of Magura district. For conducting this study, surface water samples were collected from twenty four pond water and three river water samples during both dry and wet seasons of the year 2011. Collected samples were analyzed by using the procedure outlined in the standard methods. In the study area, the average values of physico-chemical parameters for pond water are pH:6.33, DO:4.66 mg/L, EC:501.79 µS/cm, Na^+:37.64 mg/L, Ca^{2+}:29.02 mg/L, HCO_3^- :140.66 mg/L, and these values in river water are pH:6.56, DO:5.67 mg/L, EC:522 µS/cm, Na^+:38.47 mg/L, Ca^{2+}:29.35 mg/L, HCO_3^- :157.36 mg/L, during dry season and in wet season these values are pH:6.80, DO:3.59mg/L, EC:406.17 µS/cm, Na^+:27.62 mg/L, Ca^{2+}:25.06 mg/L, HCO_3^- :150.83 mg/L in pond water and pH:6.78, DO:4.4 mg/L, EC:451 µS/cm, Na^+:32.98 mg/L, Ca^{2+}:25.89 mg/L, HCO_3^- :169.03 mg/L in river water. Results of this research do not identify any toxic pollution but identify the problem areas in respect of fecal pollution. This study reveals that the surface water in the study area is unsuitable for drinking and domestic purposes, but excellent for using in irrigation purposes.

Keywords: *Surface Water, Physicochemical Properties, Water Quality*

1. INTRODUCTION

Water is a most valuable resource to human and living things, essential for the sustenance of life on earth as exemplified by its diversified uses. Declining water quality has become a global issue of concern as human populations grow, industrial and agricultural activities expand, and climate change threatens to cause major alterations to the hydrological cycle. Surface waters are most vulnerable to pollution due to their easy accessibility for disposal of wastewaters. Extensive withdrawal of ground water for different purposes is lowering the water table in the aquifer, and the ground water level has gone down drastically. On the other hand, Arsenic contamination in ground water has almost become a calamity especially in Bangladesh. About 97,802 people lived in Magura municipality and its area is 47.31 sq. km. About 114 pond water bodies and 1 river are present in this area. The people of Magura Municipality are fully dependent on groundwater for their drinking, domestic and other purposes. The population of this area is increasing day by day. Due to the growth of population in municipality area, the rate of extraction of ground water from the sources has been exponentially increased and as a result the ground water level is lowering in the aquifer. Arsenic contamination of ground water is another problem for municipal dwellers to use the ground water for various purposes. In 1998, British Geological Survey (BGS) collected 2022 water samples from 41 arsenic-affected districts. Laboratory tests revealed that 19 Percentage of groundwater from the shallow aquifer (less than 150 m deep) exceeding the Bangladesh standard for arsenic of 0.05 mg/L in Magura district (Smith *et al.*, 2000). To avoid arsenic contamination, municipal dwellers use surface water for various purposes in their daily life is an alternative option. On the other hand, surface water bodies in this area become contaminated by organic manure, municipal waste, some fungicides, agricultural runoff and industrial effluent etc. Since contaminated water can pose a potential source of health risk to municipality dwellers. So, Magura municipality faces an acute shortage of safe water. If the surface water can be used for various purposes then the problem will be reduced and then that will help to reduce pressure on ground water with supply water demand can be ensured. Measurements of various surface water quality parameters play the key role in detecting the status of pollution and suitability of a particular water body for various purposes. So, at the present situation physicochemical characteristics of surface water in the magura municipality area is very much important. This study determines the present surface water quality in the Magura municipality area and to evaluate the surface water quality to be used for various purposes.

2. METHODOLOGY

2.1 Sample Collection and Analysis

Surface water samples from different ponds and the river were collected for this study. Sample collection points are shown in Figure 1.

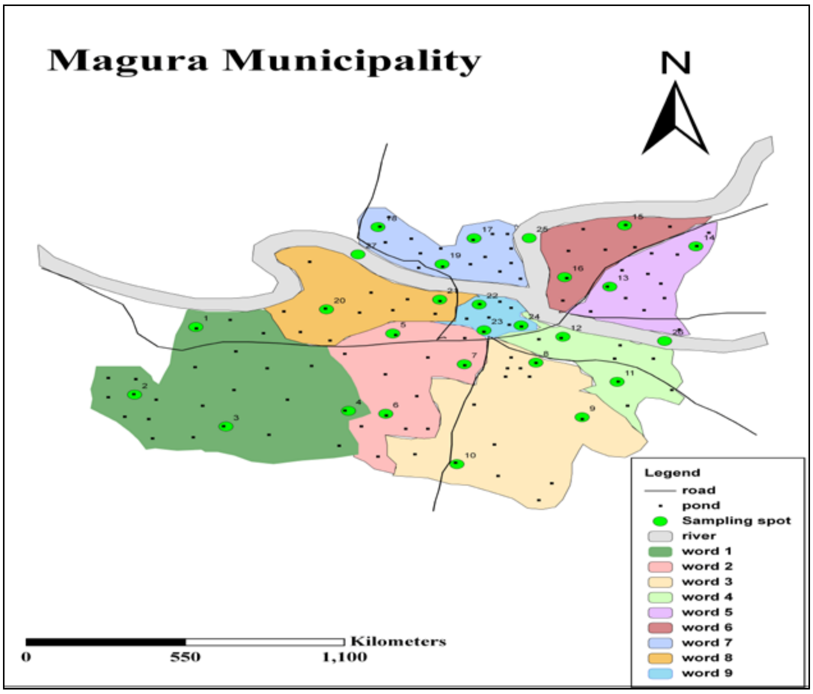

Figure 1: Sampling location map of the surface water in the study area.

Surface water samples from different ponds and the river were collected for this study. Samples were collected in plastic bottles cleaned by rinsing thoroughly with 8M HNO_3, followed by repeated washing with distilled water. Multiple samples were collected from the same spot at two seasons to study the seasonal variation of the results. Three to four sub samples of equal volume were collected from different position in each sampling spot. Water samples were collected from 1m below of the surface water layer. The sub samples of each sampling spot were mixed well and a sample of 2 L was collected in sampling bottles and then securely sealed with proper leveling (sample number and location). Aeration during sampling was avoided as far as possible. Twenty four pond water samples and three river water samples were collected for this study in each season. Winkler's bottles were used to collect samples for BOD measurements and water samples for Iron measurement were collected in plastic containers acidified with 4-5 drops of HNO_3 (0.01N) to prevent precipitation. Temperature, Transparency, pH, Electrical conductivity (EC), Total dissolved solids (TDS), Dissolved oxygen (DO) were measured at the sampling site. Other parameters were analysed immediately after sample collection at the laboratory. For chemical analysis, standard preservation techniques and methods (Greenberg, *et. al*.,1992 and Ramesh and Anbu, 1996) of analysis were used. Data were analysed by Excel 2000 software and Minitab pro 16 software was used for statistical analysis. Different parameters of water quality were analyzed by the following methods:

A glass thermometer was used for the measurement of temperature. Transparency was measured by Sacchi disc. pH, EC and TDS were measured at the sampling site by H1-9635, Microprocessor Conductivity/TDS meter. Measurement of DO was done by Dissolved oxygen meter (Model: PDO-519) at the sampling site. Biochemical oxygen demand (BOD) was measured from the differences of initial and 5 days DO by DO meter. Salinity was measured by Conductivity meter (model, HI-8033). Bicarbonate was measured by Potentiometric titration method. Sodium and Potassium were measured by Flame photometric method (Flame photometer- models PEP 7 and PEP 7/C). Calcium and Magnesum were measured by Titrimetric method, Chloride was measured by Mohr's titration method, Ortho-Phosphate and Sulphate were measured by Turbidimetric method (Thermo spectronic, UV-visible Spectrophotometers), Nitrate was measured by Ultraviolet spectrophotometric screening method, Iron was measured by Phenanthroline method (using Spectrophotometer, model-Hach/DR/2400). Coli form was measured by Multiple tube fermentation procedure (Pal, 1994).

2.2 Equations of Data Analysis

Irrigation water quality was judged by some determining factors such as Sodium Absorption Ratio (SAR), Soluble Sodium Percentage (SSP), Residual Sodium Bicarbonate (RSBC), Total hardness (Ht) etc were calculated by using the following recommended relationships.
SAR was calculated by the following equation given by Richards (1954),

$$SAR = \frac{Na}{\sqrt{\left(\frac{Ca + Mg}{2}\right)}} \tag{1}$$

SSP was calculated by the following equation by Todd (1980),

$$SSP = \frac{(Na + K) \times 100}{Ca + Mg + Na + K} \tag{2}$$

The RSBC was calculated according to Gupta and Gupta (1987),

$$RSBC = HCO_3 - Ca \tag{3}$$

The TH was calculated by the following equation by Raghunath (1987),

$$TH = (Ca + Mg) \times 50 \tag{4}$$

Where, all the ionic concentrations are expressed in milli-equivalents per litre (meq/L).

3. RESULTS AND DISCUSSION

3.1 Surface Water Chemistry in Magura Municipality

The surface water chemistry may be derived from nature or anthropogenic sources i.e. waste, excessive fertilizer in crop field, acid precipitation etc. Table 1 and Table 2 represent the physic-chemical properties of the surface water during dry season and wet season in magura municipality where major ions are represent in mg/L.

3.1.1 Physical properties

During dry season, the water temperature of all water samples in the study area ranges from 25.6 to 26.5 ^oC for pond water and 25.4 to 26.9 ^oC for river water. In the wet season, this ranges from 30 to 33 ^oC for pond water and 31 to 32.1 ^oC for river water. In pond water the transparency values in the study area ranges from 25.05 to 30.53 cm and 17.38 to 22.67cm and in river water 34.78 to 37.29cm and 26.48 to 30.75cm during dry season and wet season respectively. The decrease of transparency in surface water may cause to increase the clays and silts from shoreline erosion, re-suspended bottom sediments, organic detritus from domestic, municipal wastewater discharges and surface runoff. Water bodies that have high transparency values typically have good water quality. This study reveals that the pH value ranges from 5.85 to 6.95 for pond water and 6.46 to 6.62 for river water in the dry season whereas in the wet season this ranges from 6.43 to 7.7 for pond water and 6.74 to 6.79 for river water. In the dry season all water sample show slight acidity and in the wet season some sample represent acidity and some show alkalinity. The acidic pH may have resulted from humic acid (HA) formed from decaying organic matter The lowering of pH values indicates anthropogenic pollution. The organic and inorganic substances come into the water from agricultural runoff, domestic wastewater discharge and surface runoff. The electrical conductivity of pond water from 263 to 818 µS/cm and 221 to 724 µS/cm and river water from 511 to 536µS/cm and 446 to 455µS/cm during dry season and wet season respectively. The electrical conductivity of all samples in the study area is lying within standard level of EC of most natural waters. During wet periods, conductivity will decline as the concentration of dissolved ions becomes more dilute, whereas dry periods will lead to increased conductivity values. Municipal, agricultural, and industrial discharges can contribute ions to receiving waters or can contain substances that are poor conductors (organic compounds) changing the conductivity of the receiving waters. Thus, specific conductance can also be used to detect pollution sources.

Table 1. The physico- chemical parameters of the collected water samples from Magura municipality during dry season.

		Physical Parameters										Cations				Anion					Trace
	S/N	pH	EC (µS/cm)	Temperature (°C)	Transparency (cm)	DO (ppm)	BOD (ppm)	TDS (ppm)	TSS (ppm)	TS (ppm)	Salinity (ppt)	Na^+ (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	K^+ (ppm)	PO_4^{3-} (ppm)	HCO_3^- (ppm)	Cl^- (ppm)	NO_3^- (ppm)	SO_4^{2-} (ppm)	Fe (ppm)
Pond	1	6.2	332	25.6	28.2	4.8	0.9	230	90	320	0.2	25.4	21.9	9.3	12.9	0.5	98.3	25.7	4.8	26.9	0.1
	2	6.4	455	25.9	27.5	4.6	0.8	270	120	390	0.3	31.9	27.0	12.3	15.7	2.0	91.1	38.7	7.3	36.5	0.1
	3	6.5	513	25.7	27.9	4.7	1.1	315	130	445	0.3	38.4	30.9	13.5	16.2	0.7	131.9	40.2	5.1	34.7	0.1
	4	6.3	467	25.6	28.8	4.9	1.2	314	110	424	0.3	29.6	29.1	13.9	16.8	2.1	135.8	39.8	5.8	33.5	0.1
	5	7	595	25.9	27.5	4.3	1	354	150	504	0.3	42.7	35	17.0	19.8	2.0	137.9	47.6	4.3	39.5	0.1
	6	6.3	816	26	25.1	3.9	1	533	240	773	0.3	69	40.9	21.6	33.1	0.8	238.5	69.5	7.6	43	0.1
	7	6.4	712	25.7	27.3	4.7	1.6	448	120	568	0.3	53.2	36.0	19.4	26.4	1	203.8	53.7	4.1	40.2	0.1
	8	6.3	519	25.9	28.8	4.9	1.5	352	100	452	0.2	28.4	31	18.5	18.8	0.9	171.1	28.8	3.4	41.9	0.1
	9	6.1	326	25.8	29.0	5	1.1	215	50	265	0.1	19.9	24	8.8	10.8	0.7	92	18.3	3.4	31.1	0.0
	10	5.9	303	26	30.5	5.2	1.2	208	40	248	0.1	22.3	18.0	8.4	10.2	0.6	87.8	20.8	4.8	27.9	0.0
	11	6.3	436	26.2	28.0	4.8	0.9	271	90	361	0.3	31.5	26	12.6	16.1	2.0	97.5	35.7	6.8	33.1	0.1
	12	6.3	480	25.9	26.8	4.6	1	347	140	487	0.2	39.4	30.8	13.2	19.9	1.5	155.6	37.5	8.2	34.3	0.1
	13	6.2	402	25.7	27.5	4.8	1.4	281	110	391	0.2	31.9	26	11.0	17.2	1.3	117.8	30.8	6.8	30.3	0.1
	14	6.3	514	26.1	26.3	5	1.5	335	110	445	0.3	38.1	30.9	14.1	18.1	2.2	152	37.3	4.8	31.4	0.1
	15	6.2	494	25.9	28.5	4.7	0.8	325	80	405	0.2	35.6	30.0	13.8	17.4	1.2	147.7	36.9	4.1	31	0.1
	16	6.0	818	26	27.0	4.1	1	510	190	700	0.4	68.5	40	22.0	30.0	3.4	213.6	74.1	4.9	44.9	0.1
	17	6.3	263	25.8	30.3	5.1	0.9	165	40	205	0.1	15.8	16.0	9.6	8.4	0.8	66.6	13.1	3.0	25.1	0.0
	18	6.4	355	26.3	29.5	4.9	1.1	230	70	300	0.2	23.6	23.1	10.1	12.8	0.9	94.8	26.8	3.5	28.9	0.0
	19	6.2	380	26.5	28.0	4.8	1	240	80	320	0.2	25.0	25	11.0	12.9	1.4	102	27.8	3.4	28.9	0.1
	20	6.6	381	26.2	27.6	4.7	0.8	259	70	329	0.2	24.2	26	13.0	10.2	1.7	117.7	27.7	3.4	27.1	0.1
	21	6.3	468	25.9	29.8	4.7	1	296	90	386	0.2	31.7	28.0	14	16.6	1.5	129.4	33.8	3.7	31.3	0.1
	22	6.6	709	26.3	25.6	4	1.1	451	200	651	0.3	63.9	34.9	16.1	27.9	2.1	189.7	66.8	6.7	36.8	0.1
	23	6.4	615	25.8	26.8	4.2	1.2	428	180	608	0.3	55.4	32.0	17.2	24.5	1.3	200.3	53.5	3.7	33.8	0.1
	24	6.5	690	26	27.1	4.4	1.2	445	180	625	0.3	58.2	34	18.4	25.9	1.5	203.1	59.7	3.4	35.5	0.1
	Ave	6.3	501.8	26	27.9	4.7	1.1	325.9	115.8	441.8	0.2	37.6	29	14.1	18.3	1.4	140.7	39.4	4.9	33.7	0.1
	Max	7	818	26.5	30.5	5.2	1.6	533	240	773	0.4	68.9	40.9	22.0	33.1	3.4	238.5	74.1	8.2	44.9	0.1
	Min	5.9	263	25.6	25.1	3.9	0.8	165	40	205	0.1	15.8	16	8.4	8.4	0.5	66.6	13.1	3.0	25.1	0.0
	Stdev	0.2	157.4	0.23	1.4	0.3	0.2	99	52.4	148.5	0.1	15.7	6.2	3.9	6.6	0.7	47.4	16.3	1.6	5.4	0.0
River	25	6.5	536	26.9	35.5	5.7	1.3	343	470	813	0.3	39.4	31	14.8	19.2	1.3	154.8	37.3	4.1	35.1	0.2
	26	6.6	519	25.4	34.8	5.9	1.2	337	440	777	0.2	37.9	29.0	14	18.4	1.2	160.4	32.2	3.3	31.1	0.2
	27	6.6	511	26.1	37.3	5.4	1.3	331	490	821	0.2	38.1	28.0	14.2	18.3	0.9	156.8	33.8	3.6	30.7	0.1
	Ave	6.6	522	26.1	35.9	5.7	1.3	337	466.7	803.7	0.2	38.5	29.4	14.3	18.6	1.1	157.4	34.4	3.7	32.3	0.1
	Max	6.6	536	26.9	37.3	5.9	1.3	343	490	821	0.3	39.4	31	14.8	19.2	1.3	160.4	37.3	4.1	35.1	0.2
	Min	6.5	511	25.4	34.8	5.4	1.2	331	440	777	0.2	37.9	28	14	18.3	0.9	154.8	32.2	3.3	30.7	0.1
	Stdev	0.1	12.8	0.75	1.3	0.3	0.1	6	25.2	23.44	0.1	0.8	1.5	0.4	0.5	0.2	2.8	2.6	0.4	2.4	0.1

TDS in water sample of the study area varies from 165 to 533ppm in pond water and 331 to 343ppm in river water in dry season, similarly in the wet season this ranges from 155 to 461ppm in pond water and 313 to 318 ppm in river water. Total suspended solid concentration is higher in wet season than dry season and this concentration ranges from 100 to 290ppm in pond water and 580-650 ppm in river water in dry season and 40 to 240ppm in pond water and 450 to 490ppm in river water during wet season. Suspended solids in water of the study area are often the result of sediments carried by the water. The source of these sediments includes natural and anthropogenic (human) activities in the watershed, such as natural or excessive soil erosion from agriculture, forestry or construction, urban runoff, industrial effluents, or excess phytoplankton growth. The amount of total solids in the study area are ranges from 205 to 773ppm in pond water and 777 to 821ppm in river water and 255 to 751ppm in pond water and 898 to 966 in river water during dry season and wet season respectively. In the study area, the amount of dissolved oxygen varies from 3.9 to 5.2ppm and 2.9 to 4.2ppm in pond water and 5.4 to 5.9ppm and 4.1 to 4.7ppm in dry season and wet season respectively. The amount of dissolved oxygen gas depends on the nature of the organism, its degree of activity, the pollutants present, the water temperature and other factor. The amount of any gas, including oxygen, dissolved in water is inversely proportional to the temperature of the water; as temperature increases, the amount of dissolved oxygen (gas) decreases. High algal production in the surface waters can lead to depleted dissolved oxygen concentrations. In dry season the concentration of Biological oxygen demand ranges from 0.8 to 1.6 ppm in pond water and 1.2 to 1.3 ppm in river water whereas this value ranges from 1.8 to 2.2ppm in pond water and 2 to 2.4ppm in river water in the wet season. The concentration of BOD in wet season is higher than the dry season because the higher amount of organic matter in water present in wet season then the dry season and these organic matters come from surface runoff, agricultural and forest runoff, domestic waste water and municipal waste water discharge.

Table 2. The physico- chemical parameters of the collected water samples from Magura municipality during wet season.

	S/N	Physical Parameters										Cations				Anion					Trace
		pH	EC (μS/cm)	Temperature (°C)	Transparency (cm)	DO (ppm)	BOD (ppm)	TDS (ppm)	TSS (ppm)	TS (ppm)	Salinity (ppt)	Na^+ (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	K^+ (ppm)	PO_4^{3-} (ppm)	HCO_3^- (ppm)	Cl^- (ppm)	NO_3^- (ppm)	SO_4^{2-} (ppm)	Fe (ppm)
Pond	1	6.7	298	32	22.7	3.9	2	209	130	339	0.1	22.2	20	8.8	11.3	0.1	106.8	13.8	1.2	22.6	0.1
	2	6.6	260	33	19.5	3.8	2.2	182	170	352	0.1	18.3	16	7.0	8.5	0.7	98.6	10.0	1.2	20.4	0.1
	3	6.8	388	32.2	17.4	3.6	2.1	281	190	471	0.2	27.1	22.0	11.1	14.2	0.5	140.9	26.3	3.8	29.6	0.1
	4	6.7	382	32.5	21.1	3.7	2	267	180	447	0.1	27.3	23.5	10.2	12.	0.7	142.6	16.4	2	25.9	0.1
	5	7.7	382	31	20.8	3.3	2.2	267	210	477	0.1	26.6	25	9.8	12.1	1.1	146.1	17.4	1.8	25.6	0.1
	6	6.5	657	32	18.2	2.9	2	461	290	751	0.3	51.8	33.0	17.6	26.2	0.5	245.7	46.3	4.8	29.8	0.2
	7	6.6	460	32.5	22	3	1.8	373	240	613	0.1	28.9	27	14.9	16.4	0.5	211.3	26.6	2.9	29.5	0.1
	8	6.9	451	33	18.7	3.4	2	317	220	537	0.1	23.2	31.0	15.4	11.2	0.5	181.9	21.5	1.8	27	0.1
	9	6.7	276	31	21.5	3.9	2.2	193	130	323	0.1	13.8	20	8.6	9.0	0.5	103.1	11.2	2.7	21.2	0.1
	10	6.6	243	32	22.5	4.1	2.2	171	110	281	0.1	13.0	18.0	7.8	6	0.3	93.6	10.8	2.5	18.7	0.1
	11	6.7	375	30	19.5	3.9	2	233	150	383	0.2	25.9	24	10.6	12.8	1.3	106.2	24.6	3.9	23.6	0.1
	12	7.1	448	31	20.5	3.7	1.9	315	210	525	0.2	31.5	26.9	11.5	16.6	0.9	167.9	25.4	4.4	26.8	0.1
	13	7.0	341	30.2	18.2	3.4	1.8	239	230	469	0.1	20.5	24	9.5	9.8	0.7	126.9	18.5	2.2	24.2	0.1
	14	6.4	412	30.7	20.3	3.5	1.8	328	200	528	0.2	33.1	27.0	11.8	15.1	0.4	175.7	26.9	2.8	27.7	0.1
	15	6.8	419	31	22.5	3.9	2.1	293	180	473	0.2	27.2	28	12.2	12.3	0.4	155.9	23.9	2.8	28.1	0.1
	16	6.9	724	30.5	17.7	3.1	1.9	458	260	718	0.3	55.4	35	19.4	29.1	2.4	222.3	52.8	2.1	37.1	0.2
	17	6.7	221	30.2	19.5	4.2	2.2	155	100	255	0.1	13.5	15.0	7.4	6.6	0.3	78.3	9.2	2.4	20.5	0.1
	18	6.9	294	30.8	22.5	4	2.1	206	120	326	0.1	18.5	21.1	8.8	7.6	0.3	111.3	12.8	0.6	21.1	0.1
	19	6.8	313	30	17.7	4.1	2.2	219	130	349	0.1	19.1	23	10.0	10.1	1.0	110.3	17.0	2.1	25.2	0.1
	20	6.9	345	32	20.8	3.9	2.1	242	140	382	0.1	20.5	24.0	11	8.5	1.1	130.4	18.3	2.6	25.5	0.1
	21	6.7	372	33	19.5	3.7	1.8	261	150	411	0.1	19	26.9	12.7	13.2	0.6	140.6	17.8	2.6	27.1	0.1
	22	6.8	558	32.2	20.0	2.9	1.9	390	270	660	0.3	44.8	31.1	14	22.6	0.7	199.6	42.2	3.8	30.4	0.2
	23	7	569	31	20	3	2.1	398	280	678	0.3	40.4	30	16.0	23.7	0.7	212.5	39.7	2.7	30.6	0.2
	24	6.9	560	31.3	20.3	3.2	1.9	395	270	665	0.3	41.1	30.0	16.2	22.2	0.5	211.4	40.7	2.2	29.6	0.2
	Ave	6.8	406.2	31.5	20.1	3.6	2.0	285.5	190	475.5	0.2	27.6	25.1	11.8	14.1	0.7	150.8	23.8	2.6	26.2	0.2
	Max	7.7	724	33	22.7	4.2	2.2	461	290	751	0.3	55.4	35	19.4	29.1	2.4	245.7	52.8	4.8	37.1	0.2
	Min	6.4	221	30	17.4	2.9	1.8	155	100	255	0.1	13.0	15.0	7.0	6	0.1	78.3	9.2	0.6	18.7	0.1
	Stdev	0.3	129.9	1	1.6	0.4	0.1	88.9	58.3	144.1	0.1	11.6	5.2	3.4	6.4	0.5	47.3	12.2	1.0	4.2	0.01
River	25	6.7	446	31	27.4	4.4	2.3	313	610	923	0.17	32.5	26	12.4	17.6	0.5	165.7	27.3	2.9	25.7	0.3
	26	6.8	455	32.1	30.8	4.7	2.4	318	580	898	0.2	33.4	26	12.9	15.4	0.9	172.9	25.8	3	24.8	0.26
	27	6.8	452	31.6	26.5	4.1	2	316	650	966	0.15	33.0	25.7	12	16.1	0.5	168.5	27.3	2.8	25.5	0.31
	Ave	6.8	451	31.6	28.2	4.4	2.2	315.7	613.3	929	0.13	33	25.9	12.4	16.4	0.6	169.0	26.8	2.9	25.3	0.3
	Max	6.8	455	32.1	30.8	4.7	2.4	318	650	966	0.17	33.4	26	12.9	17.6	0.9	172.9	27.3	3	25.7	0.31
	Min	6.7	446	31	26.5	4.1	2	313	580	898	0.15	32.5	25.7	12	15.4	0.5	165.7	25.8	2.8	24.8	0.26
	Stdev	0.1	4.58	0.6	2.3	0.3	0.2	2.52	35.1	34.4	0.01	0.5	0.2	0.5	1.1	0.2	3.63	0.9	0.1	0.5	0.01

In the study area, both seasons show low concentration of salinity and that ranges from 0.1 to 0.37 ppt and 0.1 to 0.3 ppt in pond water and 0.23 to 0.26 ppt and 0.15 to 0.17 ppt in river water during dry season and wet season respectively. The salinity of a system will tend to change depending on the recharge of the system: during wet periods, salinity will decline as the concentration of salts becomes more dilute, whereas dry periods will lead to increased salinity values.

3.1.2 Cation content

The sodium concentration in pond water 15.82 to 68.94 ppm and 13.01 to 55.4ppm and in river water 37.91 to 39.41ppm and 32.5 to 33.43ppm in dry season and wet season respectively. It might be the contamination of water from urban runoff, industrial waste water discharge, agricultural runoff etc. Potassium concentrations in study area range between 8.38 to 33.1ppm in pond water and 18.31 to 19.21ppm in river water during dry season. On the other hand in the wet season these ranged between 6 to 29.05ppm in pond water and 15.44 to 17.57ppm in river water. Municipal discharges and agricultural runoff into receiving water may be the reason for this high concentration. During dry season the calcium concentrations in study area ranges between 16.03 to 40.89ppm in pond water and 28.04 to 30.98ppm in river water. Similarly in the wet season these ranges between 15.02 to 34.96ppm in pond water and 25.7 to 26ppm in river water. This high concentration may be due to primarily municipal, industrial discharges into water, and the presence of CO_2 which cause $CaCO_3$ to dissolve. The concentration of magnesium in study area varies from 8.38 to 22.02ppm and 7.01 to 19.44ppm in pond water and 13.98 to 14.82ppm and 12 to 12.9 in river water during dry season and wet season respectively. The concentrations of Mg^{2+} are high in the region where Ca^{2+} concentrations are also high and generally less than Ca^{2+} concentrations. This may due to discharge of industrial effluent in to water and geology of the watershed in which the stream or pond is located.

3.1.3 Anionic content

Bicarbonate in water sample of the study area varies from 66.56 to 238.53ppm in pond water and 154.84 to 160.39ppm in river water in dry season, similarly in the wet season this ranges from 78.3 to 245.7ppm in pond water and 165.7to 172.9 ppm in river water. Bicarbonate is mainly derived from CO_2. Two sources generally contribute to the CO_2 in water. One source is carbon dioxide derived from the respiration of plant roots and oxidation of organic matter. Another source is the carbon in the carbonate minerals, such as calcite and dolomite. The concentrations of chloride varies from 13.12 to 74.07ppm in pond water and 32.21 to 37.3ppm in river water during dry season and in the wet season these ranges between 9.21 to 52.81ppm in pond water and 25.75 to 27.3ppm in river water. The concentration of chloride in dry season shows higher than the wet season. It is due to the dilution of water in wet season. This may be due to discharges of domestic and industrial wastes, urban effluent, surface runoff etc. Sulphate concentrations in study area range between 25.05 to 44.87ppm in pond water and 30.74 to 35.06ppm in river water in dry season whereas these ranges in the wet season are 18.7 to 37.05ppm in pond water and 24.75 to 25.67ppm in river water. The observed higher concentrations may be due to weathering of sulphide bearing or by direct dissolution of evaporation deposits, or bacterial or oxidizing action in the oxidation of organo-sulphur compounds or discharged of industrial effluents. The concentrations of nitrate in study are varying from 3.03 to 8.2ppm and 0.6 to 4.77ppm in pond water and 3.33 to 4.05ppm and 2.76 to 2.97ppm in river water during dry season and wet season respectively. This may be due to fertilizer use, decayed vegetable and animal matter, domestic effluents, sewage sludge disposal on land, industrial discharges, leachates from refuse dumps etc. Nitrogen is important nutrient element in aquatic system that considered to be the primary drivers of eutrophication of aquatic ecosystems. In the dry season the concentrations of phosphate in the study area range between 0.5 to 3.44ppm in pond water and 0.9 to 1.3ppm in river water. On the other hand these ranges in the wet season are 0.11 to 2.36 ppm in pond water and 0.47 to 0.88 in river water. This may be due to agricultural and forest runoff, urban effluent etc. into receiving water. Phosphate is another nutrient element that is considered to be the primary drivers of eutrophication of aquatic ecosystems.

3.1.4 Other properties

Iron concentrations in study area range between 0.0 to 0.1ppm in pond water and 0.1 to 0.2ppm in river water in dry season whereas these ranges in the wet season are 0.1 to 0.2ppm in pond water and 0.26 to 0.31ppm in river water. The observed higher concentrations may be due to dissolution of bedrocks and minerals, sewage effluent and municipal and industrial discharges etc. The amount of total coliform in dry season ranges from 400 to 2100/100ml in pond water and 1500 to 2800/100ml in river water and ranges of fecal coliform 300 to 700/100ml in pond water and 400 to 700 in river water. During wet season, amount of total coliform ranges 700 to 4300/100ml in pond water and 2100 to 4300/100ml in river water and amount of fecal coliform ranges 300 to 1100 in pond water and 700 to 900 in river water. The amount of total coliform and fecal coliform are higher in wet season than dry season. This may be due to discharge of excreta from human beings, domestic waste, and surface runoff is higher in wet season than dry season. The fairly high values of total Coliform and faecal Coliform are indicative of increasing pollution of the ponds by organic means particularly through the discharge of sewage and domestic effluents into the ponds. Higher number of Coli form found in the river water samples may be for human feces and municipal wastes.

3.2 Evaluation of Surface Water for Drinking Purposes

Generally, water for drinking purposes should be colourless, odourless, free from turbidity, clear and free from excessive dissolved solids, and free from pathogenic organisms. Compare of the study area water quality with the water quality guidelines and standards for drinking uses by international organization or country are shown in Table 3. This table shows that in the study area the average value of pH in pond water during the dry season is slightly acidic and do not fall within the acceptable limit for drinking uses but rest of the time in the year, pond and river water are contain the acceptable limit of pH for drinking purposes. DO is another important water quality parameter. In the whole year, pond and river water in the study area reflect lower values of DO than the acceptable limit for drinking uses. The surface water in the study area during the both season is unsuitable for drinking uses due to the very high values of total suspended solids than the acceptable limits. The water in the pond and river also show higher concentration of potassium than the acceptable limit for drinking uses in the whole year. The excessive amount of potassium may prove detrimental to the human nervous and digestive systems. Other chemical constituents of surface water in the study area remain within the acceptable limit for drinking purposes during the both seasons. The worst bacterial quality was found in all samples. The average value of total coliform in pond water is 854.17/100 ml and in river water 2100/100 ml during dry season and in the wet season this values are 1512.5/100 ml in pond water and 2900/100 ml in river water. In pond water the average values of fecal coliform are 366.67/100 ml and 566.67/100 ml and in river water 600/100 ml and 766.67/100 ml during dry season and wet season respectively.

Table 3. Compare of the study area water quality with the water quality guidelines and standards for drinking uses by international organization or country.

		Parameter (Units)	Temperature (°C)	pH	Dissolved Oxygen (DO) (mg/L)	Total Dissolved Solids (TDS)(mg/L)	Total Suspended Solids (TSS)(mg/L)	Sodium (Na^+) (mg/L)	Potassium (K^+) (mg/L)	Calcium(Ca^{2+}) (mg/L)	Magnesium(Mg^{2+}) (mg/L)	Chloride(Cl^-) (mg/L)	Nitrate (NO^{3-}) (mg/L)	Sulphate(SO_4^{2-}) (mg/L)	Phosphate(PO_4^{3-}) (mg/L)	Iron (Fe) (mg/L)
Guidelines and Standards		[1]Bangladesh (Standards)	20-30	6.5-8.5	6	1000	10	200	12	75	30-35	150-600∞	10	400	6	0.3-1
		[2]WHO (guidelines)		*		1000	*	200		*		250		250		#
		[2]EuropeanUnion (Standards)		#		#	#	200				250	50	250		0.2
		[2]Canada (guidelines)						200				250	50	250		0.2
		[2]Australia (guidelines)		6.5-8.5		&		180				250		500		0.3
		[2]NewZealand (guidelines)											50			0.0 1
		[2]Japan (Standards)		5.8-8.6				200		300	300	200				0.3
		[2]UnitedStates (Standards)		6.5-8.5		500						250	10	250		0.3
[3]Average Values	Dry Season	Pond water	25.95	6.33	4.66	325.92	115.83	37.64	18.27	29.02	14.11	39.35	4.88	33.65	1.42	0.08
		River water	26.13	6.56	5.67	337	466.67	38.47	18.63	29.35	14.32	34.42	3.67	32.28	1.13	0.14
	Wet Season	Pond water	31.46	6.80	3.59	285.54	190	27.62	14.05	25.06	11.76	23.75	2.56	26.15	0.69	0.12
		River water	31.57	6.78	4.4	315.67	613.33	32.98	16.36	25.89	12.44	26.78	2.88	25.30	0.62	0.29

(Sources: [1]ECR, (1997); [2]Bhardwaj *et al*., (2006); [3] Present study)

Note: *: no guideline; #: not mentioned; &: acceptable to consumers no abnormal changes; ∞For Coastal Areas of Bangladesh, in case of non-availability of alternative sources value is 1000.

According to the all international and national regulations and guidelines, the acceptable limit of total and fecal coliform in drinking water is 0/100 ml. So, All samples show very much higher value than all the regulation and guidelines for drinking water. In the study area, 91.67% pond water and 100% river water samples show category D (high risk) and 8.33% pond water samples show category E (very high risk) in the wet seasons and during the dry season all samples showed category D (According to DPHE, 2005).

3.3 Evaluation of Surface Water for Domestic Uses

Generally, domestic purposes of water including washing of cloths, dishes, bathing and others. Some water quality parameter standards for domestic water uses are shown in Table 4. This table reflects that all water samples in both seasons in the study area satisfy the domestic water uses standards based on physico-chemical characteristics of surface water. But these water samples are failed to satisfy the permissible limits of total coliform in water according to Bangladesh water quality standard for surface water for domestic uses and that limit is 50 or less per 100ml. According to the Colorado Department of Public Health and Environment Water Quality Control Commission, 2009 regulations, all of the samples in the study area are consider as cless 2 water based on the presence of fecal coliform and that water can used for secondary contact such as fishing, water side recreation etc(According to CDPHE-WQCC, 2009).

Table 4. Some water quality parameter standards for domestic water uses.

Parameter	Unit	Standard values of parameters		[¤]Average Values in the study area			
				Dry Season		Wet Season	
				Pond water	River water	Pond water	River water
[*]Total Dissolved solids (TDS)	ppm	< 300	Excellent	325.92	337	285.54	315.67
		300-600	Good				
		600-900	Fair				
		900-1200	Poor				
		>1200	Unacceptable				
[#]Chloride (Cl^-)	ppm	250		39.35	34.42	23.75	26.78
[∝]Total Hardness	ppm	≤120	Excellent	130.5	132.17	110.96	115.83
		120-500	permissible				
		>500	unsuitable				

Sources: [*] Ahmed and Rahman, (2003); [#]Ramesh and Anbu, (1996); [∝]Taha, *et.al*, (2003); [¤]Present study.

3.4 Evaluation of Surface Water for Industrial Purposes

The water demands for industries depend upon the specific of each particular industry. Some industries require pure water, whereas others may require some specific minerals to be absent. The National Academy of Science and National Academy of Engineering (1972) and Hem (1989) provided international standards for some industrial projects that shown in Table 5. Table 5 represents that in the whole year the surface water of the Magura municipality area is suitable for wood chemical industries and unsuitable for textiles, soft drinks bottling and chemical pulp and paper industries. This water is also suitable for Synthetic rubber, Petroleum products, Hydraulic cement manufacture and canned and dried fruits and vegetables after primary treatment.

Table 5. Water quality requirements for some industries as proposed by NASNA E (1972) and Hem (1989)

Parameters			Ca^{2+}	Mg^{2+}	HCO_3^-	Cl^-	SO_4^{2-}	NO_3^-	Fe	TH	TDS	pH
Units			mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	
Textiles			0.0	0.0	0.0	0.0	0.0	0.0	0.1	25	100	-
Chemical pulp and paper			20	12	0.0	200	0.0	0.0	0.5	100	0.0	6-10
Wood chemicals			100	50	250	500	100	5	0.3	900	1000	6.5-8
Synthetic rubber			80	36	0.0	0.0	0.0	0.0	01	350	0.0	6.2-8.3
Soft drinks bottling			100	0.0	0.0	500	0.0	0.0	0.3	0.0	0.0	-
Petroleum products			75	30	0.0	300	0.0	0.0	01	350	1000	6-9
Hydraulic cement manufacture			0.0	0.0	0.0	250	0.0	0.0	25	0.0	600	6.5-8.5
Canned and dried fruits and vegetables			0.0	0.0	0.0	250	0.0	10	0.2	250	500	6.5-8.5
[#]Average Values in the study area (surface water)	Dry Season	Pond	29.02	14.11	140.66	39.35	33.65	4.88	0.08	130.5	325.92	6.33
		River	29.35	14.32	157.36	34.42	32.28	3.67	0.14	132.17	337	6.56
	Wet Season	Pond	25.06	11.76	150.83	23.75	26.15	2.56	0.12	110.96	285.54	6.80
		River	25.89	12.44	169.03	26.78	25.30	2.88	0.29	115.83	315.67	6.78

Note: [#] Present study.

3.5 Evaluation of surface water for irrigation purposes

Irrigation water criteria depend on both the chemical composition and the nature of plants to be irrigated, soil type, climate, amount and method of irrigation and drainage. Table 6 shows the classification of water quality for evaluates their suitability for irrigation purpose. From this table it has been found that both waters are suitable for soil of moderate drainage based on the values of electrical conductivity of water samples according to USSL (1954).

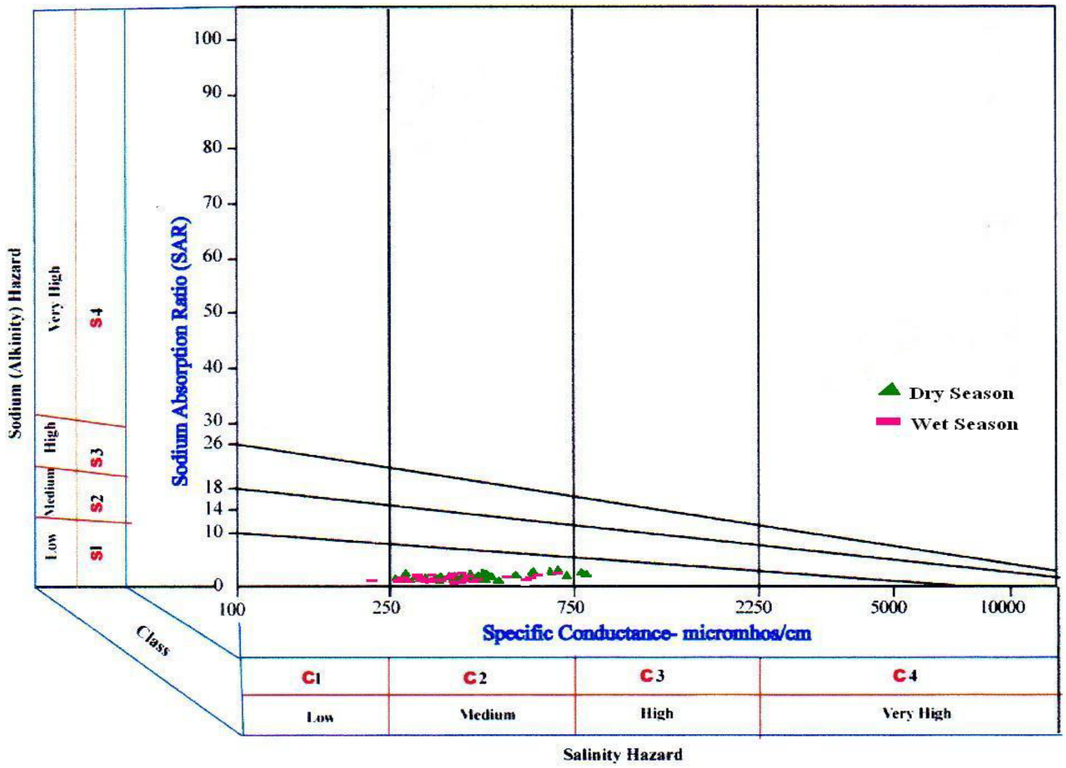

Figure 2: SAR Conductivity plot.

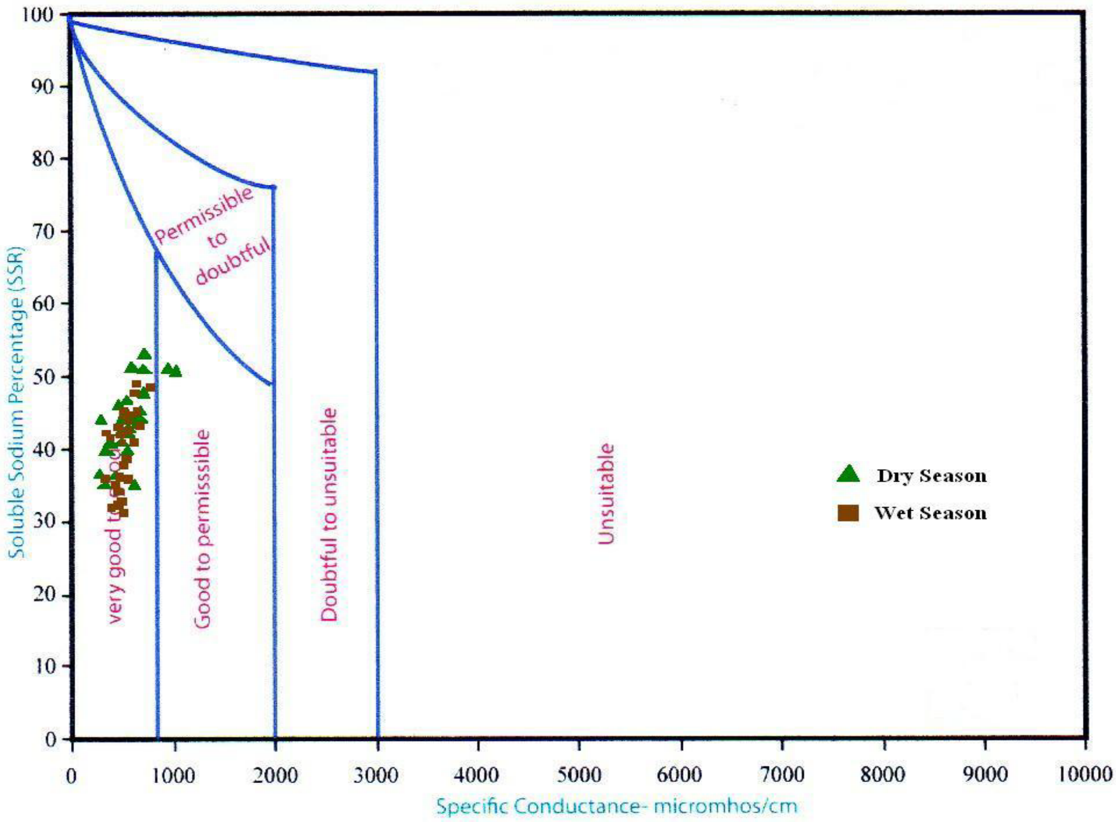

Figure 3: Wilcox diagram of surface water for irrigation.

Table 6. Classification of water quality for evaluates their suitability for irrigation purpose.

parameters	Units	Ranges	Water Class	Dry Season			Wet Season		
				[6]Average Values in the study area (surface water)		Suitability	[6]Average Values in the study area (surface water)		Suitability
				Pond water	River water		Pond water	River water	
[1]EC	µS/cm	**<250**	Salinity hazard low	501.79	522	Both water are suitable for soil of moderate drainage	406.17	451	Both water are suitable for soil of moderate drainage
		250–750	Salinity hazard medium						
		750–2,250	Salinity hazard high						
		>2,250	Salinity hazard very high						
[2]SSP	%	<20	Excellent	43.56	44.87	Both are permissible for irrigation	40.18	44.40	Both are permissible for irrigation
		20-40	Good						
		40-60	Permissible						
		60-80	Doubtful						
		>80	Unsuitable						
[1]SAR		<10	Low sodium water	1.40	1.46	Both can be used on all soils	1.12	1.33	Both can be used on all soils
		10-18	medium sodium water						
		18-26	high sodium water						
		>26	Very high sodium water						
[3]Chloride	meq/L	<2	Excellent	1.11	0.97	Both are excellent for irrigation	0.67	0.76	Both are excellent for irrigation
		2–6	Good to injurious						
		>6	Injurious to unsuitable						
[3]Sulphate	meq/L	<4	Excellent	0.7	0.67	Both are excellent for irrigation	0.54	0.53	Both are excellent for irrigation
		4–12	Good to injurious						
		>12	Injurious to unsuitable						
[4]Bicarbonate	meq/L	< 1.5	No restriction	2.31	2.58	Both are slight to moderate restricted for irrigation	2.47	2.77	Both are slight to moderate restricted for irrigation
		1.5 – 8.5	Slight to Moderate						
		> 8.5	Severe						
[5]Total Hardness	mg /L	0 – 75	Soft	130.5	132.17	Both are moderately hard	110.96	115.83	Both are moderately hard
		75 – 150	Moderately hard						
		150 – 300	Hard						
		>300	Very hard						
[2]TDS	mg /L	200–500	Best quality water	325.92	337	Best quality water for irrigation	285.54	315.67	Best quality water for irrigation
		1000–2000	Water involving Hazard						
		3000-7000	Used for irrigation only with leaching and perfect drainage						

Sources: [1]USSL (1954); [2]Wilcox (1955); [3]Eaton (1942); [4] Ayres and Westcot (1985); [5]Sawyer and Mc Carty (1967); [6] Present study.

Sodium Adsorption Ratio (SAR) is an important parameter in determining the suitability of irrigation water because high sodium makes the soil hard as well as reduces its permeability. According to U.S. Salinity Laboratory Staff (1954), all water in the study area can be used on all soils. The US Salinity Laboratory's diagram is used widely for rating the irrigation water where SAR is plotted against EC. Figure 2 illustrates that most of the water samples fall in the field C2S1, indicating medium salinity and low sodium water. This water is desirable for irrigation. Based on the classification after Wilcox (1955) for SSP, all the water samples in the study area fall under permissible class which indicates all this water could be safe for irrigation and These water contain less than 350 mg/L total dissolved solids and can be classified as best quality water for irrigation (Table 6). Wilcox diagrams were also plotted to determine the probability effects of the water on soil hydraulic properties if it is used for the irrigation activities in the area. On the Wilcox diagram prepared according to EC vs SSP values, most of the water samples of the study area fell into the classification "very good to good" for irrigation (Figure 3). For irrigation purposes water hardness classification is done by Sawyer and McMcartly (1967). Based on this classification, all the water samples in the study area fall under moderately hard class which indicate that these water could be safe for irrigation. According to Eaton (1942) classification, chloride (Cl^-) and sulfate (SO_4^{2-}) contents of the surface water samples in the study area are excellent for irrigation and The surface water of the study area can be classified as slight to moderate restricted for irrigation based on the bicarbonate (HCO_3^-) values of water according to Ayres and Westcot (1985) (Table 6). The Comparison of the surface water quality in the study area with the National and International recommended Guidelines for Irrigation Water Quality represents in Table 07. This table reflects that the average SAR, SSP, Ht, RSBC and other values (except potassium concentration) of surface water satisfy the recommended guidelines. So, all of the obtain values of the surface water samples in the study area are found to be within the safe limits for irrigation.

Table 7: Compare of the surface water quality in the study area with the National and International recommended Guidelines for Irrigation Water Quality

Parameter (Units)			Tem (oC)	pH	EC (mS /cm)	TDS (mg /L)	Ht (mg /L)	SAR	SSP (%)	RSBC (meq/ L)	Na^+ (meq/ L)	K^+ (meq/ L)	Ca^{2+} (meq/ L)	Mg^{2+} (meq/ L)	HCO_3^- (meq/ L)	Cl^- (meq/ L)	NO_3^- (meq/ L)	SO_4^{2-} (meq /L)	PO_4^{3-} (meq /L)
Sources	DOE (1997)*		20-30	6.5-8.5	-	0-1000	200-500	-			8.70	0.308	3.75	2.5-2.95	-	4.23-6.90	0.161	400	6
	BWPCB (1976)*			6.5-9.2		0-1500									-	16.90	0.726	400	
	WHO (1983)#		-	6.5-8.5	-	0-1000	500	-			8.70	-	10	4.17		7.04	0.161	400	-
	UCCC (1974)#			6.5-8.4	700-3000	450-2000		0-3			2.96-8.87				1.49	3.75	0.081		
	Wilcox (1955) #				250-2000				0-60										
	Richards (1954) #				0-3000			0-18					0-20						
	Ayres and Westcot (1985) #			6.0-8.5		0-2000		0-15			0-40	0-0.051		0-5	0-10	0-30	0-0.161	0-20	0-0.063
¤Average Values in the study area (surface water)	Dry Season	Pond	25.95	6.33	0.501	325.92	130.5	1.40	43.56	0.86	1.64	0.47	1.45	1.16	2.31	1.11	0.08	0.7	0.05
		River	26.13	6.56	0.522	337	132.17	1.46	44.87	1.11	1.67	0.48	1.47	1.18	2.58	0.97	0.06	0.67	0.04
	Wet Season	Pond	31.46	6.80	0.406	285.54	110.96	1.12	40.18	1.22	1.20	0.36	1.25	0.97	2.47	0.67	0.04	0.54	0.02
		River	31.57	6.78	0.451	315.67	115.83	1.33	44.40	1.48	1.43	0.42	1.29	1.02	2.77	0.76	0.05	0.53	0.02

Note: * National Guidelines; #International Guidelines; ¤ Present study.

4. CONCLUSIONS

From the values of physicochemical parameters of surface water in the study area it can be concluted that any toxic pollution and salinity problem has not found but fecal pollution is one of the major concern in this area. This research reveals that the surface water quality in Magura municipality is ranging from suitable to unsuitable for the different purposes. Result of this research work has shown that surface water (both ponds and river water) of Magura municipality is not suitable for drinking because of the high values of total suspended solids, potassium concentration, fecal pollution and lower values of DO than the standards or guidelines for drinking uses. This pollution may be occur from domestic effluents, fertilizer use, decayed vegetable and animal matter, sewage sludge disposal, leachates from refuse dumps, industrial discharges and municipal wastes and effluents discharged into the surface water. This study also explicit that the surface water is can be used for some selective domestic purposes but not all domestic purposes because of the higher fecal pollution of water. Higher number of Coli form found in the surface water samples may be for human feces and municipal wastes. Polluted stream flows into the surface water and latrine within 10 m from the pond were the responsible factors for comtamination of surface water by fecal coliform. This water is also suitable for wood chemical industries and after the primary treatment this water could be used for many other industries. The surface water in the area studied is suitable for agricultural purpose. The SAR, SSP, RSBC, TH, TDS, EC and other contents of surface water in the study area satisfy the national and international guideline for irrigation water quality. So, the surface water in the Magura municipality area can be used different purposes after some conventional treatment.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

MEMBRANE BIOREACTOR PROCESS USING A SIMPLE CERAMIC FILTER FOR WASTEWATER RECLAMATION AND REUSE

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ABSTRACT

The applicability of a simple ceramic filter to MBR process has been examined. The ceramic filter made of locally available materials has been developed and applied to arsenic removal from groundwater in Bangladesh. A hollow cylindrical shaped filter was made with the combination of 80% clay soil and 20% rice bran, and after sundry it was burnt in a local pot burning kiln. The filter was submerged in an acrylic cylindrical column reactor. Synthetic domestic greywater was prepared by using three commercial daily-use products (shampoo, liquid dish cleaner and powdered laundry detergent) and fed into the reactor continuously. The effluent through the filter was obtained by the gravitational pressure. The average flux performance was 0.28 m/d with an average HRT of 1.7 days. Efficient activated sludge separation, high flux performance and high removal performance (BOD and MBAS removal were more than 99%) were ensured. The no necessity of the filter cleaning throughout the run period emphasized the non-clogging of the filter. Besides the very low manufacturing cost of the filter, use of the gravitational filtration also reduced the operational cost. This filter would be applicable to MBR facilities in developing countries for wastewater reclamation and reuse.

Keywords: *MBR, Ceramic filter, Greywater treatment, MBAS, Low cost technology*

1. INTRODUCTION

The concept of wastewater reclamation and reuse is becoming a promising concern in recent years due to water shortage especially in the area of rapid urbanization and development (Verstraete et al., 2009; USEPA, 2004). To solve the growing water stress the interest in water reuse, either from an environmental or an economical point of view, became evident. Generally all the water draining form a household, except the toilet wastes, is termed as greywater. So, it corresponds to the wastewater from baths, showers, hand basins, washing machines, dishwashers and kitchen sinks, but excludes the streams from toilets (Christova-Boal et al., 1996; Li et al, 2009; Jefferson et al., 1999; Otterpohl et al., 1999; Ottoson and Stenstrom, 2003; Pidou et al., 2008) though some authors exclude kitchen wastewater from the other greywater streams (Al-Jayyousi, 2003; Little, 2002; Wilderer, 2004). The main advantage of recycling greywater is that it is a large source with a low organic content. To illustrate, greywater represents upto 70% of total consumed water but contains only 30% of the organic fraction and 9-20% of the nutrients (Kujawa and Zeeman, 2006). Moreover, in individual households, it has been established that greywater could support the amount of water needed for toilet flushing and outdoor uses such as car washing and garden watering (Karpiscak et al., 1990). Therefore the issue of greywater reclamation and reuse has drawn significant research attention in the recent past for minimizing the fresh water shortage and sustainable water management as well as environment protection. Several studies already conducted for this issue including physical-chemical and biological treatments (Friedler et al., 2005), constructed wetland (Gross et al., 2007), membrane bioreactor (Merz et al., 2007), upflow anaerobic sludge blanket (Elmitwalli and Otterpohl, 2007) and biofilm reactor (Imura et al., 1995). Among the treatment technologies, membrane bioreactor (MBR) processes are regarded as key elements of advanced wastewater reclamation and reuse schemes due to its supremacy of producing reusable effluent, small foot print and also low sludge production quality (Cote et al., 1998; Davis et al., 1998; Engelhardt et al, 1998; Thomas et al., 2000). Furthermore submerged MBR is more common because of its lower energy consumption (Yamamoto et al., 1989; Ueda et al., 1996; Ueda and Hata, 1999). But the MBR processes are still costly in terms of membrane and maintenance cost and energy consumption.

Therefore cost saving, simple and least-maintainable MBR process is ideal in the case of its wide application, especially in developing countries.
Although the membrane used in an MBR is usually made of an organic polymer or inorganic ceramics, alternative membrane materials, such as clay and fly ash, have also been studied for use in an MBR recently (Jedidi et al., 2010, 2009; Palacio et al., 2009; Khemakhem et al., 2007). A ceramic filter made with locally available and cheap materials (clay soil and rice bran) was developed for the application of arsenic removal from groundwater (Shafiquzzaman et al., 2011). The ceramic filter was manufactured in a rural area in Bangladesh, and its arsenic removal performance was examined on a laboratory scale as well as at the field level. The iron oxide floc that adsorbed the arsenic in the water was completely separated by the filter using gravitational filtration. Because of the effective separation performance of the ceramic filter, a high performance of arsenic removal was obtained during the one year of operation in the rural area of Bangladesh. This ceramic filter having pore size of 1 – 5 μm (Shafiquzzaman et al., 2011) seemed to be applicable to an MBR as a membrane because it would be able to separate the activated sludge floc as efficiently as a commercial polymeric membrane or mesh filter (Satyawali and Balakrishnan, 2008; Fuchs et al., 2005). In addition, gravitational filtration in an MBR is simple and has cost savings compared to a suctioned-filtration type and has been applied in actual MBR facilities (Ueda and Hata 1999; Izumi et al., 1995).
Therefore the cost of installation and operation of an MBR would be reduced by using a ceramic filter with gravitational filtration, which could be applicable in developing countries. However, few studies on a low-cost MBR process have been performed for applications in developing countries. The aim of this study was to clarify the possible application of the ceramic filter to MBR as a membrane for domestic greywater treatment which was recently applied for arsenic removal from the groundwater in Bangladesh. The flux, BOD and MBAS removal performance, and the filter cleaning frequency in a long-term operation were examined in laboratory experiments.

2. MATERIALS AND METHODS

2.1 Experimental Set Up and Operational Conditions

An acrylic cylindrical column (14 cm inner diameter and 150 cm height) was used as the reactor as shown in Fig.1. A rubber cap with effluent pipe was attached on the open side of the ceramic filter and the filter was submerged in the reactor. A concrete block with a stand was used to tight and holds the filter, and the empty space around the block was filled by stone chips to avoid the sludge accumulation.

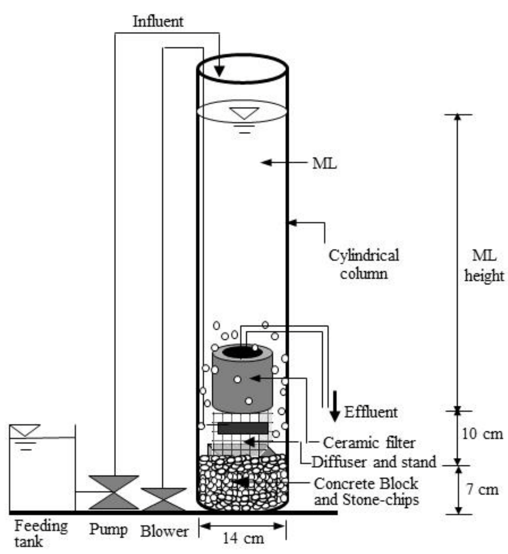


Fig.1. Schematic representation of the system

A diffuser was set in the stand below the filter so that the rising air bubbles can provide the filter surface with more shear stress, which is effective for removing attached sludge, and to mix the mixed liquor (ML) in the reactor and also to maintain an aerobic environment for the normal growth of activated sludge.

ML was taken from an actual MBR facilities (domestic wastewater treating capacity of 1100 p.e. (220 m^3/day)) in Shiga Prefecture, Japan. This ML was used as seed for the experiment. Synthetic greywater was fed into the reactor continuously and the permeate effluent through the filter was obtained from the out let pipe by the gravitational pressure (Fig.1). The specified height of the ML was manually controlled. The reactor was operated under intermittent aeration (1 hr with and 1 hr without aeration) mode. The filters were not cleaned throughout the experiment. The reactor was monitored by daily measurement of permeate effluent volume, temperature, pH and dissolved oxygen (DO). The specified water quality parameters were analyzed periodically for the permeate effluent and MLSS and MLVSS were analyzed for ML. There was no sludge wastage except for analyzing during the operation. The detail specifications of the reactor were shown in Table 1.

Table 1: Specification of the reactor

Description	Specification
ML height, cm	128
Hydrostatic pressure, MPa	0.013
Initial MLSS, g/L	4.2
Operating mode	Intermittent aeration
Run period, day	169

2.2 Synthetic greywater as influent

The synthetic greywater used in this study was prepared by mixing of three commercial daily-use products; shampoo, liquid dish cleaning and powdered laundry detergent. Each 10 liter of synthetic greywater consisted of 12 mL of shampoo, 6 mL of liquid dish cleaner and 1 gm of powdered detergent. The characteristics of the synthetic greywater were stated in Table 2. BOD load was as 0.033±0.009 kg/m^3/day in the rector.

Table 2: Characteristics of the synthetic greywater (Average ± Standard Deviation)

Parameter	Concentrations (g/L)
MBAS	30.27±8.38
BOD	53.74±14.92
TOC	26.73±2.87
T-N	0.59±0.14
T-P	7.02±0.32
PO_4-P	0.24±0.08
Poly-P	6.52±0.34

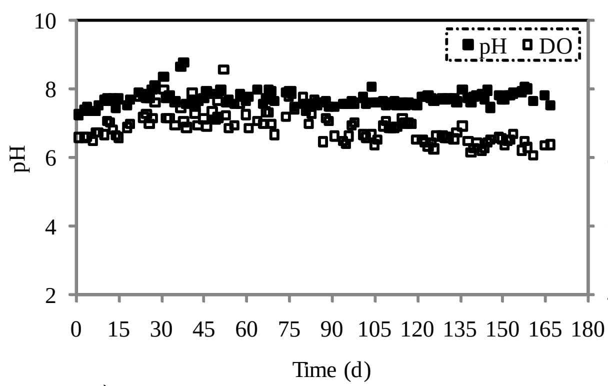
2.3 Analytical methods

During the operation, basic water quality parameters, i.e., Methylene blue active substance (MBAS), biochemical oxygen demand (BOD), total organic carbon (TOC), total nitrogen (T-N), total phosphorus (T-P), polyphosphate (poly-P) and phosphate ion (PO_4-P) were analyzed for the permeate effluent by using Japan Industrial Standard method (JIS K 0102, 1993). MBAS was analyzed by using the formation of ion pairs between the cationic dye methylene blue and anionic surfactants, which are extractable into chloroform ($CHCl_3$). The absorbance of the $CHCl_3$ extract was measured using a spectrometer at 650 nm using standard of sodium dodecyl sulfate (SDS). Five day biological oxygen demand was measured using unfiltered sample and thus represents the total BOD_5. Shimadzu TOC-VCPH/CPN was used for TOC and T-N measurement. Total carbon (TC) was first measured and then the inorganic carbon (IC) was measured. TOC was determined by subtracting IC from TC. Preliminary digestion step using nitric acid and sulfuric acid solutions was applied to find out T-P. Whereas, poly-P was measured using the weak acid hydrolysis process. PO_4-P were determined by molybdenum blue colorimetric method (ascorbic acid) using a spectrophotometer (880 nm, HITACHI U-2900 SPECTROPHOMETER). pH and DO were measured by using pH meter (HORIBA, F-21) and DO meter (HACH HQ 30d), respectively. After centrifuge separation, drying method and burning method was used for MLSS and MLVSS measurement respectively for ML.

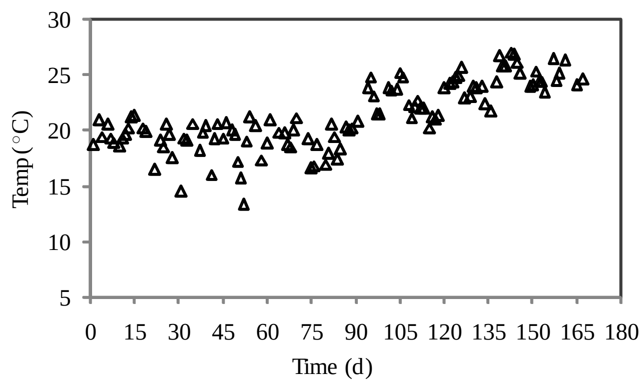
3. RESULTS AND DISCUSSION

3.1 Operational conditions

The pH was always stable within 7.0 to 8.0 (Avg±Std Dev: 7.68±0.23) and DO was within 8.0 to 9.0 mg /L (Avg±Std Dev: 8.87±0.46) as shown in Fig.2. Though the average temperature was 21.26±2.91, it was higher during the operation period of 90 day to the last day of the experiment (Fig.2) due to the summer time (May-July).



a)



b)

Fig.2. a) pH and DO b) temperature inside the reactor

3.2 Flux performance and HRT

The flux was calculated from the daily permeate effluent volume that filtered by gravitational pressure. Flux data with HRT was shown in Fig 3. During the first two weeks the flux was declined and again raised to 0.45 m/d and then it gradually decreased upto 0.2 m/d. After 75 days the flux was stable between 0.2-0.25 m/d upto the end of the experiment with the HRT of less than 2 days. The flux value more than 0.2 m/d for the entire experiment period emphasized the sufficiency of the flux and the non-clogging of the filter.

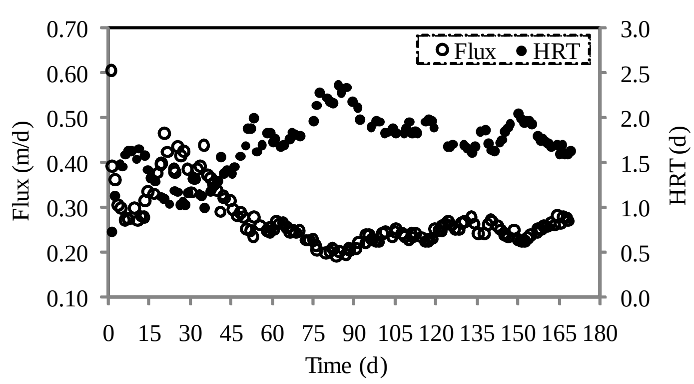


Fig.3. Flux data with HRT

From the economical point of view an increase of the flux is highly desirable and evident and the literature review showed the removal performances of MBR treating domestic wastewater are quite independent of HRTs in the range of 2–24 h (Stephenson et al., 2000) with an optimal HRT of only 1.5 h (Hu, 2002) for greywater treatment. In this study by applying the gravitational filtration the HRT was found less than 2 days but it would be possible to reduce further by inducing the suction pump.

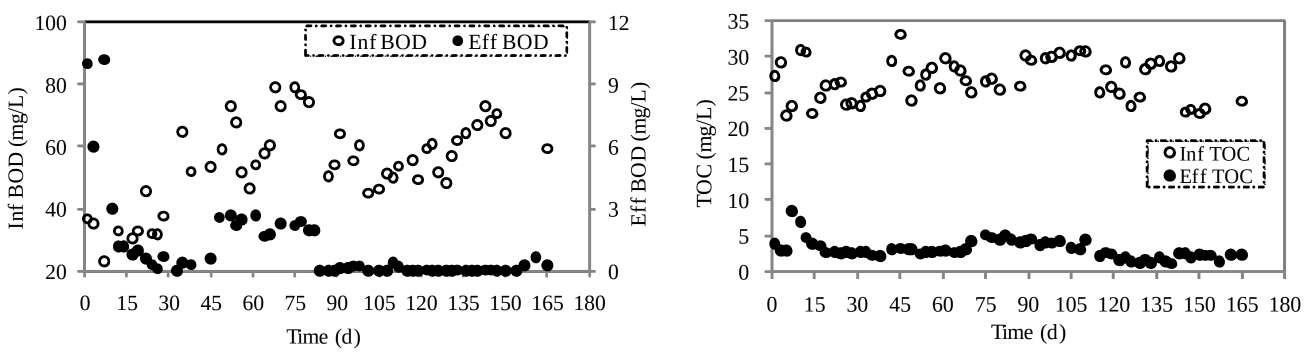
3.3 Removal performance

3.3.1. Organic removal

The removal efficiencies of BOD and TOC were shown in Table 3 and Fig 4. BOD was removed more than 97-100% and TOC was removed more than 88%. During 45 to 80 days of operation the effluent BOD concentration was near 3 mg/L but after that it decreased <1 mg/L. The effluent TOC concentration was stable throughout the operation period. This high removal performance emphasized that the biological degradation was carried out effectively with the running operational conditions.

Table 3: Effluent Quality

Parameters	Influent Concentrations (mg/L)	Effluent Concentrations (mg/L)	Removal Efficiency (%)
BOD	53.74±14.92	1.10±2.04	>97
TOC	26.73±2.87	3.11±1.30	>88
T-N	0.59±0.14	1.43±1.07	0
T-P	7.02±0.32	7.00±0.58	0
MBAS	30.27±8.38	0.12±0.41	>99



a) b)

Fig.4. Influent and effluent concentrations of a) BOD b) TOC

3.3.2. Nutrients removal

The influent and effluent concentrations of T-P, Poly-P, PO_4-P and T-N were shown in Fig.5 and in Table 3. The influent concentration of T-P was around 7 mg/L and was hydrolyzed to PO_4-P through this system. The effluent T-N was sometimes higher than influent suggested that some nitrogen released from ML. The reactor was not optimized for the nutrients removal and hence further advancement is needed regarding this issue though the reuse of the effluent would be possible where the remaining nitrogen and phosphorus are useful e.g. landscape and agricultural irrigation.

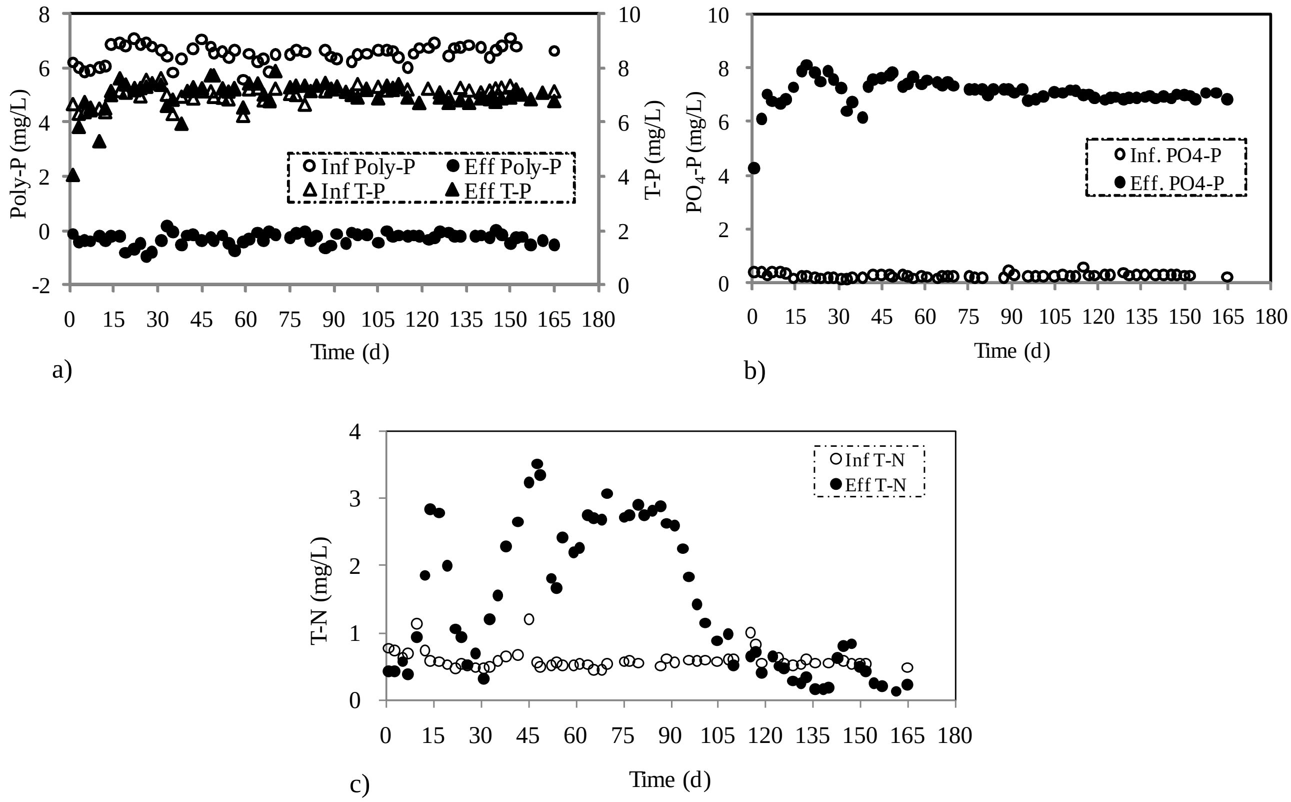


Fig.5. Influent and effluent concentrations of a) Poly-P and T-P b) PO_4-P and c) T-N

3.3.3. MBAS removal

The MBAS concentrations (effluent and influent) were shown in Fig.6 and Table 3. 99-100% removal of MBAS was obtained through this system that highlighted that the system could remove the surfactant. No foaming quality of the permeate effluent ensured the absence of the anionic surfactants which widening its reuse possibility and applications.

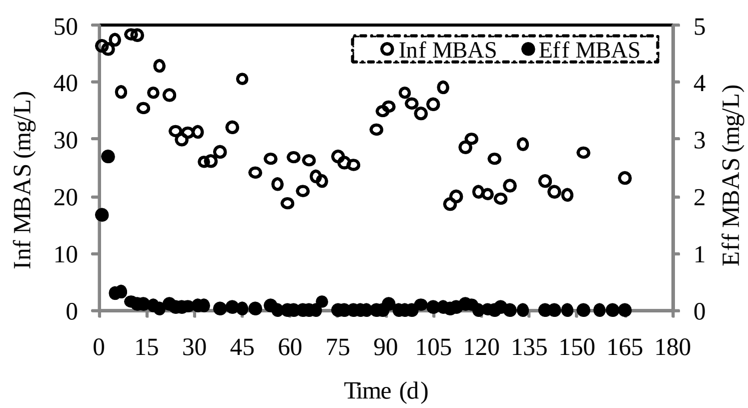


Fig.6. Influent and effluent concentrations of MBAS

3.4 Sludge concentrations

The initial MLSS in the activated sludge was around 4.5 g/L, but gradually decreased and stable at about 2.5 g/L. Despite of no sludge loss the reason of decreasing MLSS was difficult to clarify. Though some sludge may be lost with sludge foaming but this should not be the main contributor. The synthetic domestic greywater contains only surfactant and had no organic content. Therefore there were no sources to increase the microbial community and some surfactants were also liable to prohibit the microbial growth (Wu et al., 2003). The ratio of MLVSS to MLSS decreased with the operational time and reduced from 90% to as low as 83% (Fig. 7). There was no indication demonstrated that effluent quality was influenced by the decreasing of MLSS (Fig. 4 and Fig. 6).

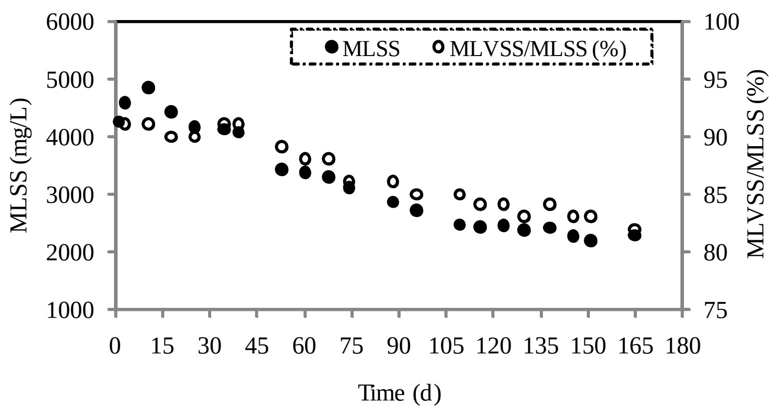


Fig.7. MLSS with time period

3.5 Maintenance of the ceramic filter

A very thin cake layer of activated sludge was found on some portions of the ceramic filter surface after the operation. This phenomenon suggested that the characteristics' of the synthetic greywater prevented to accumulate the ML on the filter surface and reduced the possibility of filter clogging. At the very last day (169th day) the flux value was >0.25 m/d (Fig. 3) emphasized the non-clogging of the filter and ensured the prolonged filter life span. However physical and/or chemical cleaning would be applied in case of clogging or fouling of the filter in the same way as conventional MBR process. Moreover the ceramic filter could be replaced by new one since its manufacturing procedure was easy and inexpensive (Shafiquzzaman et al., 2011).

3.6 Applicability to actual MBR facilities

Sufficient amount of flux as well as high organic and surfactant removal efficiency emphasized that the ceramic filter would be applicable for actual MBR facilities for domestic greywater reclamation and reuse. Low manufacturing cost of the ceramic filter, using gravitational filtration to reduce the operational cost and the easy maintenance method of the filter would be advantageous to apply this filter associated with MBR facilities in developing countries as decentralized approach (Massoud et al., 2009). The improvement of the filter shape and optimization of aeration system need to be clarified through further study before actual application.

4. CONCLUSION

In this study the applicability of the low cost and simple ceramic filter as a membrane in MBR process was examined for domestic greywater reclamation and reuse. The flux and removal performance were investigated in the laboratory using gravitational pressure filtration. The identified key conclusions were as follows:

- The ceramic filter could separate the activated sludge with a simple gravitational filtration.
- The flux performance was ranged from 0.2 - 0.25 m/d up to six month without cleaning the filter.

- Complete removal of BOD and 90% removal of TOC in intermittent aeration condition were obtained.
- 99-100% removal of MBAS highlighted that the system could remove the surfactant.
- The synthetic greywater seemed to prevent the cake layer formation on the filter surface that emphasized the non- clogging of the filter for a long period when the gravitational filtration was applied.
- The filter would be advantageous to apply in MBR facilities instead of membrane in developing countries as decentralized approach for the reclamation and reuse of domestic greywater with the improvement of the filter shape and optimization of aeration system through further study.

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THE HUNGER, POVERTY AND SILENCE : THE INVISIBLE BOND IN SOCIAL ECOLOGY

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Introduction

With the spurt of the new global order in terms of liberalization, privatization and globalization that got off to a swash bucking debut in the decades of 90's, the concept and perception of poverty and hunger have undergone a sea-change. Across the world, one billion people are now reeling under abject poverty and hunger. This is basically a threat to the human kind for their mere biological existence. Poverty is now even hatched a wider expanse of impacts over the geography of hunger. Around 895 million people across the world have been plunged to BPL (Below Poverty Line) status. Now these two problems are so close and cognate that no further explanation is at all essential to elicit their inter-relationship. Silence is again another character or traits of danger which can be traced as an integer two of the preceding problems, i.e. hunger and poverty. The collary can be framed this way- the people are silent hence they are poor; people are poor so they are hunger in another way, silence or voicelessness has created a dangerous synergy between hunger and poverty.

Voicelessness is increasing across the global society especially from within the peripheral population. The poorer populations are undermined by the society, denied by the state power, denigrated by the governance and are forced to lead a life which is disgraceful and without any destination. The present seminar will be highlighting the crux of the combination amongst the three deleterious components of the society, i.e. poverty, hunger and silence. So, the objectives of the seminar are:-

1. To assess the present level of poverty in a global perspective.
2. To elucidate the level of hunger thereafter and therein
3. To examine the concept of silence as a catalytic agent for synergizing to other previously spoken component i.e. poverty, hunger and silence.
4. To examine the relation and reticulation of all these three to present what we called an interactive danger perspective towards achieving social growth.
5. To present a strategic configuration for handling these conjunctive threats.

Methodology:

1. Interaction with resource persons
2. Broad basing of data and information
3. Processing of data and information
4. Concept propelling
5. Concept configuration
6. Customization of the concept
7. Communication to the audience

Relevance of selection of the topic:

- The basic motto of Extension science is to promote development and integrate human behavior with technology and concept for a sustainable growth and progress in society.
- In getting this done, the most important and crucial hindrances are poverty, hunger and silence.
- So, we need to know the reasons for exposition and impacts of hunger, silence and poverty.
- The proper perception of hunger, poverty and silence and their synergy would help us to derive a comprehensive strategy for combating these three enemies in a both integrated and interactive manner.
- Millions of extension programmes are being organized but still some people have opted to go silent leaving a non-communicative response to a score of proclaimed communication.
- The poverty has also been perceived in terms of poverty for money and motivation, poverty for knowledge and perception and all these become endemic when get intermingled with hunger and silence.

So in the alleviation programme as well as in micro communication approaches, data amounting to creation of hunger need to have an extension intervention for breaking the silence and making people vibrant and echoing for accessing both rights and resources.

Across the world, poverty now is not only a concern but a threat to sustainable development subsequent to the Millennium Declaration made by UNO. Poverty has bestowed no sign of reduction, it is increasing unabatedly. Whatever benefits or devidence are created somehow and somewhere are being extorted by poverty and ultimately a more impoverished world is scrambling.

The root causes of poverty are reticulated with two more barriers to development and these are hunger and silence. More than 1 billion people across the world are touching the bed at night having no food in the stomach and more than 80% of Indian women are suffering from anaemia. Every 3 seconds one child is bidding goodbye to this world because of hunger and malnutrition.

Another impairment to development is that most of the people, branded as marginal populace are remaining beyond the network of communication both interpersonal and electronic communication systems. Getting uninformed or making people uncommunicated to have got socio-political elements towards creating an information split in the society in the form of information have and have not's.

Voicelessness is a serious problem standing between development and participation. Again voicelessness is a barrier for making flow of data natural and successful. When people remain voiceless, poverty keeps surging up, and when people remain poor, both hunger and voicelessness are suppose to make a heinous covalency.

The synergy is here used as a negative combination or conglomeration amongst and between elements of threats and elements of dissonance. Imagine for a situation where people go hungry and keep silent. Speechlessness and voicelessness go together to invite both poverty and hunger. When poor people stop raising voices to claim their stakes and gone depleted off all the vigor, then it is not a problem to the people themselves but is also a problem to the planner and administrator as well.

Synergizing is a process of combining pluralities into a single but polyhedral unit. The hunger, poverty and silence thus have moved together to brand a component of the population marginalizing and impoverishing along and across the time and space.

What is Poverty?

Poverty is the condition in which a person or community is deprived of or lacks the essentials for a minimum standard of well-being and life.

Poverty is the lack of basic necessities that all human beings must have: food and water, shelter, education, medical care, security, etc. A multi-dimensional issue, poverty exceeds all social, economic, and political boundaries.

Types of Poverty

- **Extreme Poverty:** Those who live on daily income of less than US $1.25, who are not able to meet their daily basic needs.
- **Moderate Poverty:** Those who have a daily income between one and two US dollars and are able to just, barely, meet their basic needs.
- **Relative Poverty:** Those households where the income level is below a given proportion of the national average income.

Measuring National Poverty

The time lag in global poverty statistics stems partly from the prohibitive cost, skills and logistics involved in conducting household income and expenditure surveys in the developing world. Delays may be aggravated through controversy over the basis of calculating poverty, often inevitable where it dictates the distribution of aid or social welfare.

Most countries determine their national poverty line as the value of a basket of basic food and essential non-food items. Some governments work with separate urban and rural poverty lines, recognising that costs are higher in cities. The food poverty line is the most stringent measure, reducing the basket to food items only. Household surveys analyse consumption as well as income, recognising that goods may be exchanged by barter and that many families grow their own food.

Most poor countries take a very narrow view of what constitutes “essential” non-food items to include in the basket. This approach attracts criticism for ignoring other important dimensions of poverty such as limited access to education, health, water and housing.

Measuring Global Poverty

As each national poverty line reflects a different view of essential food and goods, an alternative international poverty line by reference to the average of the national poverty lines in 15 of the world’s method is needed to aggregate global poverty on a consistent basis. The World Bank calculates poorest countries. This exercise was last completed using 2005 data, resulting in an international poverty line of $1.25 per day. The figure of $1 a day which is often quoted relates to an earlier assessment based on 1985 data.

Global poverty is then assessed by reference to “data from 675 household surveys across 116 developing countries”, according to the World Bank. This data is compared to the $1.25 benchmark, not by standard currency exchange rates, but by purchasing power parity (PPP) rates which smooth out the different buying power of the dollar in each country.

The World Bank figure of $1.25 per day was intended to be a bottom marker. Unfortunately, the two countries with the largest populations in the world, India and China, have both defined national poverty lines which are even lower. India’s poverty line is $1.02 which gives a national poverty rate of 26%, compared to 42% on the international basis. In China the gap is even wider, tripling its national poverty numbers to over 200 million. These inconsistent measures are the source of much confusion.

A second tier international poverty line of $2 per day is derived from the average of national poverty lines in all lower and middle income countries. The Bank reports that 2.6 billion people live below this benchmark, a figure which has changed little since 1981. Indeed a slightly higher benchmark of $2.50 per day captures more than half of the world’s population. The World Bank is due to review these poverty lines in 2011.

Two issues have been preventing most governments from handling poverty well (Vincent. W. 2007);

1. Most governments in both rich and poor countries do not see poverty-reduction as being any priority to them, and so do not make much attempt to reduce poverty. The wider benefits of reducing poverty are not widely understood.

2. The few governments in rich or poor countries that do see poverty-reduction as being of some priority to them, have commonly wasted much of the resources they use in mistaken attempts at poverty reduction from not understanding their best policy options for that.

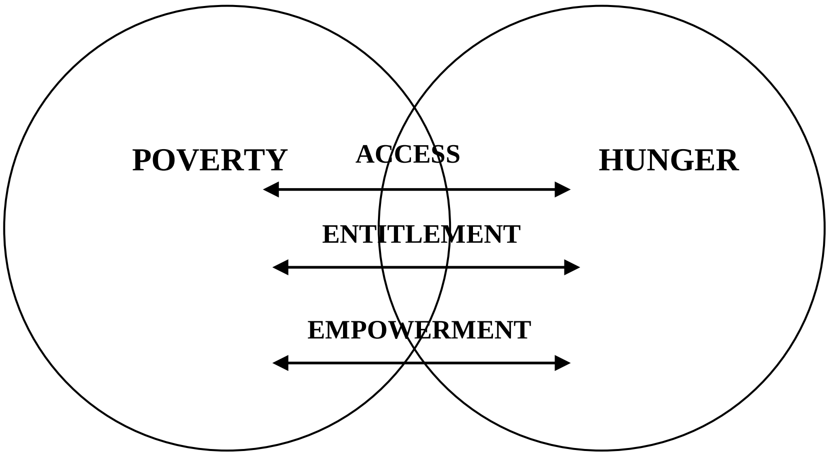
Recently food prices have been rising worldwide, partly from new Bio fuel policies, mostly helping to worsen global poverty. 2009 has also seen richer countries hitting a substantial economic downturn that could make it harder for them to help reduce poverty for some years. And of course all governments do have other problems to try to deal with, and also all have some resource limitations that restrict the actual amount that they can achieve. But mostly governments could certainly do better.

In many poorer countries, the current world recession is also causing family remittances from overseas workers or migrant workers to fall. As more migrant workers lose jobs in Western Europe and the USA, remittances to poor families in Africa, Central Asia and Eastern Europe are expected to be hardest hit.

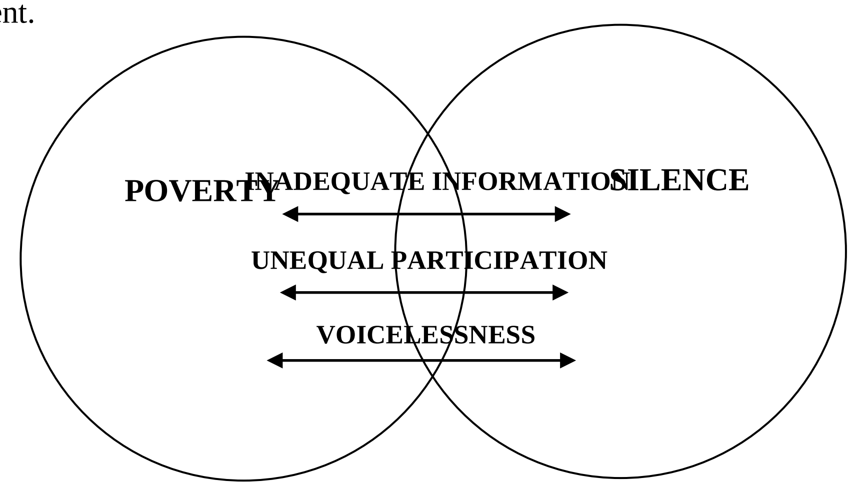
Food is available but the purchasing power is not there:

The 2007-2008 food crises taught us that the global food crises are not due to the fact there is not enough food available. Nearly all of those going hungry around the world are doing so because they cannot afford adequate food. Two-thirds of these people live in the rural areas of developing countries, in other words, where food is produced. These people are poor and go hungry because they do not have any employment opportunities and because they are largely denied access to a means of production. Hunger is therefore first and foremost the result of uneven income distribution. The repercussions of the financial crises are compounding the already precarious living situations of these people because they increase the volatility of prices and costs, making these people more economically vulnerable. At the same time, the situation of the federal budgets of the developing countries is deteriorating, and investments in social and economic infrastructures are not made or are delayed. This also accelerates the cycle of vulnerability, poverty and hunger.

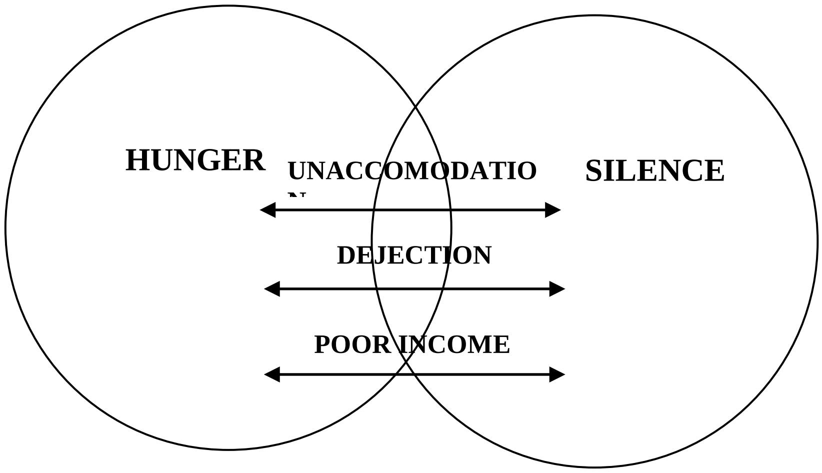
The process of synergizing poverty, hunger and silence:



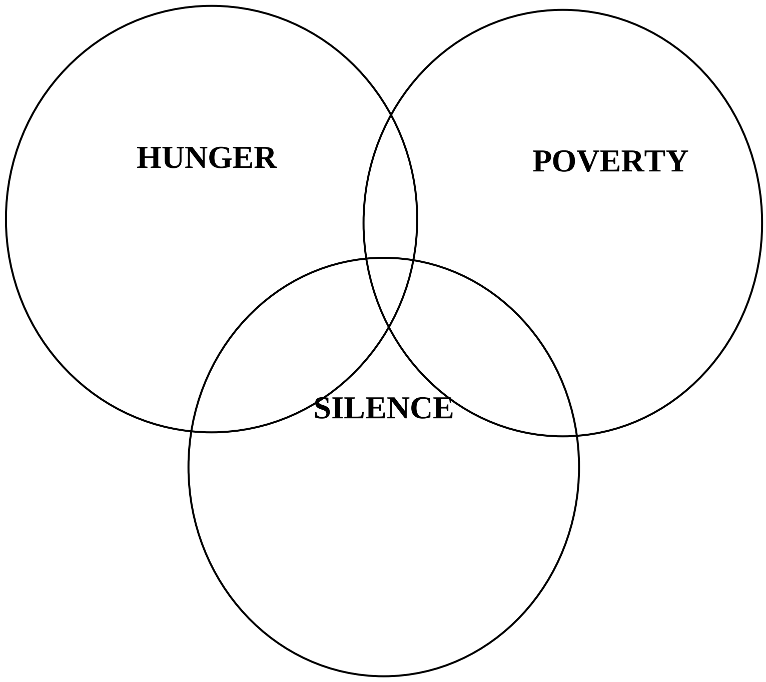
The synergy between poverty and hunger has cropped up through the three basic elements of economic socialization, viz. access, entitlement and empowerment. Poor people are denied of access to the sources of food, income and social prestige. As a result they don't feel empower. The legal and operational entitlements are the means to combine and catalyze the aspects of access and empowerment. For e.g. the entitlement hand by the woman helps and joins access to both food and income. So, the basic synergy between poverty and hunger has been structurally configured by this three elements i.e., access, entitlement and empowerment.



The synergy between poverty and silence has been created through the negative features of inadequate information, unequal participation and voicelessness. Inadequate information makes people hesitant and confused. Hence they feel shaky and go voiceless. Unequal participation means both visible and invisible barriers to participation and decision making. So this will lead to poverty, and of course the voicelessness or silence.



Hunger and silence have got diatictive relationship. A hungry child will keep screening for certain type for food and after that and being denied of food he will go asleep or voiceless. Hunger generates voice and calumniates with voicelessness and this synergy has been supported by the negative features like unaccomodation, dejection and poor income. A man with poor income has to face dejection and a dejected man has to face intolerable trauma and a man gone traumatized as supposed to lose the voices and accept the serene choice to go voiceless.



All the characteristics of a hungry poor people do culminate in the valley of silence. Poverty is the only reason for hunger. Poverty with hunger does provide a covalence of social deception and leads to a trauma of dejection. Dejection is a kind of rejection which is driven to a social lethality. So, poverty, hunger and silence are forming what we may brand as a cultural chilet to combine the three heinous components of the society, i.e. poverty-hunger-silence.

Reducing poverty starts with children.

More than 30 per cent of children in developing countries – about 600 million – live on less than US $1 a day.

Every 3.6 seconds one person dies of starvation. Usually it is a child under the age of 5.

Poverty hits children hardest. While a severe lack of goods and services hurts every human, it is most threatening to children's rights: survival, health and nutrition, education, participation, and protection from harm and exploitation. It creates an environment that is damaging to children's development in every way – mental, physical, emotional and spiritual.

More than 1 billion children are severely deprived of at least one of the essential goods and services they require to survive, grow and develop.

Getting girls to school: Some 13 per cent of children ages 7 to 18 years in developing countries have never attended school. This rate is 32 per cent among girls in sub-Saharan Africa (27 per cent of boys) and 33 per cent of rural children in the Middle East and North Africa. Yet an education is perhaps a child's strongest barrier against poverty, especially for girls. Educated girls are likely to marry later and have healthier children. They are more productive at home and better paid in the workplace, better able to protect themselves against HIV/AIDS and more able to participate in decision-making at all levels. Additionally, this UNICEF activity furthers Goals 2 and 3: universal primary education and gender equality.

Assisting in water and sanitation improvement: One in three children in the developing world – more than 500 million children – have no access at all to sanitation facilities. And some 400 million children, one in five, have no access to safe water. Meanwhile, unsafe water and sanitation cause about 4,000 child deaths per day.

Conflicts are most frequent in poor countries, especially in those that are ill governed and where there are sharp inequalities between ethnic or religious groups. An environment of unrest heightens the risk of abduction, sexual violence and exploitation of children, as well as the struggle for shelter, education and survival. Children are made to become terrorists and help terrorism. Some 221 million people in India and 142 million in China are still chronically or acutely malnourished.

More than half of undernourished people, 60 percent are found in Asia and the Pacific. Thirty per cent of infants born in South Asia in 2003 were underweight, the highest percentage in the world.

Useful facts and figures on World Poverty (Shah, 2009):

1. Nearly a billion people entered the 21st century unable to read a book or sign their names.

2. Less than one per cent of what the world spent every year on weapons was needed to put every child into school by the year 2000 and yet it didn't happen.

3. Infectious diseases continue to blight the lives of the poor across the world. An estimated 40 million people are living with HIV/AIDS, with 3 million deaths in 2004. Every year there are 350–500 million cases of malaria, with 1 million fatalities: Africa accounts for 90 percent of malarial deaths and African children account for over 80 percent of malaria victims worldwide.

4. Water problems affect half of humanity:

    - Some 1.1 billion people in developing countries have inadequate access to water, and 2.6 billion lack basic sanitation.

    - Almost two in three people lacking access to clean water survive on less than $2 a day, with one in three living on less than $1 a day.

    - More than 660 million people without sanitation live on less than $2 a day, and more than 385 million on less than $1 a day.

- Access to piped water into the household averages about 85% for the wealthiest 20% of the population, compared with 25% for the poorest 20%.
- 1.8 billion people who have access to a water source within 1 kilometre, but not in their house or yard, consume around 20 litres per day. In the United Kingdom the average person uses more than 50 litres of water a day flushing toilets (where average daily water usage is about 150 litres a day. The highest average water use in the world is in the US, at 600 litres day).
- Some 1.8 million child deaths each year as a result of diarrhea.
- The loss of 443 million school days each year from water-related illness.
- Close to half of all people in developing countries suffering at any given time from a health problem caused by water and sanitation deficits.
- Millions of women spending several hours a day collecting water.
- To these human costs can be added the massive economic waste associated with the water and sanitation deficit. The costs associated with health spending, productivity losses and labour diversions are greatest in some of the poorest countries. Sub-Saharan Africa loses about 5% of GDP, or some $28.4 billion annually, a figure that exceeds total aid flows and debt relief to the region in 2003.

5. Number of children in the world: 2.2 billion.

   Number in poverty: 1 billion (every second child).

For the 1.9 billion children from the developing world, there are:

- 640 million without adequate shelter (1 in 3)
- 400 million with no access to safe water (1 in 5)
- 270 million with no access to health services (1 in 7)

Children out of education worldwide: 121 million

Survival for children:

Worldwide,

- 10.6 million died in 2003 before they reached the age of 5 (same as children population in France, Germany, Greece and Italy)
- 1.4 million die each year from lack of access to safe drinking water and adequate sanitation.

Health of children:

Worldwide,

- 2.2 million children die each year because they are not immunized

- 15 million children orphaned due to HIV/AIDS (similar to the total children population in Germany or United Kingdom)

6. Rural areas account for three in every four people living on less than US$1 a day and a similar share of the world population suffering from malnutrition. However, urbanization is not synonymous with human progress. Urban slum growth is outpacing urban growth by a wide margin.
7. Approximately half the world's population now lives in cities and towns. In 2005, one out of three urban dwellers (approximately 1 billion people) was living in slum conditions.
8. In developing countries some 2.5 billion people are forced to rely on biomass—fuel wood, charcoal and animal dung—to meet their energy needs for cooking. In sub-Saharan Africa, over 80 percent of the population depends on traditional biomass for cooking, as do over half of the populations of India and China.
9. Indoor air pollution resulting from the use of solid fuels [by poorer segments of society] is a major killer. It claims the lives of 1.5 million people each year, more than half of them below the age of five: that is 4000 deaths a day. To put this number in context, it exceeds total deaths from malaria and rivals the number of deaths from tuberculosis.
10. In 2005, the wealthiest 20% of the world accounted for 76.6% of total private consumption. The poorest fifth just 1.5%: The poorest 10% accounted for just 0.5% and the wealthiest 10% accounted for 59% of all the consumption.
11. 1.6 billion people — a quarter of humanity — live without electricity:

Breaking that down further:

Number of people living without electricity	
Region	**Millions without electricity**
South Asia	706
Sub-Saharan Africa	547
East Asia	224
Other	101

12. The GDP (Gross Domestic Product) of the 41 Heavily Indebted Poor Countries (567 million people) is less than the wealth of the world's 7 richest people combined.
13. World gross domestic product (world population approximately 6.5 billion) in 2006 was $48.2 trillion in 2006.

- The world's wealthiest countries (approximately 1 billion people) accounted for $36.6 trillion dollars (76%).
- The world's billionaires — just 497 people (approximately 0.000008% of the world's population) — were worth $3.5 trillion (over 7% of world GDP).
- Low income countries (2.4 billion people) accounted for just $1.6 trillion of GDP (3.3%).
- Middle income countries (3 billion people) made up the rest of GDP at just over $10 trillion (20.7%).

14. The world's low income countries (2.4 billion people) account for just 2.4% of world exports.
15. The total wealth of the top 8.3 million people around the world "rose 8.2 percent to $30.8 trillion in 2004, giving them control of nearly a quarter of the world's financial assets."
16. For every $1 in aid a developing country receives, over $25 is spent on debt repayment.
17. 51 percent of the world's 100 hundred wealthiest bodies are corporations.
18. The wealthiest nation on Earth has the widest gap between rich and poor of any industrialized nation.
19. The poorer the country, the more likely it is that debt repayments are being extracted directly from people who neither contracted the loans nor received any of the money.
20. In 1960, the 20% of the world's people in the richest countries had 30 times the income of the poorest 20% — in 1997, 74 times as much.
21. An analysis of long-term trends shows the distance between the richest and poorest countries was about:
    - 3 to 1 in 1820
    - 11 to 1 in 1913
    - 35 to 1 in 1950
    - 44 to 1 in 1973
    - 72 to 1 in 1992
22. "Approximately 790 million people in the developing world are still chronically undernourished, almost two-thirds of whom reside in Asia and the Pacific."
23. For economic growth and almost all of the other indicators, the last 20 years [of the current form of globalization, from 1980 - 2000] have shown a very clear decline in progress as compared with the previous two decades [1960 - 1980]. For each

indicator, countries were divided into five roughly equal groups, according to what level the countries had achieved by the start of the period (1960 or 1980). Among the findings:

- Growth: The fall in economic growth rates was most pronounced and across the board for all groups or countries.
- Life Expectancy: Progress in life expectancy was also reduced for 4 out of the 5 groups of countries, with the exception of the highest group (life expectancy 69-76 years).
- Infant and Child Mortality: Progress in reducing infant mortality was also considerably slower during the period of globalization (1980-1998) than over the previous two decades.

24. Education and literacy: Progress in education also slowed during the period of globalization.
25. A mere 12 percent of the world's population uses 85 percent of its water, and these 12 percent do not live in the Third World.

26. Consider the global priorities in spending in 1998

Global Priority	**$U.S. Billions**
Cosmetics in the United States	8
Ice cream in Europe	11
Perfumes in Europe and the United States	12
Pet foods in Europe and the United States	17
Business entertainment in Japan	35
Cigarettes in Europe	50
Alcoholic drinks in Europe	105
Narcotics drugs in the world	400
Military spending in the world	780

27. And compare that to what was estimated as *additional* costs to achieve universal access to basic social services in all developing countries:

Global Priority	**$U.S. Billions**
Basic education for all	6
Water and sanitation for all	9
Reproductive health for all women	12
Basic health and nutrition	13

EFFECTS OF POVERTY

Health/Well-being

Children born into poverty are more likely to have a lower birth weight, high infant mortality and poorer health than better off children.

Housing and Homelessness

Conditions such as homelessness and chronic over-crowding significantly impact upon a child's physical, mental and social development and well-being. Dampness, mould and condensation can cause a range of illnesses such as asthma and allergies.

Debt

People on low incomes often experience debt. A third of households with incomes of less than Rs.2,00,000 a year have problems with debt. Costs of debt repayments often result in families going without essential items. Other costs of debt are those incurred on health, relationships and quality of life.

Educational attainment

The correlations between poverty, social class and poor educational experience and attainment have been clearly established. Poverty affects the likelihood of progressing through school to attain formal educational qualification.

Crime

The areas most affected by crime and poor investment in infrastructure are the very areas where the poorest children live and are brought up.

Participation

Poverty impacts

- The opportunities for children and young people to participate fully in their communities to engage in social activities
- Their education and training and transitions to independence.
- Children and young people living in poor households and their families often experience difficulties in accessing and benefiting from services.

VULNERABILITY- POOR POPULATION

Poor children /youth and population are vulnerable to one or more of the following;

- Poor health often leading to malnutrition and death
- Adopt demeaning professions like begging and rag pickers etc.

More vulnerable to natural disasters like floods, drought and earthquakes etc.

Multidimensional nature of poverty

When poor people speak about well-being, they speak about the material, social, physical, psychological, and spiritual dimensions, in addition to security and the freedom of choice and action. In Ethiopia, an older woman said, "a better life for me is to be healthy and peaceful and to live in love without hunger." In Russia, "well-being is life free from daily worries about lack of money." A Brazilian said well-being is achieved "when there is cohesion, no quarrels, no hard feelings, happiness, peace with life." In Thailand, well-being was simply defined as "happiness: it is found in peace and harmony in the mind and in the community."

Conversely, poverty and ill-being are the lack of material well-being, insecurity, social isolation, psychological distress, and lack of freedom of choice and action. Not having enough to eat or possessing any assets to cope with shocks was mentioned over and over again. A poor woman in Egypt said, "A poor person is a person who does not own anything that provides him with a permanent source of living. If he has a permanent source of income, he will not ask for other people's assistance." With few exceptions, poor people reported that insecurity and unpredictability of life have increased in the past few years. In Russia, a poor man said, "Every day I am afraid of the next." A poor woman in a favela (slum) in Brazil said, "There is no control over anything, at any hour a gun could go off, especially at night." Poor people also stated that, unlike the rich, they did not have the luxury of long-term planning horizons. As a poor woman in Bulgaria put it, "to be poor means to live from day to day, you have no money, no hope." The new poor in the former Soviet Union countries, who had no previous experience with poverty, often expressed shame, anger, and hopelessness in discussing their present conditions.

What is Hunger?

According to Oxford English Dictionary, 1971, hunger is defined as;

- ✓ The uneasy or painful sensation caused by want of food; craving appetite.
- ✓ The want or scarcity of food in a country.
- ✓ A strong desire or craving.

What are the causes of hunger and poverty?

This is a fundamental question, with varied answers.

Poverty is the principal cause of hunger

The causes of poverty include poor people's lack of resources, an extremely unequal income distribution in the world and within specific countries, conflict, and hunger itself. As of 2008 (2004 statistics), the World Bank has estimated that there were an estimated 982 million poor people in developing countries who live on $1.25 a day or less (Chen and Ravallion. 2004). This compares to the later FAO estimate of 1.02 billion undernourished people. Extreme poverty remains an alarming problem in the world's developing regions, despite the advances made in the 1990s till now, which reduced "dollar--now $1.25 a day" poverty from (an estimated) 1.23 billion people to 982 million in 2004, a reduction of 20 percent over the period. Progress in poverty reduction has been concentrated in Asia, and especially, East Asia, with the major improvement occurring in China. In Sub-Saharan Africa, the number of people in extreme poverty has increased. The statement that 'poverty is the principal cause of hunger' is, though correct, unsatisfying. Why then are (so many) people poor?

Harmful economic systems are the principal cause of poverty and hunger

Hunger Notes believes that the principal underlying cause of poverty and hunger is the ordinary operation of the economic and political systems in the world. Essentially control over resources and income is based on military, political and economic power that typically ends up in the hands of a minority, who live well, while those at the bottom barely survive, if they do.

Conflict as a cause of hunger and poverty

At the end of 2005, the global number of refugees was at its lowest level in almost a quarter of a century. Despite some large-scale repatriation movements, the last three years have witnessed a significant increase in refugee numbers, due primarily to the violence taking place in Iraq and Somalia. By the end of 2008, the total number of refugees under UNHCR's mandate exceeded 10 million. The number of conflict-induced internally displaced persons (IDPs) reached some 26 million worldwide at the end of the year. Providing exact figures on the number of stateless people is extremely difficult But, important, (relatively) visible though it is, and anguishing for those involved conflict is less important as poverty (and its causes) as a cause of hunger. (Using the statistics above 1.02 billion people suffer from chronic hunger while 36 million people are displaced [UNHCR 2008]).

Hunger is also a cause of poverty

By causing poor health, low levels of energy, and even mental impairment, hunger can lead to even greater poverty by reducing people's ability to work and learn.

Climate change: Climate change is increasingly viewed as a current and future cause of hunger and poverty. Increasing drought, flooding, and changing climatic patterns requiring a shift in crops and farming practices that may not be easily accomplished are three key issues.

Causes of poverty can range from **war, natural disaster (such as tsunami), overpopulation to government corruption.**

EFFECTS OF HUNGER

1. Physiological:
   - Stresses
   - Energy imbalance
   - Nervous breakdown
   - Cardiac problem
   - Blindness
   - Loss of memory

2. Social:
   - Entropy
   - Conflict
   - Political unrest
   - Depression
   - Suicides

3. Economic:
   - Poor performance
   - Economic recession
   - Backwardness

4. Ecological:
   - Depletion of natural resources
   - Degradation of resources
   - Pollution
   - Global warming
   - Loss of biodiversity

Useful facts and figures on World hunger:

GLOBAL HUNGER

- 1.02 billion people do not have enough to eat - more than the populations of USA, Canada and the European Union;
  (FAO news release, 19 June 2009)
- The number of undernourished people in the world increased by 75 million in 2007 and 40 million in 2008, largely due to higher food prices;
  (FAO news release, 9 Dec 2008)
- 907 million people in developing countries alone are hungry;
  (The State of Food Insecurity in the World, FAO, 2008)
- Asia and the Pacific region is home to over half the world's population and nearly two thirds of the world's hungry people;
  (The State of Food Insecurity in the World, FAO, 2008)
- More than 60 percent of chronically hungry people are women;
  (The State of Food Insecurity in the World, FAO, 2006)

- 65 percent of the world's hungry live in only seven countries: India, China, the Democratic Republic of Congo, Bangladesh, Indonesia, Pakistan and Ethiopia. *(The State of Food Insecurity in the World, FAO, 2008)*

CHILD HUNGER

- Every six seconds a child dies because of hunger and related causes; *(State of Food Insecurity in the World, FAO, 2004)*
- More than 70 percent of the world's 146 million underweight children under age five years live in just 10 countries, with more than 50 per cent located in South Asia alone; *(Progress for Children: A Report Card on Nutrition, UNICEF, 2006)*
- 10.9 million children under five die in developing countries each year. Malnutrition and hunger-related diseases cause 60 percent of the deaths; *(The State of the World's Children, UNICEF, 2007)*
- The cost of under nutrition to national economic development is estimated at US$20-30 billion per annum; *(Progress for Children: A Report Card on Nutrition, UNICEF, 2006)*
- One out of four children - roughly 146 million - in developing countries are underweight; *(The State of the World's Children, UNICEF, 2007)*
- Every year WFP feeds more than 20 million children in school feeding programmes in some 70 countries. In 2008, WFP fed a record 23 million children. (WFP School Feeding Unit)

SILENCE (Powerlessness and Voicelessness)

"Nobody hears the poor. It is the rich who are being heard."—a discussion group of poor men and women, Egypt

"Poverty is humiliation, the sense of being dependent and of being forced to accept rudeness, insults, and indifference when we seek help."—a poor woman, Latvia

"When the poor and rich compete for services, the rich will always get priority."—a discussion group of poor men and women, Kenya.

The world looks different when viewed through the eyes of a poor person. In preparation for the *World Development Report 2000/ 2001: Attacking Poverty,* the World Bank conducted a research study that brought together the experiences of over 60,000 poor women and men from 60 countries around the world. Using open-ended qualitative and participatory research techniques, the Voices of the Poor study aimed to understand poverty from the perspective of poor people and to illuminate the human experience behind the poverty statistics.

The study establishes, first, that poverty is multidimensional and has important noneconomic dimensions; second, that poverty is always specific to a location and a social group, and awareness of these specifics is essential to the design of policies and programs intended to attack poverty; and third, that despite differences in the way poverty is experienced by different groups and in different places, there are striking commonalities in the experience of poverty in very different countries, from Russia to Brazil, Nigeria to Indonesia. Poor people's lives are characterized by powerlessness and voicelessness, which limit their choices and define the quality of their interactions with employers, markets, the state, and even nongovernmental organizations (NGOs). Institutions both formal and informal mediate and limit poor people's access to opportunities.

The defining experiences of poor people involve highly limited choices and an inability to make themselves heard or to influence or control what happens to them. Powerlessness results from multiple, interlocking disadvantages, which, in combination, make it extremely difficult for poor people to escape poverty. By and large, poor people say that insecurity of life has increased and they have not been able to take advantage of new opportunities because of corruption and a lack of connections, assets, finance, information and skills.

Many poor people define poverty as the inability to exercise control over their lives. Old men in Nigeria say, "If you want to do something and have no power to do it, it is taluchi/ poverty." Limited resources force poor people to think in terms of very short time horizons. "You can't think of the future because you can only see how to survive in the present," says a group of young adults in Ecuador. Poor people are often forced to make agonizing choices: feed the family or send children to school; buy medicine for a sick family member or feed the rest of the family; take a dangerous job or starve. In Brazil, drawing the connection between power, control, and well-being, a poor woman says, "the rich man is the one who says I am going to do it, and does it." The poor, in contrast, cannot fulfil their wishes or develop their capacities.

Low self-confidence both results from poverty and increases powerlessness and isolation from opportunity. A group of young men in Bower Bank, Jamaica, rank low self-confidence as the second biggest impact of poverty. "Poverty makes us not believe in ourselves; we hardly leave the community. Not only are we not educated but we don't have a street-wise education."

The study of Philippines highlighted the causes of deliberate silence by some 'elite society' while a large marginal communities are reeling under object poverty and stint of hunger. The silence against discrimination and impoverishment, agony and humiliation is amounting to the worsening of the situation and is pulling down to a venomous consequence. So, what is needed is the sharing of this stigma and agony and raising a voice against these biases and curse of society feel the agony and share the ignominy of the helpless people with more and more awareness and communication might be the much needed intervention.

Consequences of being silent or uninformed leads to confusion, difficult to access, difficult to conclude, difficult to motivate, digitized and not qualified, have potential threats and experience trauma or shock.

Then why people are uninformed? The poorer populations are mostly secluded from the rest of the privileged population; another reason for being uninformed is that they are scared of intermingling, not being connected, opted for being uninformed, alienated, potential threats.

Poverty in India:

Despite the economic growth and outsourcing of foreign jobs by western companies to India, majority of the Indian population still wallow in extreme poverty and disease. Behind India's new-found economic strength, the World Bank estimates that 456 million (42% of the total Indian population) now live under the global poverty line of $1.25 per day (Anon, 2005). This means that a third of the global poor now reside in India. Government figures may indicate a reduction in poverty. But the truth is, with increasing global food prices, poverty is spreading everywhere like a swarm of locust. Conditions are worst in the rural areas where close to 70% of India's population resides (Ramakrishnan, 2009). 40-50% of the population in Bihar and Orissa live below the poverty line (Anon, 2007)

India is not be rated as the most corrupt country in the world as far as the figures go but corrupt practices exist here in India like any other place in Africa. Public school teachers rarely go for classes but still get paid at the end of the month. The ministry of education in India is struggling to identify thousands of so-called ghost-worker names of teachers that only exist on payroll – yet victory is far-fetched.

The streets of major cities in India, like the rural areas, are populated by persons who can't even get the minimum amount of calories that is required for survival due to the low quality of food they eat. To many Indians, basic amenities such as proper sanitation, portable water, and health care are luxuries they can only get in their dreams. Many young girls have resorted to prostitution as a way to escape from poverty and provide for their families. Increasing number of girls are dropping out of schools to look for jobs. In some extreme cases, parents force their girls out of school themselves. This ugly trend spells doom for India's economy in the future (Anon, 2007).

As families cut back at the number, quality and quantity of food they consume per day; meat, a source of protein, is no more an option in the menu of many families. People now opt for less nutritional meals (just to put something in the stomach) which has a bad effect on child's growth. Meanwhile, it's estimated that 40% of children in India are suffering from stunted growth. Millions of people in India are unable to meet these basic standards, and according to government estimates, in 2007 there were nearly 220.1 million people living below the poverty line.

A number of factors are responsible for poverty in the rural areas of India. Rural population primarily depends on agriculture, which is highly dependent on rain patterns and the monsoon season. Inadequate rain and improper facilities can obviously cause low, or in some cases no production of crops. Additionally, the Indian family unit is often large, which can amplify the effects of poverty. Also, the caste system still prevails in India, and this is also a major reason for rural poverty – people from lower caste are often deprived of a number of facilities and opportunities. The government has planned and implemented poverty eradication programme, but the benefits of all these programme have yet to reach the core of the country (Anon, 2007).

The phenomenal increase in the city populations is the main reason for poverty in the urban areas of India. A major portion of this additional population is due to the migration of the rural families from villages to cities. This migration is mainly caused by poor employment opportunities in villages. This situation is exacerbated by the fact that there are few job opportunities in the urban areas of India.

Since 1970, the Indian government has implemented a number of programs designed to eradicate poverty, and has had some success with these programs. The government has sought to increase the GDP through different processes, including changes in industrial policies. There is also a Public Distribution System, which has been somewhat effective so far. Other programs include the Integrated Rural Development Programme, Jawahar Rozgar Yojana, the Training Rural Youth for Self Employment (TRYSEM) and to the credit of the government, other on-going initiatives (Anon, 2007).

A comparison of Below Poverty Line (BPL) population (in %) between rural and urban areas of India (2004-05)

Sl. no	Indian states	Rural Poverty					Urban Poverty					Rural and Urban Combined
		SC	ST	OBC	Others	Over all	SC	ST	OBC	Others	Over all	
1	Andhra Pradesh	15.4	39.5	9.5	4.1	11.2	39.9	50.0	28.9	20.6	28.0	15.8
2	Assam	27.7	14.1	18.8	25.4	22.3	8.6	4.8	8.6	4.2	3.3	19.7
3	Bihar	64.0	53.3	37.8	26.6	42.1	67.2	57.2	41.4	18.3	34.6	42.4
4	Chhattisgarh	32.7	54.7	33.9	29.2	40.8	52.0	41.0	52.7	21.4	41.2	40.9
5	Delhi	0.0	0.0	0.0	10.6	6.9	35.8	9.4	18.3	6.4	15.2	14.7
6	Gujarat	21.8	34.7	19.1	4.8	19.1	16.0	21.4	22.9	7.0	13.0	16.8
7	Haryana	26.8	0.0	13.9	4.2	13.6	33.4	4.6	22.5	5.9	15.1	14.0
8	Himachal Pradesh	19.6	14.9	9.1	6.4	10.7	5.6	2.4	10.1	2.0	3.4	10.0
9	Jammu & Kashmir	5.2	8.8	10.0	3.3	4.6	13.7	0.0	4.8	7.8	7.9	5.4
10	Jharkhand	57.9	54.2	40.2	37.1	46.3	47.2	45.1	19.1	9.2	20.2	40.3
11	Karnataka	31.8	23.5	20.9	13.6	20.8	50.6	58.3	39.1	20.3	32.6	25.0
12	Kerala	21.6	44.3	13.7	6.6	13.2	32.5	19.2	24.3	7.8	20.2	15.0
13	Madhya Pradesh	42.8	58.6	29.6	13.4	36.9	67.3	44.7	55.5	20.8	42.1	38.3
14	Maharashtra	44.8	56.6	23.9	18.9	29.6	43.2	40.4	35.6	26.8	32.2	30.7
15	Orissa	50.2	75.6	36.9	23.4	46.8	72.6	61.8	50.2	28.9	44.3	46.4
16	Punjab	14.6	30.7	10.6	2.2	9.1	16.1	2.1	8.4	2.9	7.1	8.4
17	Rajasthan	28.7	32.6	13.1	8.2	18.7	52.1	24.1	35.6	20.7	32.9	22.1
18	Tamil Nadu	31.2	32.1	19.8	19.1	22.8	40.2	32.5	20.9	6.5	22.2	22.5
19	Uttar	44.8	32.4	32.9	19.7	33.4	44.9	37.4	36.6	19.2	30.6	32.8

	Pradesh											
20	Uttarakhand	54.2	43.2	44.8	33.5	40.8	65.7	64.4	46.5	25.5	36.5	39.6
21	West Bengal	29.5	42.4	18.3	27.5	28.6	28.5	25.7	10.4	13.0	14.8	24.7
22	*All India	36.8	47.2	26.7	16.1	28.3	39.9	33.3	31.4	16.0	25.7	27.5

Source: Planning Commission 2004-05.

Hunger in India:

Nearly 21.1% of the entire rural population and 15% of the urban population of India exists in this difficult physical and financial predicament. Despite its economic successes**,** India leads the world in hunger. The Global Hunger Index (GHI) is a tool for regularly tracking the state of global hunger and malnutrition developed by International Food policy Research Institute (IFPRI) shows that India has made a slow progress towards ensuring food security (Hazra, 2009). According to the 2008 Global Hunger Index, which is calculated by the International Food Policy Research Institute (IFPRI), India has over 200 million people who are food insecure - in other words, who are not sure where their next meal is coming from. India is the second most populated country in the world. With a population of 1.173m, the hungry make up 19% or one in five of the country (Ramakrishnan, 2009). The percentage is probably better than it was fifty years ago, but the absolute number is growing.30% of babies in India are born underweight. Malnutrition accounts for nearly 50% of child deaths in India. 70% of children (under 5 years old) suffer from anaemia and more than 80% don't get vitamins supplement (WFP, 2009). Statistics show that 2.1 million children under 5 years old die of malnutrition yearly. 43% of Indian's children under 5 are underweight (BMI<18.5), the highest in the world as of 2008 (Sinha, 2009).

From a nation dependent on food imports to feed its entire population, India today is not only self-sufficient in grain production, but also has a substantial reserve. The progress made by agriculture in the last four decades has been one of the biggest success stories of free India. Agriculture and allied activities constitute the single largest contributor to the Gross Domestic Product, almost 33% of it. Agriculture is the means of livelihood of about two-thirds of the work force in the country. It is true that the country now produces enough food to feed its entire population. However this upbeat version of the food situation in India neglects the reality of widespread chronic malnutrition in the country. The country is the second largest food producer in the world, yet over 300 million people go without two square meals a day. Ironically, food worth Rs.58, 000 crore (by the Government's own admission) is wasted every year. The reason is that the country processes just 2% of the produce from its $182 billion food industry. India could feed its entire population, but still India had among the highest numbers of hungry children in the world and placed in 66th position out of 88 countries in the GHI 2008. Nearly half of the population still suffers from chronic hunger and under-nutrition (Hazra, 2009).

The results of the Indian State Hunger Index 2008 (constructed in a similar fashion as the GHI) highlight the continued overall severity of the hunger situation in India, while revealing the variation in hunger across states within India. It is indeed alarming that not a single state in India is either low or moderate in terms of its index score; most states have a serious hunger problem, and one state, Madhya Pradesh, has an extremely alarming hunger problem. This fact is very much reflective from the following table:

The Indian States at the Hunger Index and its underlying components:

State	Prevalence of calorie under-nourishment (%)	Proportion of underweight children below five years (%)	Under-five mortality rate (deaths per hundred)	India State Hunger rank
Punjab	11.1	24.6	5.2	1
Kerala	28.6	22.7	1.6	2
Andhra Pradesh	19.6	32.7	6.3	3
Assam	14.6	36.4	8.5	4
Haryana	15.1	39.7	5.2	5
Tamil Nadu	29.1	30.0	3.5	6
Rajasthan	14.0	40.4	8.5	7
West Bengal	18.5	38.5	5.9	8
Uttar Pradesh	14.5	42.3	9.6	9
Maharashtra	27.0	36.7	4.7	10
Karnataka	28.1	37.6	5.5	11
Orissa	21.4	40.9	9.1	12
Gujarat	23.3	44.7	6.1	13
Chhattisgarh	23.3	47.6	9.0	14

Bihar	17.3	56.1	8.5	15
Jharkhand	19.6	57.1	9.3	16
Madhya Pradesh	23.4	59.8	9.4	17
India	20.0	42.5	7.4	

Source: India State Hunger Index Report, 2008.

The sequel of Poverty, Hunger and Silence:

The findings presented by Doona E. Beegle (2003) revealed that poor people had no choice but to experience great shame and grieving in poverty. They believe that outsiders perceived poverty to be their fault. The school children, hailing from poor family had to face physical, emotional, sociological and economic barriers to literacy and education at all stages of their lives. Of particular note were lack of jobs, basic needs, including house, food and health care, money and control over one's life.

Progress amidst challenges:

Some countries have made progress meeting this Goal, but success is mixed. India and China are on track to meet the income target at least, but in a classic example of national disparities. Most sub-Saharan African countries will likely miss both targets. The region has 204 million hungry and is the only region of the world where hunger is increasing. More than 40 per cent of Africans cannot even get sufficient food on a day-to-day basis. The target set at the 1996 World Food Summit was to halve the number of undernourished people by 2015 from their number in 1990-92. (FAO uses three year averages in its calculation of undernourished people.) The (estimated) number of undernourished people in developing countries was 824 million in 1990-92. In 2009, the number had climbed to 1.02 billion people. The WFS goal is a global goal adopted by the nations of the world; the present outcome indicates how marginal the efforts were in face of the real need.

So, overall, the world is not making progress toward the world food summit goal, although there has been progress in Asia, and Latin America and the Caribbean.

The Strategic Implications:

- ✓ Integration of development approaches is an essential precondition for achieving comprehensive development.

- ✓ Three basic dimensions of integrated development are food, poverty and voice. To fight against poverty these three dimensions need to be perceived isochronously.

- ✓ So, hunger should be seen as the worst consequences to the diadictic interactions between poverty and silence.

- ✓ People's Forum should be created wherein the poor will find their voices reaching the decision making end, food will be made accessible across the line of demarcation and poverty will be ameliorated and not be allowed to make a revisit.

- ✓ Social and institutional networking along with a capable leadership can accept the challenges to face the dangerous trident of hunger, poverty and silence.

- ✓

✓ Conclusion:

In concluding the entire complex discourses it is suggested that the poverty alleviation approaches should not be considered an extremely linear phenomena. It has got social, economic and cultural dimensions. To fight against poverty the poor people must be made uttering and voicing. In doing this they must be tuned to the web of communication and network of information through the nutrition, food, social esteem, cultural support and policy stand as well. At the same time anyone of it, either food or money, will go ephemeral in solving this problem. It is true and real that poverty is associated with silence and for the silent and hungry people ,poverty is
systemic.

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KVK : AN IMPACT STUDY ON TRANSFORMING SOCIAL ENTROPY IN ODISSA, INDIA

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Abstract

The perception of need is the basic psychological input to generate demand for training intervention through KVK. Once the needs are properly perceived and effectively mobilized for the socialization of technology the performance of KVK becomes more meaningful and goal-oriented. The present paper has examined the differential need perception for 'KVK intervention in a dichotomized respondent system i.e. adopted and non-adopted villages. The results suggest that the direct impact of the variables media exposure (X_{12}), Scientific aspiration (X_{11}) and social participation (X_{10}) have been substantial, while in non-adopted villages, the variables income(X_8), education (X_2)and age (X_6) have recorded substantial direct impacts on training need perception.

Key words : *Need, perception, direct effect, income, media exposure.*

Introduction:

Krishi Vigyan Kendra (KVK) has been conceived as the lighthouse of agriculture and rural development in India. This is Comprehensive Institutional arrangement to catalyse the Transformation Process, a process that transform a state and still rural society into a vibratina and economically strong community.

In India right now more than 584 KVKs are working as centered for capacity building, specially for women. It also synthesizes humane capacity with technology viability. It is basically a technology socialization process.

Any technology socialization process, again, assumes the characters of the Kinetic of Social Osmosis.

In any phenomenon of social osmosis technology passes through accomdation, assimilation and acculturation process (Acharaya, S & Pradhan,K2002).

The Technological Socialization Process for farm women through KVK has turned to be a unique social process. In the first stage KVK accommodate the innovations or adoptable technologies in the micro-farming system through a capacity building of farm women. It is better branded as reconina or cognitive phase of technology socialization process. After that the assimilation process will go on happening by a self governed osmosis process where in the "exotic" ideas would be internalized in gender term and perspectives the role for women in India in agriculture are so far been Polymorphic which is ranging from child care to seed sowing, nutritional care of the family to post harvest operation.

The word "training" as referred in Webster (1947) means: as an act, process, or method of one who trains. Training is a specialized and practical form of education (Plenty *et* a/. 1948). He further stated that training in commerce and industry is a specialized and very practical form of education. Basically, it prepares people to do their jobs well. To accomplish this it develops the skills that make for rapid, effective work, the knowledge. That is meant for intelligent actions and attitudes that bring willing cooperation with fellow employees and with management.

Sobha (2001) emphasized that training and technologies information has to be provided to the farm women to improve their skills, level of decision making and effective participation.

Padmanabhan (2001) emphasized the need for empowerment of rural women in agriculture through effective training and extension services arises from the gradual decrease in the availability of cultivable land, increasing population pressure and growing environmental degradation which have far reaching implications for food and nutritional security.

Majhi and Patra (1996) suggested that special training programme should be conducted to develop the scientific orientation, entrepreneurial abilities and working knowledge of farm women on agricultural activities. Programmes on sericulture, lac cultivation, mushroom cultivation apiculture etc. may be incorporated particularly for tribal farm women to develop the livelihood.

OBJECTIVES:

1. Estimation of impact of KVK in both adopted and non-adopted villages in Odisssa
2. Selection of predictor and predicted variables in terms of the mentioned estimation
3. Analysis and elucidation of causal relation between impact on kvk and the predictor variables
4. To derive certain mciro- level policies in favour of proper KVK functioning

METHODOLOGY

The researcher took the assistance of the concerned training organiser and programme co-ordinator of the ten selected K.V.K's for the identification of trained and untrained farm women from the available record of the village adopted under women empowerment from the adopted village 15 trained and from the non-adopted village 15 untrained farm women were selected with help a random sampling technique. Thus from the ten KVKs 150 trained and 150 untrained framwomen were selected as sample respondent of the study

Statistical Tools

The Mann-whitney u-test, Rank order co-relation, Regression analysis, Canonical discriminant analysis has been done.

Variables:

Twelve predictor characters were selected viz. Age(x_1), Education(x_2), Occupation(x_3), Family size (X_4), Holding size (x_5), House status (x_6), Material possession (x_7), Income (x_8), Farm power (x_9) social participation(x_{10}),Scientific aspiration (X_{11}),Media exposure(x_{12}), for acsessing the relationship with the predicted character Training need(y_1).

Results and Discussion:

Table-1 Presents the Mann-Whitney U - test for comparing between two sets of data from adopted and non-adopted villages in terms of perceived training need.

It has been found that the adopted and non-adopted villages distinctly, differ, while the comparison is based on perceived training need, components of crop-production, Horticulture, plant protection Argil. Engineering, Fishery, Extension Education.

TABLE -1: Perceived Training Need (Yi): Impact Of Kvk On Both Adopted And Non-Adopted Villages

Variable	**NA**	**A**	**Test Statistics**					
			Mann-Whitney U	**Wilcoxon W**	**Z**	**Sig**	**Kolmogorov-smirnov Z**	**Sig.**
Crop Production (Y_1)	1.83	2,08	928.00	2203.00	-2.23	**0.03**	1.00	0.27
Horticulture. (Y_2)	1.74	2,07	754.50	2029.50	-3.43	**0.00**	1.80	**0.00**
Plant protection (Y_3)	1.63	2.00	762.50	2037.50	-3.38	**0.00**	1.80	**0.00**
Ag. Engineering (Y_4)	1.58	1.88	922.50	2197.50	-2.27	**0.02**	1.20	0.11
Animal Husbandry (Y_5)	1.77	1.91	1049.00	2324.00	-1.39	0.17	1.30	0.07
Fishery(Y_6)	1.26	1.68	899.50	2174.50	-242	**0.02**	1.10	0.18
Forestry (Y7)	2.08	2.03	1001.50	2276.50	-1.72	0.09	1.20	0.11
Extension Education (Y_8)	4.11	2.46	889.50	2164.50	-2.51	**0.01**	1.50	**0.02**
Home Science (Y_9)	2.11	2.14	1136.00	2411.00	-079	0.43	0.90	0.39
Training need Score (y_1)	2.01	2.03	859.50	2134.50	-269	**0.01**	1.30	0.07

(The level of significance have accepted as less than 0.05)

Rank order corrrelation: adopted and non-adopted villages: components of perceived training need (y_1) and 12 predictor characters.

Table 2: Present the Rank order correlation (Spearman's rho) to display the correlation between the sub components and the predictor variables.

It has been found that the predictor variable age has been significantly correlated with the subcomponents (Training Need): Crop production, Horticulture, Plant protection, Fishery, Extension Education and perceived training need score.

Education has been found to be correlated with forestry components, occupation has been tuned to age.

Family size has recorded no such significant correlation with any other component. So training need has became a universal perception below the farm size category per cent.

Holding size has been significant for associating a training need in Horticulture and Animal husbandry.

The House status has been predominately tuned to plant protection, Agril. Engineering, Forestry and Training need score.

The Material possession has been significantly tuned Horticulture. Income has come significantly attuned to plant protection and Agril. Engineering to imply that an optimum income support is essential to elicit the need for training in these two areas.

Scientific aspiration has well been tuned to crop production and Horticulture in terms of eliciting the training need.

Media exposure has gone significantly attuned to all the sub components excepting crop production and Home sieve science.

TABLE-2: Rank Order Correlation: Components Of Perceived Training Need (y_1) And 12 Predictor Characters:

Variable	Correlation		Spearman's rho							
	CROPPRDN	HORT	PLANT PROT	AGENG	ANIMHUS	FISHERY	FORESTRY	EXT-EDU	HOMESC	TN-SCORE
Age (x_1)	**0.297**	**0.3**	**0.302**	0.069	0.096	**0.208**	0.18	**0.205**	0.17	**0.244**
Education (x_2)	-0.035	0.092	0.012	0.178	0.107	0.044	**0.235**	0.074	0.033	0.108
Occupation (x_3)	**0.319**	0.054	0.091	-0.05	0.047	-0.021	-0.072	-0.07	-0.002	0.027
Family size (x_4)	0.016	-0.02	-0.05	0.096	0.188	0.077	0.128	0.181	-0.128	0.145
Holding size (x_5)	0.171	**0.218**	0.14	0.078	**0.23**	0.044	0.054	-0.001	0.038	0.092
House status (x_6)	-0.021	0.141	**0.247**	**0.269**	0.172	0.101	**0.363**	0.097	0.163	**0.246**
Material possession(x_7)	-0.016	**-0.202**	-0.063	0.029	0.108	0.025	0.121	-0.028	0.07	0.046
Income(x_8)	0.148	0.149	**0.273**	**0.197**	0.107	0.039	0.124	0.19	0.034	**0.198**
Farm power(x_9)	0.142	-0.001	0.027	0.156	0.142	-0.188	0.177	0.1	0.004	0.108
Social -participation(x_{10})	-0.038	-0.132	0.042	0.018	-0.115	-0.014	-0.04	0.126	0.04	-0.035
Scientific aspiration(x_{11})	**0.203**	**0.206**	0.093	-0.022	0.133	0.001	0.132	-0.126	0.168	0.115
Media exposure(x_{12})	0.178	**0.397**	**0.0212**	**0.361**	**0.35**	**0.265**	**0.292**	**0.198**	0.047	0.281

P<0.01, PO.05

Regression coefficient: perceived training need (y_1) and the causal variables:

Table 3 Present the regression co-efficient and has depicted that the variables Family size and Income have exerted significant impact on the perception of training need in both adopted and non-adopted village.

Table 3 Regression Coefficient: Perceived Training Need (y_1) And The Causal Variables:

Variable	Un standardized		Standardized coeffi		
	B	Std Error	Beta		Sig
Constant	0.958	0.506	-	1.894	0.061
Family Size	0.320	0.226	0.141	1.417	0.160
Income	0.191	0.111	0.171	1.720	0.089

Dependent variable training need score:

Model Summary:

R	R.Sq.	Adj.Rsq	Se (Est)
0.216	**0.046**	0.027	1.115

Family size and income behave as binary impulse of resource- diode to characterize the perception of training need and subsequently goes on operationally impacting on the generation of training need from amongst the farmwomen.

Canonical Discriminant function: unstandardized coefficients:

Table 4 Depicts the canonical discriminant Function (CDF) to imply that the variables, Social participation, Media exposure and Horticulture are selectively impacting in generating better perception of training need in adopted villages.

In non-adopted villages not a single variable does build up a strategic relation for generating training new part.

Table 4 : canonical Discriminant unstandardized coefficients Function

variable	Function
Social participation	0.355
Media exposure	1.193
Horticulture	1.515
Constant	-4.520
Eigenvalue	0.275
% of variance	100.000
Canonical correlation	0.464
Wilks lambda	0.784
Chi -square	23.425
Significant	0.000

Function At Group Controids

CODE	FUNCTION
NA	**-0.519**
A	**0.519**

Unstandardized canonical discriminate functions evaluated at group means

Table-5 presents that the path analysis for detecting the impact of exogenous variable on perceived training need (y_1) in adopted villages. It has been found that the variables Age (x_1), House status (X_6) Scientific Aspiration (x_{11}) and Media Exposure (x_{12}) have recorded substantial correlation with the perceived training needs.

This indicates that the training needs have been perceived in terms of chronological age of the respondent or the training needs have been perceived by different age categories of the farmers.

The training needs have been bestowed upon by the respective scientific aspiration nurtured by the farmer's respondent. Generally the training on technologies, complex and advanced, should be prescribed for those having higher level of scientific aspiration.

It has also been found that the training needs have shown a perceptual variation incompliance with the media exposure experienced by the respondent.

The variable Age (x_1) Social participation (x_{10}) Scientific Aspiration
(xn) and Media exposure (X_{12}) have exerted discreet direct effect on the perception of training need in adopted villages.

Social participation (x_{10}) has exerted the highest indirect effect on the perception of training need to imply its tremendous associational viscocity for imbibing the effect of other variable.

The variable House status (X_6) has routed the highest indirect effect on as many as three variables to characterise the perception of training need in the adopted village.

The residual effect being 0.5576, It is to conclude that even with the combination of twelve exogenous variable 55.76 per cent of the variance embedded in the consequent variable could not be explained.

Table - 5path analysis: total effect of exogenous variables on perceived training need (y_1) kvk adopted villages

Exogenous variables	Total effect (r)	Total direct effect (TDE)	Total indirect effect (TIE)	Substantial indirect effect		
				I	II	III
Age (X_1)	0.314	0.224	0.089	0.051 (x_6)	0.025(x_{12})	0.01 5(x_2)
Education (X_2)	0.046	-0.057	0.103	0.057(x_1)	0.038(x_{12})	0.036(x_{10})
Occupation (X_3)	-0.109	0.068	-0.177	0.078(x_{10})	0.067(X_6)	0.059(x_1)
Family size (X_4)	0.068	-0.037	0.105	0.077 (x_{11})	0.073(x_{12})	0.044(X1)
Holding size (X_5)	0.046	0.02	0.018	0.110(x_{10})	0.086(X_{11})	0.073(x_{12})
House status (X_6)	0.332	0.171	0.161	0.171 (X_6)	0.140(x_{12})	0.080(x_{10})
Material possession (X_7)	-0.040	-0.021	-0.019	0.047 (x_1)	0.041 (X_{10})	0.021 (X_7)
Income (X_8)	0.102	-0.005	0.107	0.052(x_6)	0.036(x_{12})	0.019 (x_1)
Farm power (X_9)	-0.036	0.029	-0.065	0.077(x_1)	0.067(X_{11})	0.024(x_2)
Social participation (X_{10})	0.003	0.239	-0.236	0.106 (X_{12})	0.071 (x_{11})	0.057(x_6)
Scientific Aspiration (x_{11})	0.379	0.321	0.058	0.093(x_{12})	0.053(x_{10})	0.033(X_6)
Media Exposure (X_{12})	0.459	0.409	0.050	0.073(X_{11})	0.062(x_{10})	0.059(X_6)

Residual effect 0.5576; Highest indirect effect (X_6) (3 times)

Conclusion:

The whole text of the research unleashes the measure that each of the adopted and non-adopted villages need unique approaches and interventions, to generate proper and adequate need for perception while adopted villages can make a head way in technology socialization through media exposure, income remains as a crude and crucial need for generating better need for perception. Besides the villages having morphological and metabolic differences, should be comprehensively analysed to derive the unique requirements for training intervention and perception therefrom.

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ESTIMATION OF SOCIAL ENTROPY: THE DICTUM OF NEO-MODERNISM IN AGRICULTURAL KNOWLEDGE ENVIRONMENT IN THE CONTEXT OF GLOBAL WARMING

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ABSTRACT

The creation, growth and decay of any social system, retaining myriads of life forms, have basically been the concerns and exposition of energy. Social Metabolism envisage a natural connectivity amongst physical, biological and social systems ad their under lying cybernetics. It is the flow of energy that drives the social systems generating information, applying information and transforming the present social process in to a desired social out come. Energy remains retained, shelved and configured within a cell and within a social capsule and also is subject to a ruptured release for unlashing motivations and psychological capabilities. The present paper examines the collision and collusion between imposed technologies vis-a-vis extraneous knowledge vs. intrinsic vis-a-vis in situ knowledge. The rejection of innovation, prescribed by experts, has got a reverse osmosis impact in the entire technology socialization process. Generally the rejected and discontinued technology has been branded as laggard but the logic behind rejections not esteemed properly. The farming system performance cannot be conceived as an indoor, interactive drama, rather it is a splendid disposition of social metabolism, where in the flow in and flow out of energy can be audited and monitored for sustainable farming performance of both the farmer and the manager or the both encapsulated in a single entity. The factors like cropping intensity, size of holding, education, motivation, access to information, the socialization level of technology have been found generating reticulate impact on the social osmosis through the intervening characters like, adoption, rejection and discontinuance, on and over a spectrum of prescribed innovation.

Keywords: *Social Osmosis, Social Metabolism, Entropy, Discontinuance, Laggard, Motivation, Access to Information*

INTRODUCTION

Food security and Knowledge:

The building block of civilization is knowledge. Knowledge is pursuits and acquisition, concepts and application and so on. The history of ten thousand years of agrarian civilization is basically the history of humane innovation to tame the nature and shape the life, the way we desire. From hunting economy to present day technology driven society, the role of human knowledge keeps getting exponential. The conflict between indigenous and exotic knowledge is classical as well as ephemeral. This has become worst with the process of urbanization as well as modernisation in the very system of agricultural production and management. Our extreme hegemony in favour of making farmers adopting energy intensive technology and contra-ecological approaches has made a near disastrous situation, can be branded as an entropy of knowledge and technology.

Knowledge conflict in farming system and the contra adoption process

The traditional and *in situ* knowledge are being contradicted by imported knowledge in agro-ecosystem, which again is undergoing constant reforms, adjustments and evolution. In certain cases, where indigenous knowledge keeps offering a space for social osmosis, prescribed knowledge are assimilated and acculturate. In other cases, withdrawal and non-compliances are happening simply because the initial knowledge balance, characterizing a unique social echelon has failed to assimilate exotic knowledge. These all lead to a knowledge dissonance attributing to a negative social metabolism over a slice of temporal distribution.

Indigenous knowledge, Food security and Social entropy

The problem of food security is not only the biological inaccessibility to food, but also a motivational issue which is impounded with the flow of energy from a natural ecosystem to a biological ecosystem being governed by a social ecosystem. The constant dissonance and chaos would lead to a reeling but claustrophobic situation. The defying of indigenous knowledge associated with an imposition of exotic knowledge can go a long way in the erosion of social trust and ethical strength, essential for proper socialization of technology. Where local knowledge and praxis are not logically socialized and ethically structured in a modernization process, the people down to earth will subject to face a chronic problem of motivational insecurity, isochronously with a food insecurity.

The different aspects of knowledge dissonance and the crux of social entropy in farming system

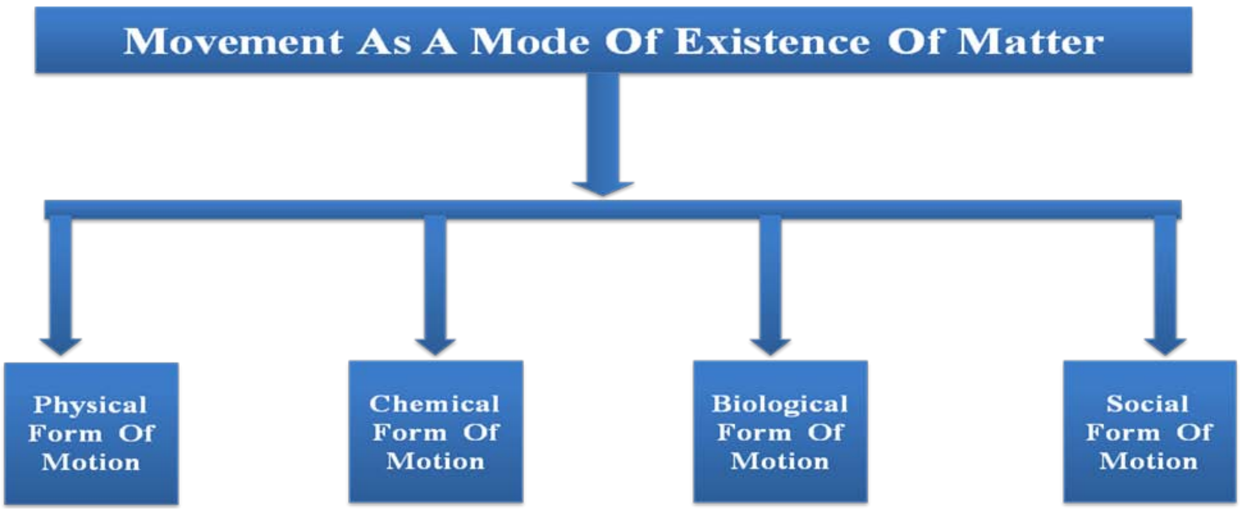
A farming system is defined as a population of individual farm systems that have broadly similar resource bases, enterprise pattern, household livelihood and constraints and for which similar development strategies and intervention can be applied. Farming system in India has been characterised with high level of adoption, rejection and discontinuance. Agriculture in India demands transfer of technology, external supply of inputs as well as knowledge, where rural people have become mere recipient of input and technology. In India in general and West Bengal in particular through the continuous imposing of knowledge and motivating the rural people a gap has been found between motivation unleashed and accomplished made and there is a gradual dissolving of the most advance societies due to intrinsic disorder that may be referred to as social entropy. This is responsible for institutional conflict, organizational disorder or social entropy. Social entropy is a macro-sociological system theory. It is a measure of the natural decay within a social system. It can be defined as the decomposition of social structure or of the disappearance of social distinctions. Social entropy is the amount of motivation unavailable for performing in system. Mitchel (2009) studied on a village (Jacobs) in 1998 through creative destruction developed and predicted the fate of communities that became the base of their development on the co-modification of rural heritage. You, L *et al.* (2006) while reporting agricultural production statistics on geopolitical and on national basis concludes that there is a need to know the status of production or productivity within specific sub regions, watersheds or agro-ecological zones. His study depicts entropy based approach to make spatially disaggregated assessments of distribution of crop production. Jen, K. A *et al.* (1999) in his multi-method field study of 92 work groups explored the three types of workgroups diversity (Social category diversity, Value diversity and informational diversity) and two moderators (task type and task interdependence) where these workgroups not only became central to organization but also presented their own intrinsic problem of coordination, motivation and conflict management.

Social equilibrium, Rural poverty and Flow of energy

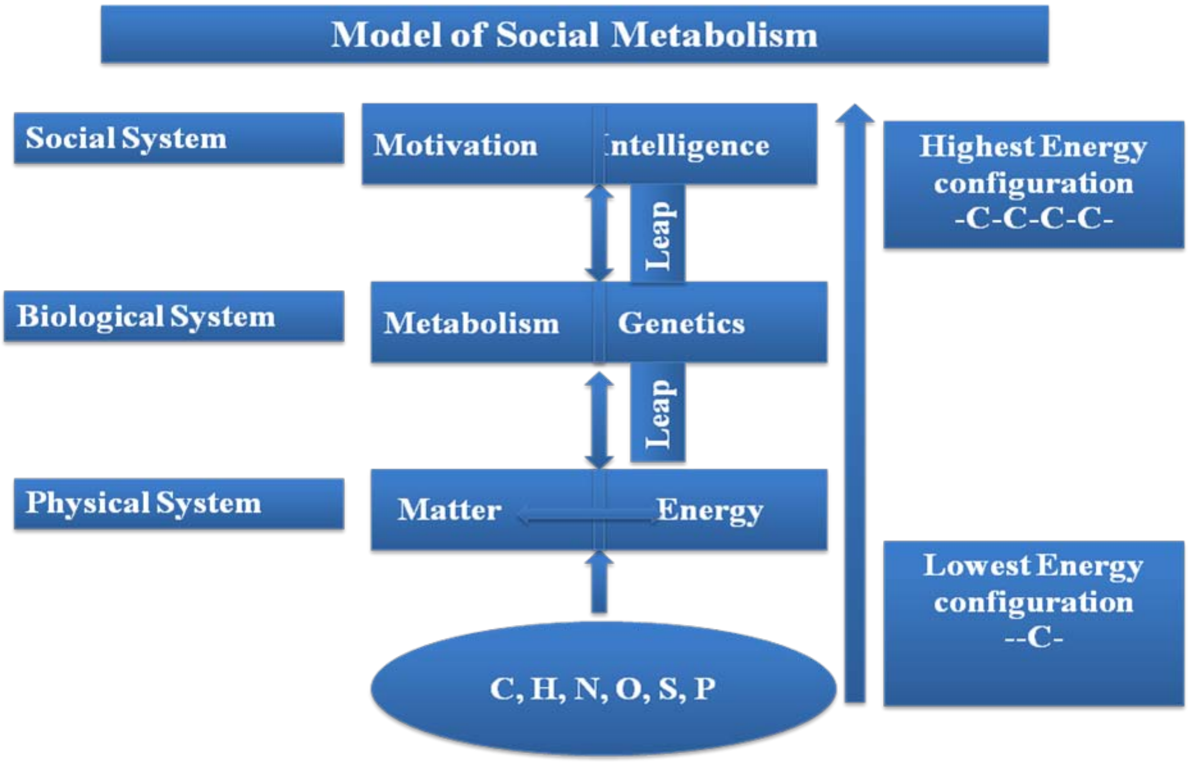
Therefore, keeping core periphery contradiction in the development process that has caused structured chaos and dissonance in view, the present research has been conducted for the prediction of the social entropy amongst the farmers from a score of socio-personal, socio-psychological and communication variation. Farming systems deals with production system and production function, it is load based, crop based, and natural resource base and thus crop productivity is a function of physical, biological and social subsistence. The stage of equilibrium, physical, biological and social is the prime concern of any system, it is more important for extension system because it aims at adding disequilibrium to a depletive function e.g. (Poverty) in order to invite neo-equilibrium (sustainable livelihood). People are more concerned about the conservation of energy that to in terms of fuel energy or electricity, power etc but seldom we speak out conservation of social energy and recycling of motivation**.** Methodology for conservation of social energy: Training; education; meditation; simulation; psychotherapy; stimulation.

Social Metabolism and Social Entropy

Every day an immense mass of the materials and the energy of nature are, through work activity, appropriated by the social body, only to be adapted to its needs, through production activity and distributed to the various parts through circulation, transformed into the social fabric by means of absorption (as for food) by both institutions and individuals, and returned into the lap of nature through the consumption of goods and bodily forces. Schaffle clearly outlined the mechanism of that social metabolism by means of which the energy and the matter existing in nature enables the social body to maintain itself. The economic and physiological exchange of material does not entail the destruction of the material and energy but, rather, it entails their re-organization into sources of energy and into institutions which make their social use possible. Basically, Schaffle applied thermodynamic principles to social exchange. According to this principle energy and matter are not destroyed but are only transformed, disorganised and then reorganised for other uses. An efficient mechanism of social metabolism can neither allow any energy to be lost nor permitting increasing entropy, would the result be crisis within the social organism itself. (Schaffle, A. 1874).

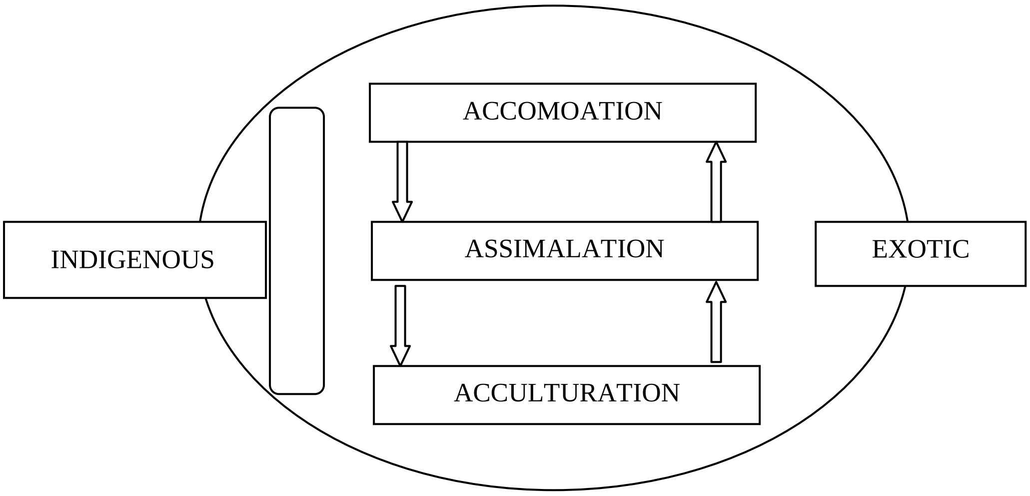


At certain stage of development, chemical process lead to formation of Protein body and on the basis of emergence of life i.e. to the biological form of motion of matter. It follows that some forms of motion of matter can turn into the other forms of motion of matter, which is reflected in the law of conservation and transformation of energy and matter. Each stage in the development of matter corresponds to a form of motion, differ qualitatively, and the highest forms of motion of matter cannot be reduced to the lowest.



Social Osmosis: The science of knowledge exchange:

Social osmosis is the indirect infusion of social, cultural knowledge. Effectively social control is diffused and by happenstance authentic experience is displaced by degrees of mediated separation before a subject acquires knowledge of a social phenomenon (Raaj K. Sah, 1990).



Knowledge always undergoes a social osmosis process to exchange, imbibe and assimilate.

KNOWLEDGE ENTROPY IN FARMING SYSTEM : ISSUES OF COMPLIANCES AND CONFLICTS

An empirical study was conducted to elicit the factors and reasons for non-compliance and conflict in the process of technology transfer, technology socialization either. In the study 73 respondents were selected randomly from 250 growers of village Ghoragaccha of Block Haringhata in Nadia district of West Bengal, India. Socio-personal variables like age (x1), Education (x2), Family education status (x3), Family size (x4), Cropping intensity (x5), Farm size (x6), Annual income in Rs/year/capita (x7), Socio-psychological variables like scientific orientation (x8), Independency (x9), Innovation proneness (x10), Risk orientation (x11), Economic motivation (x12), Orientation towards competition (x13), Attitude towards discontinuance (x14), Attitude towards rejection (x15), Communication variables like Social participation (x16), Utilization of source of information (x17), and training received (x18) as predictors, whereas, among predicted or dependent variables, Noncompliance (Y1), Disagreement (y2), Conflict (Y3), Alienation (Y4) Social Entropy (Y5) were taken. Social entropy (y5) was obtained first by multiplying all the four predicted variables y1, y2, y3, y4 and then dividing the resultant product by 4.

Data were collected directly from the farmers with the help of structured schedule through personal interview methods. Collected data from the selected farmers were analysed with the help of several statistical tools like mean, standard deviation, coefficient of variation, correlation, regression and path analysis.

A. The farmers having less innovation proneness are more vulnerable to entropy situation.

Table 1: Coefficient of Correlation: Entropy (Y5) vs. 18 Independent Variables

Sl. No.	Variables	Coefficient of Correlation
1	Age in years (x1)	-0.067
2	Education (x2)	0.033
3	Family Education status (x3)	0.115
4	Family Size (No. Of members) (x4)	-0.027
5	Cropping Intensity (x5)	0.184
6	Farm size in bigha (x6)	0.074
7	Annual Income (x7)	0.025
8	Scientific orientation (x8)	-0.121
9	Independency (x9)	-0.129
10	Innovation Proneness (x10)	-0.124
11	Risk orientation (x11)	**-0.239***
12	Economic motivation (x12)	0.007
13	Orientation towards Competition (x13)	0.085
14	Attitude towards discontinuance(x14)	0.146
14	Attitude towards Rejection (x15)	0.152
15	Social participation (x16)	-0.114
16	Utilization of Cosmopolite Sources of information (x17)	0.041
17	Training received in days in last 3 years (x18)	0.029
18	*significance of r at 5%=	0.230
19	**significance of r at 1%=	0.300

B. With the increase of income the rural people, for certain cases, are showing increasing dissonance against proposed technology, might be, there are now more exposed to choice of alternatives than before. Stepwise regression and backward elimination techniques considering highest regression coefficient for social Entropy (Y5) as dependent variable and remaining 18 variables as predictors.

Economic gain has both consolation and contradiction. Consolation generates because present problem has been resolved and contradiction simmers because whether to justify the glory over the inglorious one. The competition in agrarian society is increasing while hegemony starts ignoring the access to income by others. The sudden surplus income creates a stress in the serene and soft relationship, the binding force is family.

[Y5 = 22.17 – 2.21 X11**

R2 =0.06, R2 (adj) = 0.04, Se (estd.) =4.77

Where, Y5 is social entropy

X11 = Annual Income (Rs/year/Capita)

R= Régression Coefficient SE = Standard Error

Residual effect = 0.6902095]

C. Motivation and Social Entropy : Technical discourses vs. Farmers' voice

Economic motivation is skewed version of emotion pinpointed for economic gain, may be through competition, denial to others rights, or through a clandestine performance which again can be clever or a deceiver one. The elements of consumerism, an unhealthy competition, the other side of monolithic development has done more harms than the goods delivered by it. Innovation proneness has got profuse impact on generating competition to supersede the laggards and ultimately make them subjugated in a system hierarchy. If not properly refined every ego has got deleterious impact over the peers or the defeated ones amongst the peers. Farm size with high economic motivation has made one victorious and the others deleted ones. This has got, certainly, a catalyzing role in making social entropy a more complex hecatomb to make life confined and claustrophobic: this is what we call Social Entropy**.**

Table 2: Path Analysis for Estimating Direct, Indirect and Spurious Effect

Entropy (Y5) vs. 18 exogenous Variables							
Sl. No.	Variables	Direct effect	Indirect effect	Total effect (r)	Substantial Indirect effect		
					I	II	III
1	Age in years (x1)	**-0.02799**	**-0.03901**	**-0.067**	**0.02751 (x2)**	**-0.02179 (x6)**	**0.01990 (x10)**
2	Education (x2)	**-0.07394**	**0.10694**	**0.033**	**0.09631 (x3)**	**-0.03617(x10)**	**0.02750 (x6)**
3	Family Education status (x3)	**0.11994**	**-000494.**	**0.115**	**-0.05937 (x2)**	**0.03206 (x7)**	**-0.02759 (x9)**
4	Family Size (No. of members) (x4)	**-0.13765**	**0.11065**	**-0.027**	**0.06436 (x6)**	**-0.02452 (x8)**	**-0.1624 (x9)**
5	Cropping Intensity (x5)	**0.08339**	**0.10061**	**0.184**	**-.04264(x10)**	**0.02735(x3)**	**0.02603(x9)**
6	Farm size in bigha (x6)	**0.14627**	**-0.07227**	**0.074**	**-0.06057 (x4)**	**-0.04341 (x8)**	**0.02959 (x12)**
7	Annual Income (x7)	**0.11055**	**-0.08555**	**0.025**	**-0.05007 (x10)**	**0.04502(x12)**	**0.03478 9x3)**
8	Scientific	**-0.10436**	**-0.01664**	**-0.121**	**0.06085 (x6)**	**0.05043 (x12)**	**-0.03761 (x11)**

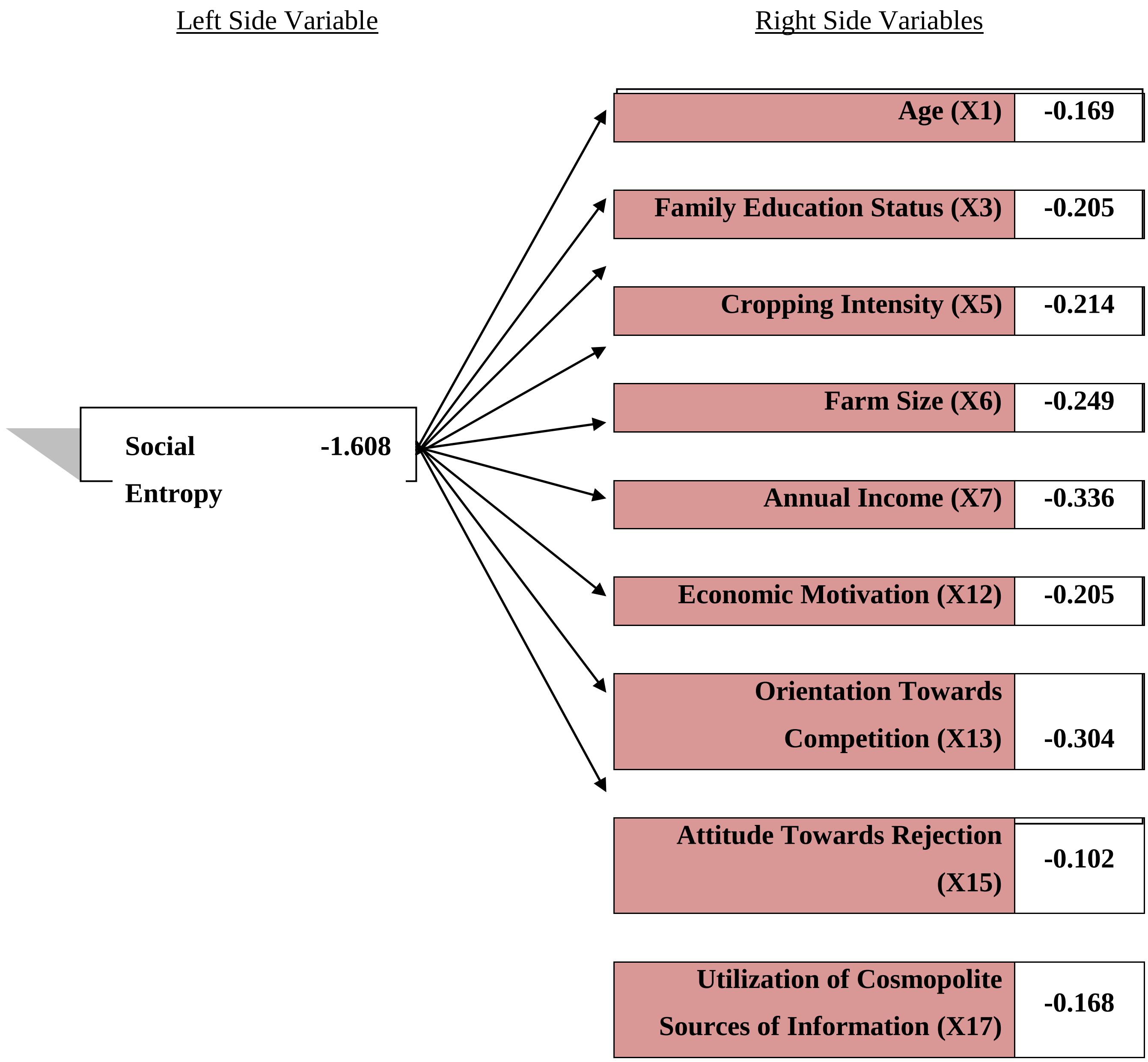
	orientation (x8)						
9	Independency (x9)	**-0.19570**	**0.0667**	**-0.129**	**0.03583 (x17)**	**0.01931 (x6)**	**0.01691 (x3)**
10	Innovation Proneness (x10)	**-0.18714**	**0.03394**	**-0.124**	**0.03505 (x7)**	**0.02804 (x12)**	**0.02435 (x3)**
11	Risk orientation (x11)	**0.12864**	**-0.05186**	**-0.239***	**0.03062 (x12)**	**-0.02356 (x15)**	**0.02311(x6)**
12	Economic motivation (x12)	**0.13456**	**-0.12164**	**0.007**	**-0.04454 (x11)**	**-0.04091 (x8)**	**-0.03869(x7)**
13	Orientation towards Competition (x13)	**0.11082**	**-0.04956**	**0.085**	**-0.01445(x4)**	**0.01404(x11)**	**-0.01295(x10)**
14	Attitude towards discontinuance (x14)	**0.10025**	**0.03518**	**0.146**	**0.02776(x8)**	**0.02200(x16)**	**-0.01957 (x9)**
15	Attitude towards Rejection (x15)	**-0.10000**	**0.05175**	**0.152**	**0.04398(x11)**	**-0.03975(x12)**	**-0.02565 (x7)**
16	Social participation (x16)	**0.10356**	**-0.014**	**-0.114**	**-0.02717(x10)**	**-0.02438(x14)**	**0.02300(x7)**
17	Utilization of Cosmopolite Sources of information (x17	**0.00234**	**-0.06256**	**0.041**	**-0.06771(x9)**	**0.02384(x6)**	**0.02100(x16)**
18	Training received in days in last 3 years (x18)	**0.00234**	**0.02666**	**0.029**	**0.02807(x3)**	**0.02326(x4)**	**-0.02056(x2)**

D. Knowledge, Motivation, Sources of Information, Family Education..... are adding entropy and chaos, gone inevitable and intrigue as a system function

It is clear from the table that family education, Economic motivation, Orientation towards competition and Attitude towards rejection has been precisely chosen for conceptualising Social Entropy. Farmers in different parts of India and here in west Bengal, are engaged in or confronted with each other to show the power or defined their rights. The ambition for earning more may deny the rights of others or a sense of flamboyant intrusion may make others feel suppressed or denied. The attitude towards rejection may not go as a placid social action, but may generate harsh social reaction, too. These all are becoming more complex by the oriented towards competition.

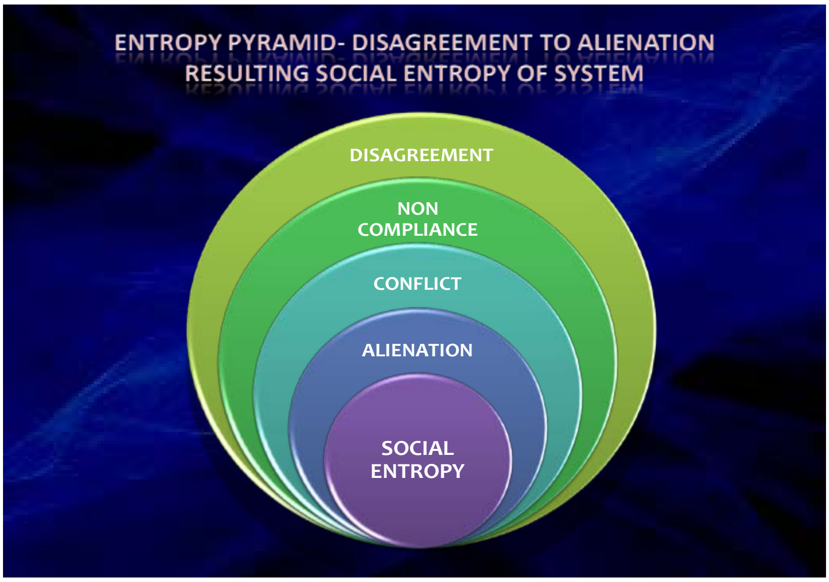
Competition never goes linear or insulated, rather it begets splash of micro-confrontations of aims and interests, a vision and vistas of goes and gateways. That's why it is really scintillating to see that the interaction between right side and left side variables have assumed the character of a 'chi late' function wherein, the predicted character 'social conflict has directed and precisely selected some of the right side factors or ultimately being defined as congenital and interactive disposition of social conflict.

Table 3: **Canonical Variate of Root 4 {Social Entropy (Y5) vs. 10 Independent Variables)**



G. Entropy Pyramid- Disagreement to Alienation resulting Social Entropy of system

Continuous dissonance between *in situ* and ex situ knowledge would lead, as the empirical study evidences, to the inevitable consequence of social entropy. If the entropy sustains to remain for a protractile period, it would generate a deleterious impact on food as well as social security. The sub orbital configuration follows the beta-coefficient in an increasing order.



CONCLUSION :

The entire paper has examined the huge aspects of dissonance and entropy in the flow of knowledge and technology socialization process having impact on social metabolism as well as food security. Agricultural production system is basically a flow and exposition of knowledge, flow-in and flow-out, that can be expressed in different forms of compliances or conflicts. While farming system as a whole is passing through unrest and chaos of knowledge non-compliance, entropy is a must to generate and of course would lead to a neo equilibrium state.

The present study was a concept paper on social entropy, an analogy of principle of Second law of thermodynamics. According to second law of thermodynamics transformation from matter to energy is an irreversible phenomenon therefore it needs to be kept at a manageable level. The gradual modernization in agriculture has produced the jerk, chaos or disorder following the attitudes of the farmers towards discontinuance of the stale technologies and their increasing attitude towards rejection. This has an explicit exhibition of non-compliant behaviour, attitude towards disagreement, conflict and ultimately gets alienated. This has gradually added to, that can be refer to, social entropy.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PROBLEMS AND PROSPECTS OF SOLAR ENERGY IN BANGLADESH: AN EXPLORATORY STUDY ON SYLHET

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ABSTRACT

Energy is the most critical resource for human development. Bangladesh's overall energy situation is quite precarious. At present only around 18% of the population of Bangladesh is connected to the electricity grid, and of the 21 million households in the country, only about 2.8 million have come under the electricity network. To pull the country from the imminent crisis, renewable energy, particularly solar energy could be the prime mover of economic development. Highlighting the importance of solar energy this research study has been undertaken with certain objective views: Finding the Prospects and constraints as well as contributing in the field of solar energy through presenting and highlighting important principles of solar energy. This paper is based on both the primary and secondary data sources. The main secondary data sources used in this study include Journals, Online reports, Newspapers, Reports etc. Major findings in the study include rural people in Banladesh are not aware of the solar energy technology. Appropriate financial arrangement is necessary for the rural people. Solar systems with different options and expert technicians should be available to the consumers. Women also should be invited for training, as they are the main users& attentive maintainer of the systems.

1. INTRODUCTION

Energy is the most imperative resource for human development. Commercial energy is obtained from two broad sources: non-renewable and renewable. Non - renewable commercial energy, the main source of which is natural gas, would expire by 2020. Bangladesh's overall energy situation is quite precarious. Whereas country's energy supply is diminishing, demand is relentlessly increasing. With this imminent power crisis wind and hydropower have a limited scope of success in Bangladesh. At present the supply of grid electricity meets primarily the demand of urban and the industrial sector in the developing countries, whereas, people live in the village areas remain outside the coverage of the electricity supply. Even people with access to electricity do not receive quality service. At present only around 18% of the population of Bangladesh is connected to the electricity grid, and of the 21 million households in the country, only about 2.8 million have come under the electricity network. Highlighting the importance of solar energy this research study has been undertaken with certain objective views: Finding the Prospects and constraints as well as contributing in the field of solar energy through presenting and highlighting important principles of solar energy.

2. METHODOLOGY

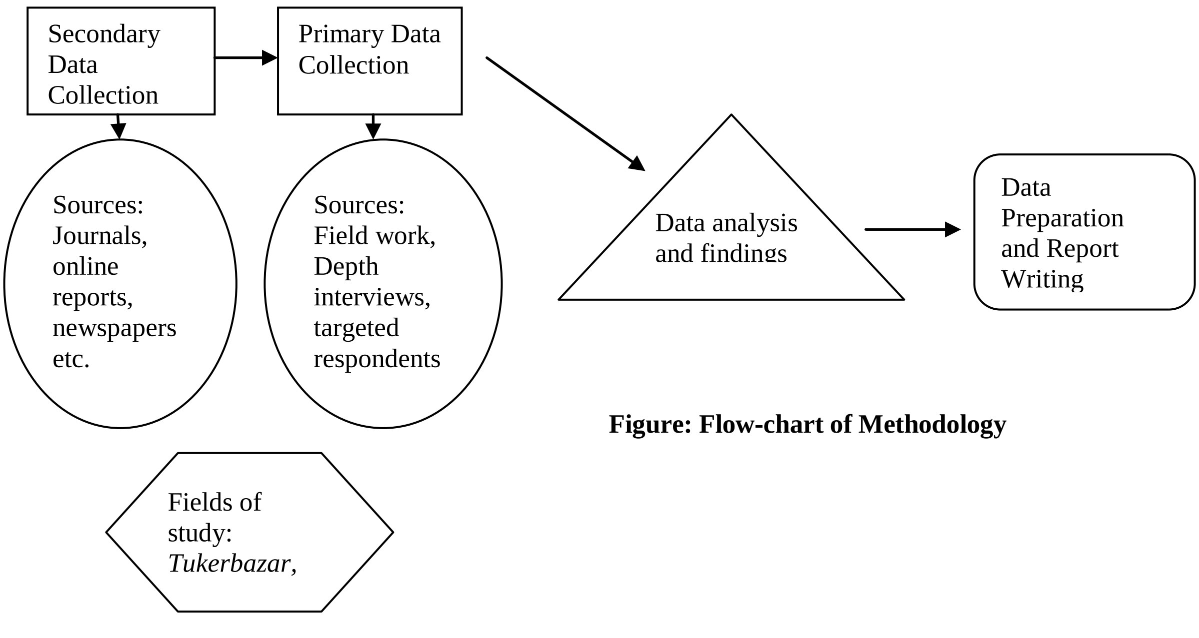


Figure: Flow-chart of Methodology

This paper is based on both the primary and secondary data sources. The main secondary data sources used in this study include journals, online reports, newspapers, reports of the solar energy-based electricity providers etc. Some first-hand or primary data were collected through depth interview with companies' personnel and targeted respondents.

Data have been collected from the regions of Tuker Bazaar and Sunamganj through personal initiatives and used purposive sampling techniques.

2.1 Structure of the Seminar Paper

The seminar paper is both qualitative and quantitative. It is qualitative because there is theoretical analysis. It is quantitative because various quantitative data are presented through table, graph etc.

□ **'Population' of the Seminar Paper**

The entire important areas using solar energy resource sector in Sylhet have taken as 'population'.

□ **Selection of Sampling Design**

The area of seminar is Sylhet and different areas at this area are surveyed. Different areas are selected on the basis of proximity and convenience for understanding current status of solar energy in Sylhet.

□ **Collection of Information**

The information was collected through questioners. (The questionnaire is given in appendix.) Some of the data used in this research are primary data collected from the respective sources i.e. from field work. There are also some data that are collected from different secondary sources i.e. journals, books, publications, and websites. Secondary data means data that are already available i.e. they refer to the data, which have already been collected analyzed by someone else. Secondary data may either be published data and unpublished data. Usually published data are available in: (a) various publications of the central, state, and local governments; (b)Various publications of foreign governments or of international bodies and their subsidiary organizations; (c) technical and trade journals: (d) books, magazines and news papers; (e) reports and publications of various associations connected with business and industry, bank, stock exchanges, etc

- **Reliability of data**:

The reliability our data tested by finding out such things about the said data: (a) who collected the data? (b) What are the sources of data?

- **Suitability of data**:

The data that are suitable for one enquiry not necessary be found suitable in another enquiry. Hence, if the available data are found to be unsuitable, we should not use them. In the context, we must very careful scrutinize the definition of various terms and units of collection used at that time of collecting data from the primary source originally. Similarly, the object, scope and nature of the original enquiry must also be studied. If we find differences in these, the data will remain unsuitable for the present enquiry and should not be used.

- **Adequacy of data**:

If the level of accuracy achieved in data is found inadequate for the purpose of the second enquiry, they will be considered in adequate and should not be used by us. The data will also be considered inadequate, if they are related to one area, which may be either narrow or wider than the area of the present enquiry.

3. ILLUSTRATIONS

3.1 Figures and Graphs

According to Bangladesh Power Development Board(BPDB), its present electricity generation capacity is4,931MWe [13]. In July 2009, average maximum generation and average demand on weekdays were3,909MWe and 4,538MWe respectively giving rise to average load shedding of 629MWe. To tackle this alarming situation and to meet the future demand, the government of Bangladesh is planning to construct some new power plants.

Yearly Solar fluxes & Human Energy Consumption

Solar	3,850,000 Ej
Wind	2,250 EJ
Biomass	3,000 EJ
Primary energy use (2005)	487 EJ
Electricity (2005)	56.7 EJ

Figure 1: Example for figure title

Main aspects of solar energy-based electricity

Photovoltaic (PV) modules use semiconductor materials to generate DC electricity from sunlight. A large area is needed to collect as much sunlight as possible, so the semiconductor is either made into thin, flat, crystalline cells, or deposited as a very thin continuous layer onto a support material. The semiconductor must be sealed into a weatherproof casing, with suitable electrical connectors.

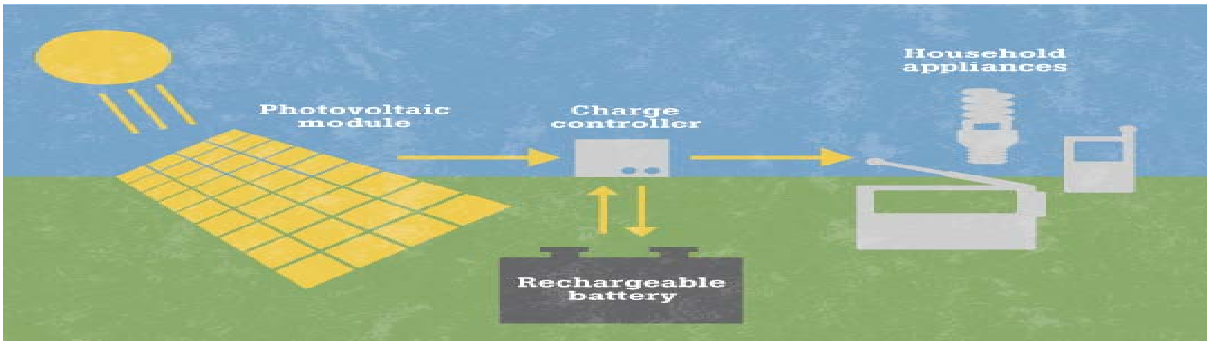


Figure 2: Layout of a solar-home-system

PV modules are specified by their 'watt-peak' (Wp) rating, which is the power generated under standard conditions, equivalent to bright sun in the tropics (they still work at lower light levels though). Most solar-home-systems use modules between about 10 Wp and 100 Wp rating.

The rechargeable batteries store spare electricity on sunny days, so that it is available at night and on cloudy days. They also provide a stable voltage (usually 12 V) for the devices which use the electricity. Standard lead-acid car batteries can be used, but they don't last long if they are heavily discharged, so specially-made solar versions are strongly recommended. Other types of rechargeable battery like nickel-cadmium and nickel-metal-hydride are increasingly used, particularly in small systems like solar lanterns. They are more expensive, but easier to make small and portable, and more tolerant of being heavily discharged. A solar-home-system should be designed with sufficient PV capacity (Wp) to provide the daily electricity demand throughout the year, and typically three days storage capacity in the battery, so that the system keeps working during a cloudy period. For example, a typical solar-home-system sold by Ashden Award-winner SELCO in India has a 35 Wp PV module and a 90 Ah/12 V battery to power four 7 W DC fluorescent lights for about four hours per day and a socket. Recently the Solar Energy Foundation in Ethiopia has introduced a 10 Wp system which powers four small LED lamps.The PV module is fixed to the roof of a home at the angle which collects maximum sunlight. A framed 35 Wp module made from crystalline cells has an area of about 0.3 m2, or about 0.7 m2 if made from lower-efficiency thin film materials, so is easy to handle and install.

The battery is kept indoors, and the terminals should be covered so that they cannot accidentally be touched or short-circuited. The PV, battery, lights and socket are all wired carefully to the charge-controller, ideally by a trained technician.

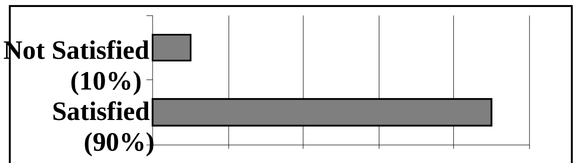
Business benefits of solar energy

Electrification of Rural Markets by Solar Energy:
One of the main economical activities of rural Bangladesh is centered on rural markets called “Haat”. Farmers from long distance come to the “Haat” with their products to sell to merchants, who usually come from cities or major towns. The trading continues till evening. Kerosene lamps called “Kupi” and “Harricane” are the major appliances to meet the lighting needs of the shops of a “Haat”. Some shops use more expensive mantle lamps called “Hazzak” to obtain brighter light.

Feedbacks from the rural people indicate that there is high user satisfaction with having a local and full time technician.The service is good and amount of light is sufficient and most importantly solar energy system is free from hazards of smoke which provides the pollution free energy source.

Sl. No.	Response	Percentage
01	Satisfied	90%
02	Not satisfied	10%

Figure 3: Customer Satisfaction using solar facility



Observations and Lessons Learnt

The major observations and lessons learnt from rural electrification through solar energy include: Rural people in Bangladesh are not aware of the solar energy technology. Therefore demonstration is necessary to reach the information to this group. Appropriate financial arrangement is necessary for the rural people to afford the system. This may include payment in installment, fee for services and other suitable modes. Users training has great impact as the users can do trouble shooting of minor problems like replacing fuse, adding distilled water, replacing bulbs etc. This may avoid technician call and increase system reliability. Solar systems with different options should be available to the consumers so that they can choose themselves according to their demand. Technician training is essential for ensuring the local technical support as well as to make the project sustainable. Women also should be invited for training, as they are the main users of the systems. They can also pay attention for maintenance. For electrification of rural markets through solar energy, local people in management gives better results as this avoids the risk of theft and nonpayment threat. Solar systems do not generate income directly, but it helps in income generating activity such as extends the working hour, creates convenient environment for business etc. Components/accessories of solar systems should be locally available so that the users can buy them easily when required. This can increase acceptability of the technology to the users.

Data analysis

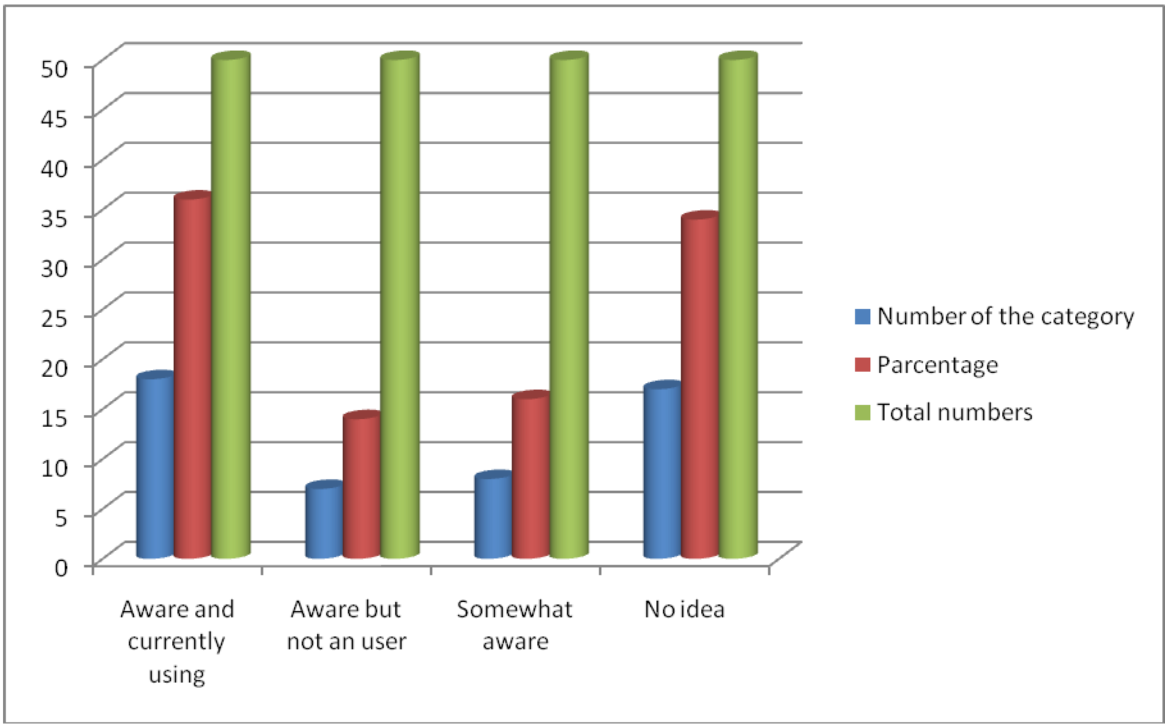


Figure 4: Awareness status of the users

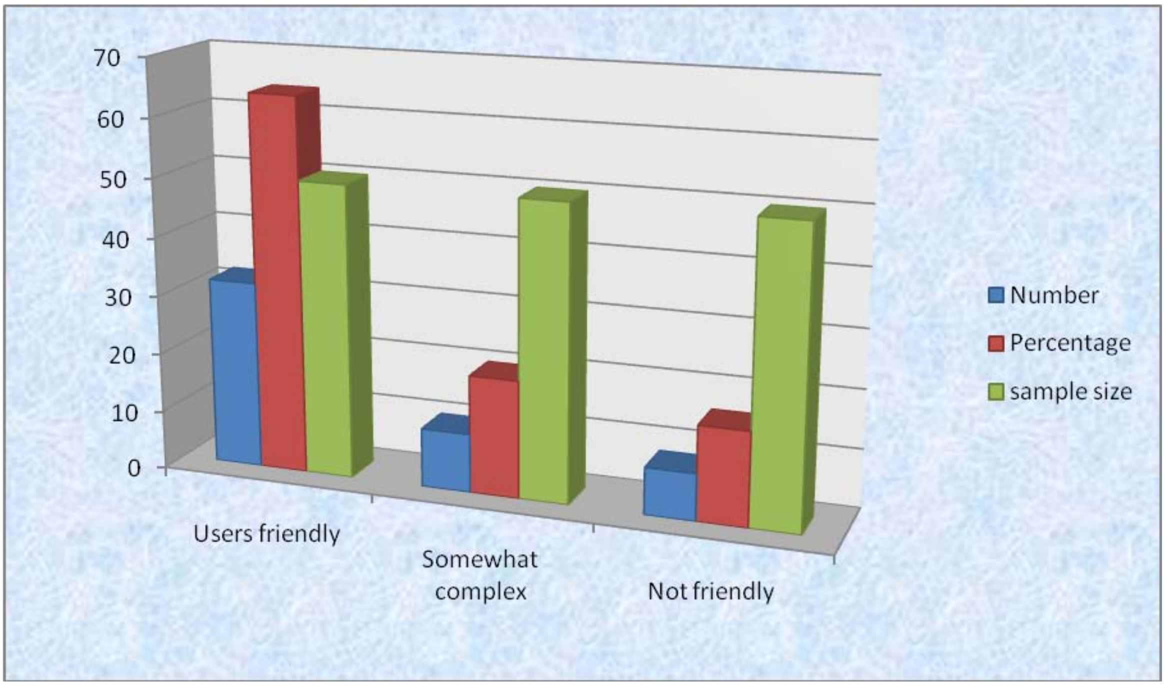


Figure 5: Ease in using solar options

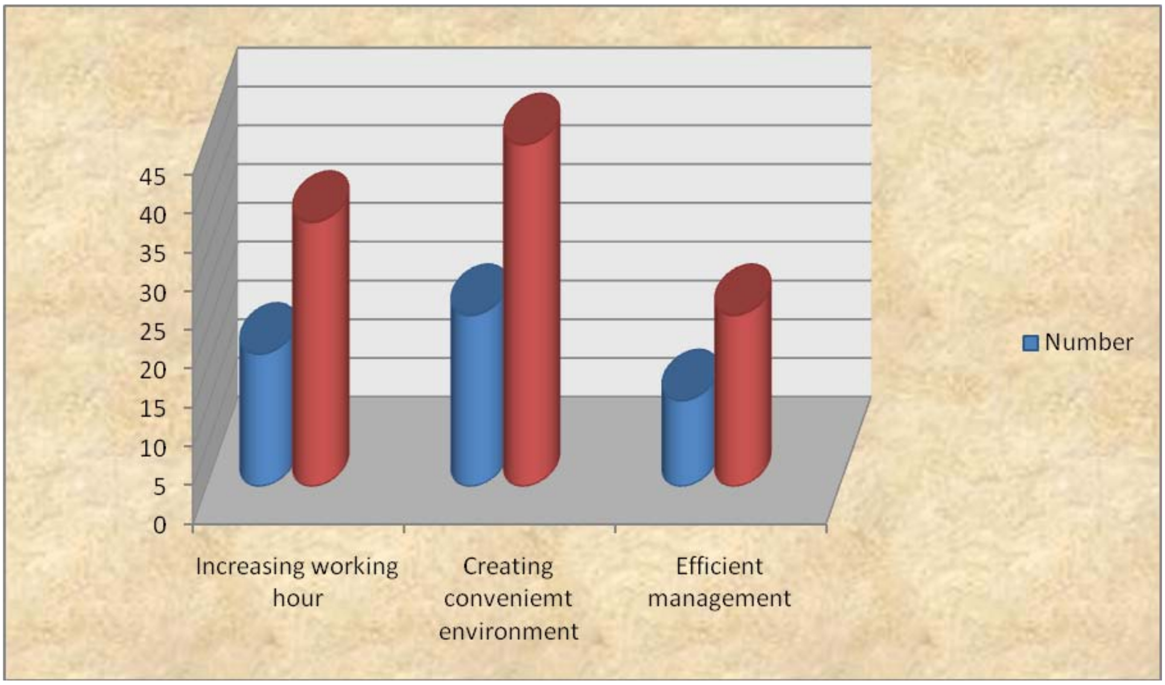


Figure 6: Assumed advantages in using solar energy

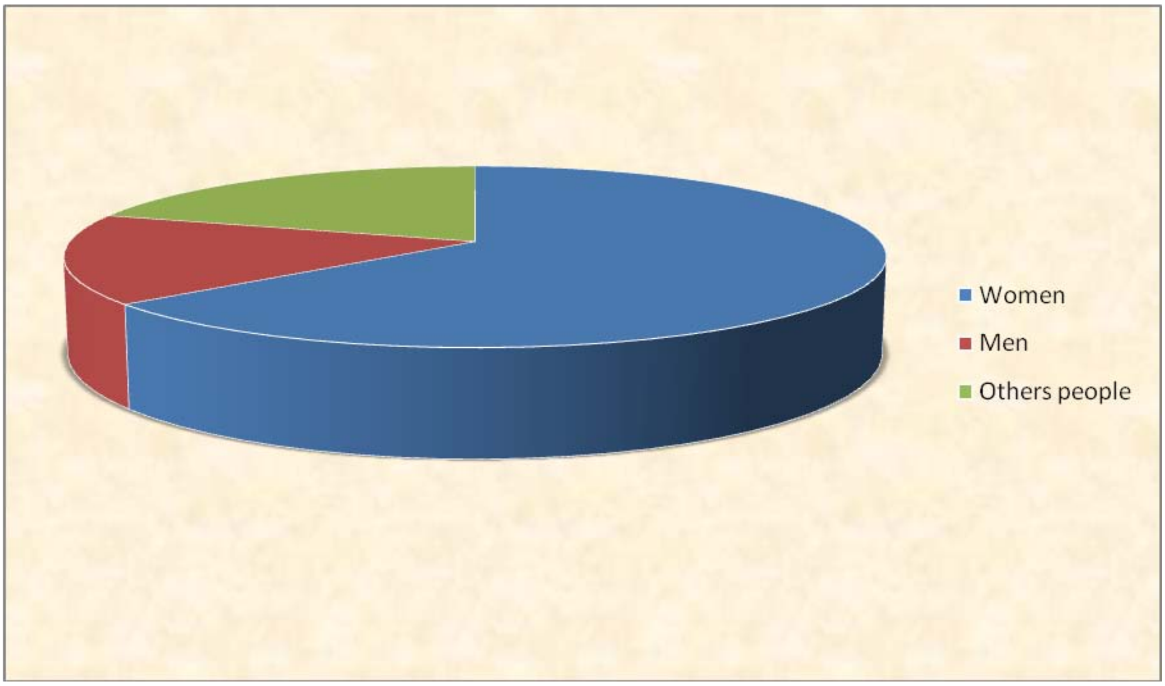


Figure 7: Users of the solar service

4. CONCLUSIONS

Providing electricity for meeting lighting needs of households and rural markets can bring several positive impacts including improvement of quality of life and increasing in income and employment opportunities. So, rural electrification through solar energy is a model of demonstration. The attraction of this model to the users is that they are free from the responsibility of maintaining the system. The risk of the whole system has been avoided with the involvement of local community in management. Demonstration of solar energy system has been successful to create interest among the rural people and demand from other location also observed. This approach could be equally applicable to the other developing countries with similar socio-economical conditions.

Solar Photovoltaic electricity is the cleanest and the most convenient of all forms of energy. Extent of electrification is an index of the quality of life in a country. With all the possible attempts on the part of PDB and REB it will not be possible to reach electricity to all villages, islands, coastal areas, hilly areas and other inaccessible parts of the country because of the scattering of households in the rural areas resulting into low consumer density, transmission and distribution cost from a central grid will be prohibitively high. The only way to overcome this difficulty is the decentralized mode prohibitively high. The only way to overcome this difficulty is the decentralized modern and unique advantage of this new technology. Secondly, it does not need any conventional fuel and as such there is no fuel cost- a great relief in these days of energy crisis. Thirdly, there is no moving part in this system resulting into quiet functioning leading to long durability and soundless environment.

ACKNOWLEDGEMENTS

Actually, research function can only be well done when it is accompanied by appropriate and timely associations from various persons and organizations. In that sense, I am very lucky to get these. I would like to provide some names to whom I am so much humbled such as MD. Abul Hasem, Regional Manager, Grameen Shakti Ltd. Sylhet region, Tahsina Haque Simu Lecturer Department of Business Administration, Shahjalal University of Science & Technology, Sylhet-3114. It is matter of sorrow not ot mention other names for the sake of precision.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ICTHIOFAUNAL BIODIVERSITY: ESTIMATION OF EROSION THROUGH PARTICIPATORY LEARNING ACTION

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ABSTRACT

The present paper has been the outcome of a rigorous research work to identify the factors attributing to the fast reduction of local fish biodiversities. The indigenous fish goners are the most important bio-ecological indicators to measure the intensity of changes disturbing ecological balances of any micro-farming system both in intrinsic and extrinsic manner. Some local and innovative measuring devices, elicited by the stakeholders themselves, were applied to calculate the FMV and fish landing in different local markets. This would ultimately make the researchers draw conclusion that the factors like chemical loads in agriculture, occupational change of the rural people, destruction of local bodies, indiscriminate use of pesticides in agricultural field and adjoining water bodies are all responsible for ushering a faster decline of these local fish species. The decline of species has been measured in terms of loss of decadal score down the years and declining availability of these fishes across the spatial distribution had been statistically intercepted to get the most expected estimation of species count and the factors contributing to them. Other PRA tools had been administered and subsequently been quantified for facilitating the objective analysis and drawing thereby a logical conclusion.

Key words: *Icthiofaunal diversity, climate change, ecology, chemicals, micro-farming system*

1. INTRODUCTION

Fishes exhibit enormous diversity in their morphology, habitats and their biology. Unlike the other commonly recognized vertebrate groups, fishes represent a heterogeneous faunal assemblage. Fishes also constitute almost one-fourth of the total number of vertebrates. An estimated 21,723 living species compared with 21,450 extant tetra pods have been described from the globe (Nelson,1984). It was also noticed that a total of 28,500 fin fishes have so far been recorded from different parts of the globe of which India shares around 2,200 (Anon., 1998 a). All individuals of most species of fish live entirely in fresh or saline waters. A few are diadromous, a part of their life cycle, they spent in fresh water and rest part in the seas. The approximate ecosystem wise distribution of fish genetic resources of India are 73 species (3.32 %) in cold waters, 544 species (24.73 %) in fresh water of plains, 143 species (6.50 %) in brackish water and 1440 specie (65.45 %) in seas around the country (Das,2002). A large number of fish species had once been very common in different water bodies of different ecological conditions all over India a few decades back but due to the destruction of habitats, pollution of water, introduction of exotic species, systematic elimination from culture system, overexploitation, population of a number of fish species have been depleting steadily from different corners of the world. Besides, eco-degradation, owing to varied natural and anthropogenic factors, the water bodies are gradually becoming non-productive (Jhingran, 1988; Pandit et al., 1994). Therefore, the situation warrants for identification of fishes and categorization including prioritization of the need based problems. The fish genetic resources which are at an eloping state, if proper conservation strategies are not worked out based on the baseline information with regard to biology and ecology of fishes will be lost for ever in near future.

2. OBJECTIVES

- To enlist different icthiofaunal components inhabiting different water bodies of the area and to classify them on the basis of their availability and abundance.
- To assess icthyofaunal productivity (production of fish/unit area/unit time).
- To identify and assess different biosocial and techno-managerial factors related to icthiofaunal diversity and icthiofaunal productivity.
- To study biology of one threatened fish so that similar studies may be taken up as a part of conservation strategy of threatened categories.
- To generate avenue for delineating strategic intervention towards providing efficient and sustainable management approaches and techniques in fish production and management in le3ntic water bodies of the area.

3. METHODOLOGY

The deliberation on the methodology has been made to understand the concepts, methods and techniques used to design the study, by collecting the icthiofaunal information, analyzing data and interpreting the findings. Fish species are collected from different selected locations of rivers, lakes, ponds, canals, streams, paddy field etc. through random nettings with the help of drag-nets, caste-nets, gill-nets, different types of local traps, barriers etc. as well as from the fisherman community and also from different local markets during different seasons. Immediately after collection, the specimens were preserved in 5 % buffered formalin solution. Indigenous naming of different fishes was determined through focus group discussion (Mukherjee, 1995). The specimens were subsequently identified following standard references (Day, 1978; Jayaram,1981; Menon,1987; Talwar and Kacker,1984;Talwar and Jhingran,1991).

The experiment was conducted in aquariums, three ponds each with an area of 2.5 decimal, 10 decimal and 33 decimal, respectively situated at East Singh, Jharkhand, India. Live specimens of *Puntius sarana* were brought to the laboratory from different collection sites viz. aquarium, earthen tank and open water bodies like Dimna Lake and Subarnarekha River for further studies. The colour, external appearance of mouth, nostril, burbles, operculum, trunk, unpaired and paired fins, scales, anus, tail, skin and lateral line were recorded through visual observations and with the help of vernier scale. The fecundity of *P. sarana* was estimated by studying 25 individuals after being collected from well managed culture ponds which were caught by caste nets and at the same time matured females of *P. sarana* were also collected from large open water bodies by both caste nets and gill nets. Participatory Learning Action (PLA) is a methodology, which helps in interacting with local communities, understanding them and learning from them. It helps in the process of involvement with local communities for indigenous knowledge-building exercise. General analysis of specific topic, question or problem; needs assessment; Feasibility studies; identifying priorities for development activities; implementing development activities where new information needs to be collected; and monitoring and evaluation of development activities.

A pilot survey was conducted in the selected villages before constructing and finalizing the data collecting devices. In course of this survey, informal discussion was carried out with some fishermen, local leaders and extension agents of the localities. An outline of the socio-economic background of the fishermen of the concerned villages, their opinion towards pisci-culture and prevailing problem concerned with the pisci-culture were enumerated to construct the structured schedule for the study.

A. Dependable Variables.

Fish productivity (Y)

B. <u>Independent Variables</u>

Age of pond holder (X_1),Educational qualification (X_2),Training taken (X_3),Family members (X_4),Gross monthly income (X_5), Ownership of pond (X_6),Water area (X_7),Depth of pond (X_8),Water retention period (X_9) ,Distance of pond from residence (X_{10}),Transparency of pond water (X_{11}),pH of pond water (X_{12}),Plankton present in the date of survey (X_{13}),Free CO_2 (X_{14}),Gross alkalinity (X_{16}),Stocking density (X_{17}),Gross cost of inputs (X_{18}),Size of fish fry (X_{19}) and Gross investment cost (X_{20})

4. RESULTS AND DISCUSSION

4.1 Trend Analysis (Decade Wise Fish Availability)

Participatory trend analysis on decadal distribution starting from 1960 to 2000 depicts the availability of seventy five fish species in terms of percentage of population depletion in East Singhbhum District (Table - 1). To analyze the nature of increase and decline of the populace of target fishes over decades taking reference from the age-old and experience profiles of 150 local fishermen.

The graphical presentations can be conceived as the results of the PLA exercises. The bar diagrams (fig. 1) depicted the quantification of different decline of a species over different decades. This has provided an inkstand vision of the decline trend and character of a given species over different decades.

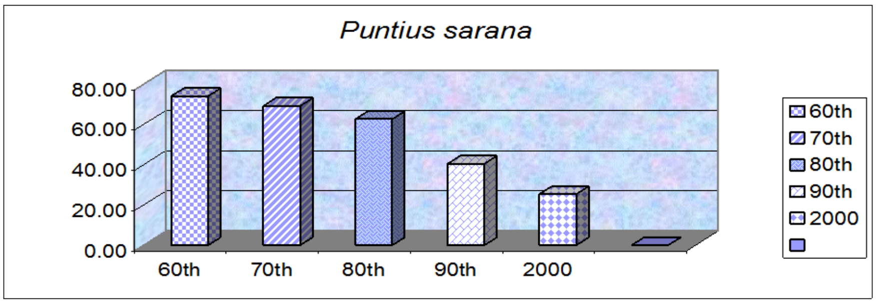

Figure 1: illustrate the distribution and decline of an economically and ecologically viable indigenous cyprinid fish ***Puntius sarana*** over different decades as percept and assessed by the fishermen from their profile of experiences and collective wisdom.

4.2 Multiple regression analysis: Fish production and twenty other causal variables.

It was envisaged from the multiple regression analysis (table 4) that the variables plankton quantity (X_{13}), stocking density (X_{17}), cost of inputs (X_{18}), size of fry (X_{19}) and the gross investment cost (X_{20}) had played the most significant roles in fish productivity. It was also discernible from the table that, a unit change in plankton density had been resulted into 0.375 unit change in the productivity. The higher abundance of plankton in a water body had a decisive effect on the feeding of fishes and their subsequent growth which ultimately increased the body weight vis-à-vis productivity.

Cost of inputs (X_{18}) had recorded a strong regression effect on the fish productivity. This would mean that with the increase in allocation of expenditure on critical inputs, the output per unit area had gone high up. As usual it would have been, the same causal relation had also been reflected in between gross investment cost and increase in productivity. The R^2 value being 0.5227 it can be inferred that all the variables (twenty) jointly had explained 52.27 per-cent of the variability in the consequent factor i.e. productivity of fish.

The variable size of fry recorded the significant regress ional effect on the fish productivity. In case we ignore the cost of fish fry, the size of the fry had a strong effect on the productivity of per unit area. Higher size of fish fry means the higher rate of survivability, which ultimately would breed on the productivity of fish. Of course, optimization of the economic size of fry itself should be a serious consideration while cost benefit ratio, input output ratio or efficiency factor is taken care of.

This would suggest that inclusion of more number of logical and contextual causal variables in the study could have explained higher amount of variability embedded with the consequent factor of productivity of fish.

4.3 Path analysis: Direct, Indirect and Residual effect of exogenous variables on the consequence of variables of fish productivity.

From the path analysis (table 3) it was found that the variable plankton quantity had exercised the highest direct effect on the fish productivity. This finding went had a gross compliance with the earlier findings as in found in the co-efficient of co-relation study. So, any fish manager now had to concentrate more on generations of plankton in a given pond area to adopt befitting management practices. In the hierarchy of importance, variable cost had occupied the second position. Variable cost implied expenditure incurred after yeomen inputs, having an immediate effect on productivity. The result rightly implied that the more the investment of the pisci-culture management the higher had been the productivity. The third variable in the tally of importance had been the stocking density (X_{17}). In the rearing stage, the density of fish populations per unit area had a decisive effect on the fish productivity. The level of stocking density had a stress on the food and had a stress on the availability of plankton, oxygen, and other materials to thrive in. Again, a judicious stocking density would help the most clandestine exploitation of the ichthyofaunal ecosystem to attain the desire productivity. In the fourth position, the variable was size of fish fry, which had also exerted the substantial direct effect on the fish productivity. The size of the fish fry helped to make a good take off in the realm of fishery management and ultimately to contribute to the level of fish productivity. It was also to be kept in mind that, in the arena of multivariate analysis, no single factor in a linear way can be helped unilaterally, and so also was the size of the fish fry. That was why only the size of a fish fry could not be, in a linear way, connected with the production.

From the tariff of the substantial indirect effect, it was found that through the variable plankton quantity (X_{13}) the highest indirect effect of nine variables had been channeled. Thus, in the web of interaction, the role of plankton quantity (X_{13}) in impacting the fish productivity had become unquestionable. So, intervention is to be made to increase the plankton quantity as to make the other factors adequately operable in increasing the fish productivity per unit area. The residual effect had been 0.4773 and it was to infer that 47.73 per cent of the variations in the consequent variable i.e. fish productivity per unit area, could not be explained by the constellation of twenty antecedent variables only. Thus, it would further be suggestive that inclusion of more number of pertinent and contextual antecedent variable would have provided higher efficiency for explaining more variations embedded in the consequent variable i.e. productivity per unit area.

4.4 RBQ: The Rank Based Quotient

Six groups meeting with 150 local fishermen, having at least 40 years of experience in fishing, were organized to generate information on factors responsible for the problems related to ‘why fish population had gone a sharp decline over the decades’. These problems were identified through participatory interactions and also had undergone categorizations, sorting and screenings. The number of participants in these interactions was 25 for each focus group. Four such groups were undergone for necessary interactions to generate adequate data. This type of participatory tool (RBQ) has already been successfully applied to assess and document the nature and degree of decline of a score of selected species over decades (1960 – 2000) (Patra et. al. 2005).
Following Rank Based Quotient (RBQ), it was found that (1) use of pesticide in different fields occupied the first rank followed by destruction of brood fishes during monsoon (2), lack of water retention capacity (3), different diseases resulted out of fish production (4), sharing problems of water bodies (5), destruction of fish spawn and fry (6), lack of financial support (7), lack of knowledge (8), destruction of breeding ground (9) non-availability of fish spawn (10), fishes grown in stagnant water due to sloppy land (11), lack of machinery (12), rapid urbanization and population explosion (13), high cost of pisci-cultural equipments (14) and problems of natural calamities (15). During last few decades, mass mortality of fishes has been observed in India and elsewhere as a result of disease associated with eco-degradation (Tarzwell and Henderson 1957; Saunders 1969; Konar 1975).

4.5 STRATEGICAL IMPLICATIONS

Conventional researches on any biotic resources have so far ignored the aspect of social need like resource and livelihood, managing resources and peoples participation, productivity and social ecology etc. Mere counting of abundance or rearity of any species could not derive any clear-cut finding on how to make proper interventions in properly dealing with the issue. The study had rightly placed laboratory-based observations in the core of people’s wisdom and experiences over time to be matched verified refined and tested. Success was achieved to observed that the factors as identified and quantified through PLA techniques (**Resource diagramming, Vector scoring, Time line, Matrix ranking, RBQ value, Causal diagram and Trend analysis**) had got a strong scientific as well as empirical evidences generated through conventional research process. In attaining sustainability as an impact of a pro-people research, technology generated there form must be need based, eco-friendly, resource-friendly, non depletive etc. Hence the research in an innovative way had derived certain social and economical dimension implanted organically with a conventional research approach.

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Table- 1 : Decade wise trend analysis of fishes of East Singhbhum, India.

Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
1	*Catla catla* (Hamilton-Buchanan)	Catla	72.80	70.93	2.57	67.87	4.33	67.33	0.78	64.33	4.56
2	*Cirrhinus mrigala* (Hamilton-Buchanan)	Mirik,Mrigal	73.47	70.27	4.36	68.13	3.03	65.47	3.92	62.90	3.93
3	*Cirrhinus reba* (Hamilton-Buchanan)	Saruansh,Nuni	66.53	62.93	5.41	59.33	5.72	47.87	19.33	35.87	25.07
4	*Labeo rohita* (Hamilton-Buchanan)	Rahu,Rui	74.80	72.27	2.67	69.87	3.32	66.53	4.77	66.53	1.00
5	*Labeo calbasu* (Hamilton-Buchanan)	Calbagus,Calbosh	55.07	49.87	9.44	42.80	14.18	34.93	18.39	22.13	36.64
6	*Labeo bata* (Hamilton-Buchanan)	Bata	62.56	56.48	9.71	44.96	20.39	38.88	13.52	21.76	44.03
7	*Labeo pangusia* (Hamilton-Buchanan)	Birbal	58.20	50.80	12.71	40.00	21.26	24.40	39.00	13.45	44.88
8	*Labeo angra* (Hamilton-Buchanan)	Perih	63.20	53.00	16.14	49.20	7.17	35.00	28.86	18.25	47.86
9	*Puntius chola* (Hamilton-Buchanan)	Puthi	DATA DEFICIENT								
Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
10	*Puntius gelius* (Hamilton-Buchanan)	Tit puthi	60.68	52.43	13.59	47.65	9.12	34.80	26.97	27.33	21.47
11	*Puntius sarana sarana* (Hamilton-Buchanan)	Potha,Sar puthi	74.00	69.07	6.66	62.53	9.45	40.40	35.39	25.33	37.30
12	*Puntius japanicus* (Hamilton-Buchanan)	Japani puthi	0.00	0.00	00.00	0.00	0.00	12.48		26.67	-110.51
13	*Puntius sophore* (Hamilton-Buchanan)	Puthi	74.53	72.27	3.05	68.00	5.91	64.67	4.89	62.93	2.69
14	*Puntius ticto* (Hamilton-Buchanan)	Dingla puthi	61.07	55.60	8.96	50.80	8.63	41.47	18.37	29.87	27.97
15	*Puntius puntio* (Hamilton-Buchanan)	Kankoi	DATA DEFICIENT								
16	*Chela faciata* (Silas)	Chela	66.73	60.27	9.69	54.93	8.84	44.80	18.44	28.80	35.71
17	*Salmostoma bacaila* (Hamilton-Buchanan)	Banspata	73.25	69.87	4.61	61.20	12.41	42.53	30.51	24.43	42.56
18	*Rasbora daniconius* (Hamilton-Buchanan)	Ramshal	DATA DEFICIENT								
19	*Hypopthalmichthys molitrix* (Valenciennes)	Silvercarp	0.00	0.00	0.00	0.00	0.00	13.33		32.00	-140.06
20	*Ctenopharyngodon idella* (Valenciennes)	Grasscarp	0.00	0.00	0.00	0.00	0.00	11.73		27.87	-137.59
21	*Cyprinus carpio* (Hamilton-Buchanan)	Commoncarp, American	0.00	0.00	0.00	0.00	0.00	22.80		36.40	-59.65
22	*Amblypharyngodon mola* (Hamilton-Buchanan)	Mourla, Morula	75.33	70.93	5.84	65.87	7.15	51.60	21.66	35.73	30.76
23	*Rohtee cotio* (Dey)	Chandkura	63.20	54.00	14.56	50.80	5.93	43.73	13.92	29.87	31.72
Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
24	*Danio dangila* (Hamilton-Buchanan)	Darnke	66.00	59.47	9.89	52.40	11.89	41.47	20.86	28.00	32.48
25	*Chela labuca* (Hamilton-Buchanan)	Darnka	60.16	55.68	7.45	44.00	20.89	29.28	33.45	19.04	34.97
26	*Omobranchus zebra* (Bleeker)	Luli	75.00	74.53	0.62	70.80	5.00	66.27	6.39	59.47	10.26
27	*Esomus danricus* (Hamilton-Buchanan)	Darnka , Darnke	73.60	73.20	0.54	70.63	3.51	67.33	4.67	60.80	9.69
28	*Garra annandalei* (Hora)	Garra	DATA DEFICIENT								
29	*Lepidocephalicthys guntea* (Hamilton-Buchanan)	Genti	71.47	67.73	5.22	63.87	5.71	58.27	8.77	48.93	16.01
30	*Botia lohachata* (Chaudhuri)	Pathar chata	68.53	64.53	5.84	56.67	12.19	47.87	15.53	32.40	32.32
31	*Hyporhamphus limbatus* (Valenciennes)	Gangdhara (Small)	DATA DEFICIENT								
32	*Xenentodon cancila* (Hamilton-Buchanan)	Gangdhara (Big)	47.95	37.60	21.58	25.20	32.98	11.20	55.56	0.00	100.00

33	*Monopteras* (Amplinpinim)cuchia	Cuchia	57.87	51.73	10.59	44.67	13.67	31.47	29.55	16.03	49.03
34	*Monopterus albus* (Zuiew)	Bhunia	58.27	47.55	18.39	34.70	27.02	23.47	32.36	1.33	94.29
35	*Mystus gulio* (Hamilton-Buchanan)	Tangra	62.93	54.80	12.92	46.27	15.57	27.87	39.77	11.83	57.51
36	*Mystus cavasius* (Hamilton-Buchanan)	Tangra	64.40	56.67	12.00	48.00	15.29	33.87	29.44	14.73	56.48
Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
37	*Mystus tangra* (Hamilton-Buchanan)	Tangra	68.00	58.20	14.41	47.20	18.90	29.60	37.29	11.75	60.30
38	*Mystus vittatus* (Bloch)	Tangra	67.60	56.80	15.98	45.80	19.37	25.80	43.67	8.20	68.22
39	*Aorichthys seenghala* (Sykes)	Arh	65.47	61.20	6.52	53.47	12.63	42.27	20.95	22.40	47.01
40	*Ompok pabda* (Hamilton-Buchanan)	Pabda	62.80	56.00	10.83	47.60	22.55	32.13	32.50	11.73	63.49
41	*Ompok bimaculatus* (Bloch)	Pabda	64.53	58.80	8.88	54.00	8.16	39.33	27.17	20.00	49.15
42	*Wallago attu* (Schneider)	Boal, boar	64.53	59.23	8.21	49.33	16.71	35.87	27.31	16.53	53.89
43	*Pangasius pangasius* (Hamilton-Buchanan)	Pangas	59.07	49.87	15.57	39.73	20.33	24.00	39.59	0.00	100.00
44	*Clarias batrachus* (Linnaeus)	Magur	68.67	62.67	8.74	57.87	7.67	46.40	19.82	36.00	22.41
45	*Heteropneustes fossilis* (Bloch)	Singhi	62.80	57.47	8.49	51.47	10.44	34.40	33.16	13.73	60.08
46	*Clarias gariepinus* (Hamilton-Buchanan)	Hybrid magur	0.00	0.00		0.00		16.28		30.80	-89.18
47	*Lates calcerifer* (Bloch)	Bhetki	45.20	36.40	19.47	22.40	38.46	9.75	56.47	0.00	100.00
48	*Chanda nama* (Hamilton-Buchanan)	Chanda	69.20	65.60	5.20	57.60	12.19	52.00	9.72	44.40	14.62
49	*Pseudambassis ranga* (Hamilton-Buchanan)	Pitpiti, Chanda	68.00	65.07	4.31	58.67	9.84	55.33	5.68	46.67	15.67
50	*Pseudambassis lala* (Hamilton-Buchanan)	Do	68.67	64.33	6.31	59.07	8.19	54.80	7.23	47.07	14.12
51	*Pseudambassis notetus* (Blyth)	Do	68.13	64.67	5.09	59.73	7.62	54.13	9.38	46.13	14.78
Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
52	*Nandus nandus* (Hamilton-Buchanan)	Nedhas, Nedha	44.80	41.47	7.43	32.13	22.49	13.73	57.27	0.00	100.00
53	*Badis badis* (Hamilton-Buchanan)	Koi	DATA DEFICIENT								
54	*Oreochromis mojambica* (Hamilton-Buchanan)	Tilapia	28.00	38.00	35.71	34.40	9.47	51.07	48.46	61.73	-20.89
55	*Oreochromis niloticus* (Hamilton-Buchanan)	Nilotika	0.00	0.00		0.00		17.00		38.60	-127.05
56	*Liza parsia* (Hamilton-Buchanan)	Perse	50.93	46.40	8.89	33.87	27.00	18.40	45.67	0.00	100.00
57	*Rhinomugil corsula* (Hamilton-Buchanan)	Cactus, Tarai	51.07	47.87	6.27	33.73	29.52	18.40	45.45	0.00	100.00
58	*Glossogobius giuris* (Hamilton-Buchanan)	Bele, Bhatu, Bhelsa	68.67	66.00	3.89	62.67	5.05	50.53	19.36	31.87	36.95
59	*Anabas testudineus* (Bloch)	Koi	56.93	52.00	8.66	42.80	17.69	35.87	16.19	17.87	50.18
60	*Colisa feciatus* (Schneider)	Kholse	64.45	61.60	4.42	54.67	11.25	48.53	11.21	38.67	20.34
61	*Aplochielus dayi* (Hamilton-Buchanan)	Techokha	DATA DEFICIENT								
62	*Aplocheilus panchax* (Hamilton-Buchanan)	Do	58.00	54.93	5.29	46.67	15.06	38.93	16.56	25.47	34.60
63	*Channa gachua* (McClelland)	Chang	74.67	72.40	3.04	69.20	4.42	63.73	7.91	56.93	10.67
64	*Channa marulius* (Hamilton-Buchanan)	Shal	48.00	40.96	14.67	26.88	34.38	12.20	54.61	0.00	100.00
65	*Channa striatus* (Bloch)	Shole	64.67	57.73	10.72	51.73	10.39	40.80	21.13	18.93	53.60
Sl No	**Scientific Name**	**Local Name**	**1960**	***1970***	**+**	**1980**	**+**	**1990**	**+**	**2000**	**#**
66	*Channa punctatus* (Bloch)	Gorai, Latha	73.07	71.33	2.37	66.40	6.91	61.47	7.42	54.53	11.27
67	*Macrognathus pancalus* (Hamilton-Buchanan)	Khet truh	72.67	68.27	5.56	64.53	5.46	58.00	10.12	43.33	25.29
68	*Rhynchobdella aculeata* (Hamilton-Buchanan)	Turh	72.80	67.60	7.14	61.47	9.07	55.87	9.11	46.67	16.47

69	*Mastacymbalus armatus* (Lacepede)	Ban	68.40	64.27	6.04	59.47	7.47	50.80	14.58	37.33	26.52
70	*Amblyceps mangois* (Hamilton-Buchanan)	Chaldhua	71.73	61.92	13.68	56.60	8.59	33.33	41.11	15.30	54.09
71	*Notopterus notopterus* (Pallas)	Pholad, Pholui	66.13	62.13	6.05	55.07	11.38	48.53	11.86	35.87	26.11
72	*Notopterus Chitala* (Hamilton-Buchanan)	Chital	49.52	44.32	10.5	28.80	35.02	12.80	55.56	0.00	100.00
73	*Gadusia chapra* (Hamilton-Buchanan)	Gaducia	DATA DEFICIENT								
74	*Hilsa ilisha* (Hamilton-Buchanan)	Ilish	56.27	50.40	10.43	36.93	26.73	28.13	23.83	15.87	43.62
75	*Chanos chanos* (Forsskal)	Chanos	DATA DEFICIENT								

LEGEND

- = % of decline over previous year. # = mean decline

Table- 2 : Identification of causal factors responsible for extinction of fish species

Sl.No.	General Pisci-cultural Problems	Ranking by rural fishermen										Rank base Quation Value %	Rank
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**(RBQ %)**	
1	Lack of Knowledge	3	2	2	2		1					33.2	
2	Lack of financial support	8	2	1			1					44.4	
3	Lack of water retention capacity	13	4	1	2							75.2	
4	Use of pesticide in different fields				8		2	1	2	1		42.2	
5	Non-availability of fish spawn			2	3	1	4		1			26.4	
6	Lack of machinaries							1	2		1	4	
7	Different diseases resulted down fish production						1	2	1	3		8.8	
8	In stagnant of water due to sloppy land								1	1		2	
9	Destruction of brood fish during monsoon		4	3	1	2	1					32.4	
10	Destruction of breeding ground			2	1	3	2					20.4	
11	Destruction of fish spawn & fry						1	3	2			4	
12	Sharing problems of water bodies	14	3	2		1						75.6	
13	Problems of natural calamities					1	2	2			1	10	
14	High cost of pisicultural equipment							1	2	2	1	5.6	
15	Rapid urbanization and population exploration									1	1	1.2	

Table – 3 : Path analysis to delineate that direct and indirect effect of exogenous variables on the consequent of variable fish productivity.

Sl No	Variables	Gross Effect(r)	Direct Effect(β)	Indirect Effect(r-β)	Substantial Indirect Effect		
					I	II	III
1.	Age (X_1)	0.0273	0.0240	0.0033	X_{13} -0.0334	X_{17} 0.0292	X_{18} 0.0246
2.	Educational qualification (X_2)	0.0921	0.0490	0.0431	X_{19} 0.0675	X_{20} -0.0537	X_{13} 0.0292
3.	Training experience (X_3)	0.0263	-0.0850	0.1113	X_{18} 0.0706	X_{13} 0.0494	X_{20} -0.0409
4.	Family size (X_4)	0.1032	0.0780	0.0252	X_{13} 0.0333	X_{17} -0.0195	X_{18} -0.0189
5.	Annual income (X_5)	-0.0393	0.0050	-0.0443	X_{20} -0.0645	X_{19} 0.0458	X_{10} -0.0289
6.	Ownership of pond (X_6)	0.0442	0.1010	-0.0568	X_{19} -0.0553	X_{20} 0.0475	X_{13} -0.0382
7.	Water area (X_7)	-0.21167	-0.0430	-0.1737	X_{20} -0.0743	X_{13} -0.0556	X_{17} -0.0350
8.	Depth of pond (X_8)	0.0837	0.0220	0.0617	X_{19} 0.0333	X_{13} 0.0242	X_{20} -0.0204
9.	Water retention period (X_9)	0.1491	0.0560	0.0931	X_{19} 0.1061	X_{20} -0.0684	X_{13} 0.0354
10.	Distance of pond from residence (X_{10})	-0.1589	-0.0740	-0.0849	X_{13} -0.0519	X_{19} 0.0258	X_{20} -0.0508
11.	Transparency of pond water (X_{11})	-0.1201	-0.0610	-0.0591	X_{13} -0.0732	X_{17} -0.0409	X_{19} 0.0346

Table – 4 : Multiple regression analysis between fish production and other 5 causal variables.

Sl. No.	Variables (X)	β	β x R	Regression coefficient (β)	SE(β)	T-value of (β)
1	Plankton (X_{13})	0.416	48.444	75.050	9.96	7.531

.					5	
2.	Stocking density (X_{17})	0.288	23.618	7.858	1.541	5.098
3.	Gross cost of inputs (X_{18})	0.183	15.297	0.114	0.039	2.944
4.	Size of catch fish (X_{19})	0.212	14.082	0.045	0.014	3.203
5.	Gross invest (X_{20})	-0.264	-1.441	-0.024	0.006	4.142

Multiple R^2 value = 0.4809; F value for R = 35.95 with 5 and 194 DF

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DEVELOPMENT OF GIS BASED DECISION SUPPORT DATABASE FOR IMPROVED HEALTHCARE WASTE MANAGEMENT IN KHULNA CITY

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ABSTRACT

Healthcare establishments (HCEs) generate a huge quantity of both the hazardous and non-hazardous wastes. These wastes are generated as a result of diagnosis, treatment and research on human and animal diseases. The hazardous wastes pose serious occupational health risks to those who generate, handle, package, store, transport, treat, and dispose them. There are no proper healthcare waste-management facilities in the government sector in Khulna city or even in the country. There is also lack of available budget to implement the schemes of collection, treatment and disposal of hospital wastes separately. However, a non government organizations (NGO) named Prodipon, Bangladesh, has been handling the heathcare wastes management in Khulna city. But yet, they are much far from the achievement of the aimed goals and objectives of proper handling, treatment and disposal of hazardous hospital wastes. From this study the mean total healthcare waste generation rate in Khulna city has been found to be approximately 0.676 kg/bed/day and separately 0.06, 0.12, 0.003 and 0.50 kg per bed per day for Infectious, Plastic, Sharp and Liquid waste respectively. There are a number of treatment and disposal methods like incineration and chemical treatment. But no method is capable of eliminating all risks completely to human health and environment. Although it has been thought that incineration is the best technological option for the treatment of healthcare waste, awareness is growing against this technology due to its intense adverse effect on the air quality. So management of health care waste is not an easy job in a developing country like Bangladesh without the usages proper technologies and allocation of enough money. Moreover, Geographic Information System (GIS) had been used to assess waste generation, generation variation, collection system and selection of transportation routes and consequently successful management of healthcare wastes in Khulna city can be ensured by the author.

Keywords: *Healthcare, waste generation, GIS, decision support, database, waste management, Khulna city.*

1. INTRODUCTION

The rapid increase of hospitals, clinics, healthcare centre and diagnostic laboratories etc in Khulna city exerts a tremendous impact on human health ecology. More than 150 hospitals, clinics, healthcare centre and diagnostic laboratories exist in the Khulna city (KCC). These facilities generate an estimated 3.6 tons of waste a day. Only a few have the necessary means to dispose the waste safely. It is reported that even body parts are dumped on the streets by these HealthCare Establishments (HCEs) (Rahman, 2006).

The prevalence of diseases that may be transmitted by hospital wastes is alarming in Bangladesh. There is evidence of hepatitis B infection among 10 percent of children (5-10 age group) and 30 percent adults. About 5 per cent of the total population in Bangladesh is thought to suffer from chronic hepatitis B infection. Although cases of HIV/AIDS are low in Bangladesh (about 13,000 cases estimated in 2001) in comparison to neighboring countries, nevertheless the numbers are rising. It is noted here that much of the clinical wastes (e.g. syringes, needles, saline drips, discarded food, gauze, vials, and ampoules) are collected by women and children who re-sell it despite of the deadly health risks (Roteb, 1998).

It is estimated that hospital wastes account for a very small fraction, notably, only about 1 percent of the total solid wastes generated in Bangladesh. In a report from the World Bank (2003), only 10-25 percent of the hospital wastes are infectious or hazardous. The amount of such hazardous waste is quite small in figure and until recently this is not handled properly (WHO, 2001). Mixing with the domestic solid wastes, the total waste steam becomes potentially hazardous (Rahman, 1999).

Various types of clinical wastes such as food & vegetable wastes, medicine strips, used gauge, cotton, tissue, organs, syringe, saline bags, blades and needles etc. are collected from 44 hospitals/clinics/pathology laboratories of the city. Prodipan provides plastic container of four colors (shown in Figure 1) in the clinics for the storage of wastes. The selected colors are (i) Black for kitchen wastes, (ii) Gray for general hospital wastes, (iii) Yellow for non-sharp or reusable wastes and (iv) Red for sharp wastes. The wastes are separated by the users at the source in different categories such as needle & sharp parts, reusable wastes, surgical wastes and food & vegetable wastes by depositing them separately in four bins marked in different colors. Food & vegetable wastes are ultimately disposed with other MSW in the same site of KCC at Rajbandha. Re-useable wastes are shredded and stored for recycling after cleaning, needle & sharp wastes are encapsulated in a 7 ft. deep concrete chamber. Remaining general medical wastes are burned in a locally made burning unit and the residues are then disposed ultimately in a nearby pit (Alamgir et al. 2003).

The present practice of improper handling of generated hospital wastes in Khulna city is playing a contributing role in spreading out the Hepatitis and HIV diseases. The liquid and solid wastes containing hazardous materials are simply dumped into the nearest drain or garbage heap respectively where they are prone to contaminate the rag-pickers that sift through the garbage dumps. The chances of infection are very high to the cleaners, concerned people in the HCEs and to the general population. The improvement of waste management for the HCEs in Khulna city will have significant long-term impact on keeping the spread of infectious diseases to a minimum and result in a cleaner and healthy environment.

Like other industries and institutes, healthcare facilities generate various kinds of wastes as a result of a variety of medical treatment and research. In the past 10 years, due to the increased number and size of healthcare facilities, medical services, and use of medical disposable products, the generation rate of healthcare wastes has increased rapidly. And so the problem is requiring an urgent attention toward achieving the objectives of safe collection, separation, transportation, treatment and disposal of hazardous hospital wastes (Kazi, 2000).

However, this study has been carried out with the following objectives:

- To assess the present state of healthcare waste management (HCWM) system as a whole in Khulna city as per World Health Organization (WHO) guidelines.
- To plot the HCEs positions at major area of Khulna on a Base Map of Khulna city with the help of Geographical Information System (GIS).
- To identify waste generation rate in Kg/Bed/Day
- To make a comparable assessment of different types of hazardous wastes generated in different hospitals and diagnostic centers at Fulbari, Khalishpur, Boyra, Dackbanglow, Sonadanga, major region of Khulna city, with the help of GIS.
- To identify the waste-management practices and technologies those are safe, efficient, sustainable, economic, and culturally acceptable in the context of Khulna city.
- To analyze the suitable treatment options and costs of waste management for decision support with GIS mapping and analysis.
- To suggest recommendations for the proper treatment and or management and finally safe disposal of the health care wastes (HCW).

2. METHODOLODY ADAPTED

2.1 Study Design

Map for the study was prepared in ArcGIS using different layers. The Latitudes and Longitudes (LAT-LON) of about 31 hospital/clinic/diagnostic centers at Fulbari, Khalishpur, Boyra, Dackbanglow, Sonadanga (Khulna) area were collected using mobile Geographical Positioning System (GPS) and layers were made in ArcGIS. For the collection of data and photographs those hospitals were visited. Furthermore surveyed data and information were included in GIS database to prepare various maps.

2.2 Mapping

The ground co-ordinates (LAT-LON) of selected HCEs in the study area were collected by field survey with the help of GPS. After that, those positions were shown on a base map of Khulna city.

2.3 Spatial Data for GIS Mapping

For spatial analyses and mapping, GIS supporting data were collected during the field survey. The data for spatial analyses were collected from primary and secondary sources such government organizations and NGOs. According to HCEs waste category, 4 plastic containers were provided to the HCEs and collected waste generation quantity of different category and evaluated waste generation rates respectively in Khulna city. The attribute data of map features were imported into the GIS environment.

Black	Gray	Yellow	Red
Kitchen wastes	General wastes	Reusable wastes	Sharp wastes

Figure 1: Container for the storage of clinical wastes in generation point *(after Alamgir et al. 2003)*

3. DATA MAPPING, ANALYSIS AND DISCUSSIONS

The study has been carried out in Fulbari, Khalishpur, Boyra, Dackbanglow, Sonadanga and nearby areas in Khulna city. The study area which are mainly residential and commercial areas have developed due to the general public requirements. But hospitals should not be considered as commercial commodity. People are undoubtedly benefited by the modern hospital facilities, but those health care facilities pose a serious health hazard when waste produced in these facilities are not managed with care.

The map showing in Figure 2 has been produced by plotting the ground co-ordinates (Latitude and Longitude collected with GPS) on a GIS base map of Khulna city. The map illustrates that most of the health care facilities in the study area have been developed in the crowded areas of Fulbari, Khalishpur, Boyra, Dackbanglow, Sonadanga and nearby areas. These health care establishments are not environmentally safe for passers-by using the roads as well as for patients taking treatments in these health care facilities.

In Figure 3, observation of hazardous waste generation rates of 24 HCEs in Fulbari, Khalishpur, Dackbanglow, Boyra, Sonadanga and nearby areas in Khulna city has been analyzed and compared graphically. Generation rates of different components of hazardous waste in kg per day have been observed separately. The total hazardous waste generation rate for individual hospital has been divided by the number of beds of the same facility to find the waste generation rate in kg per day per bed. Then the waste generation rates of HCEs have been summed out and then the resultant has been divided by 31 to find out the average hazardous waste generation which is 0.676 kg per day per bed.

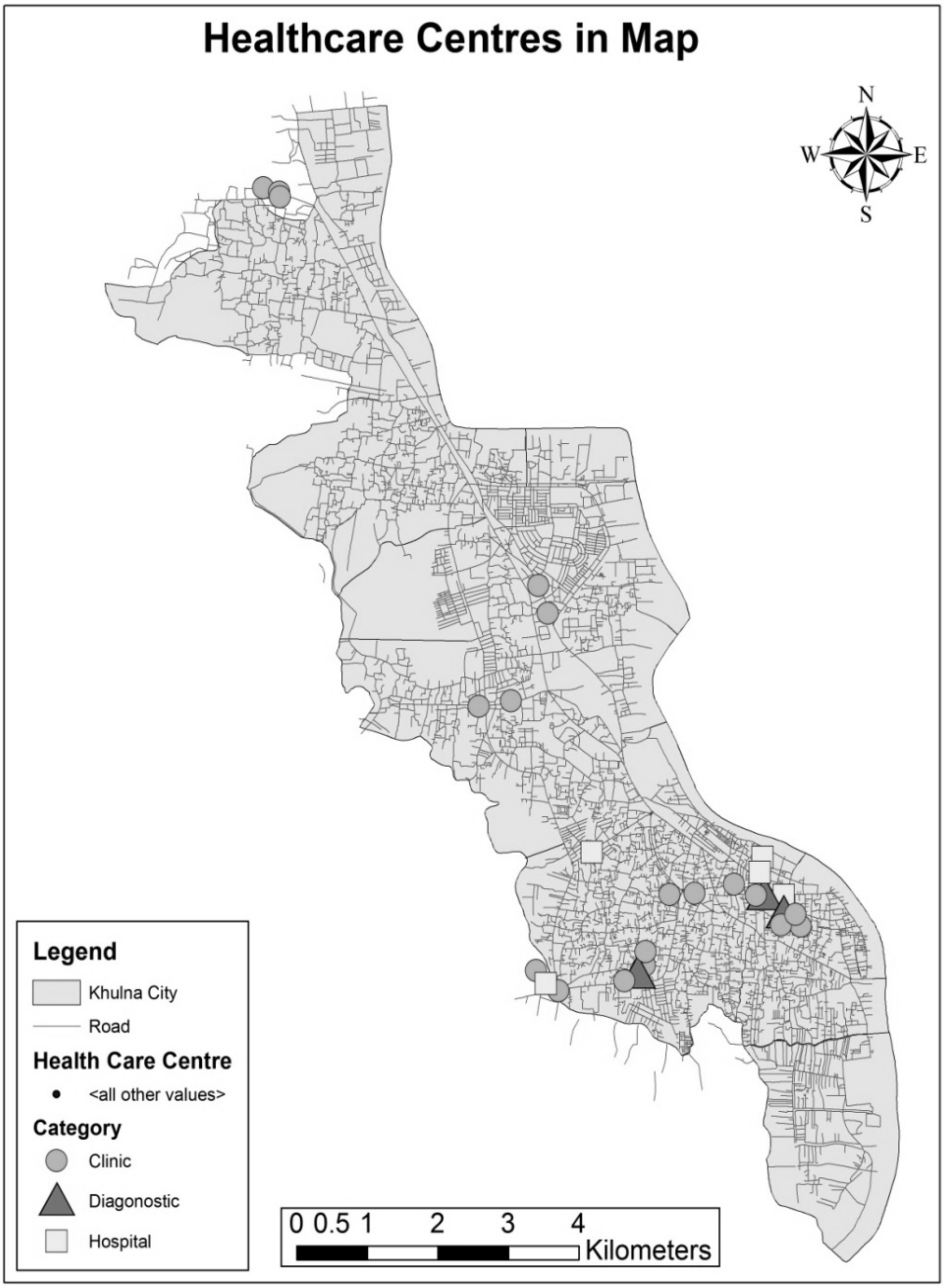

Figure 2: Map of Fulbari, Khalishpur, Boyra, Dackbanglow, Sonadanga and nearby areas (Khulna) showing HCEs

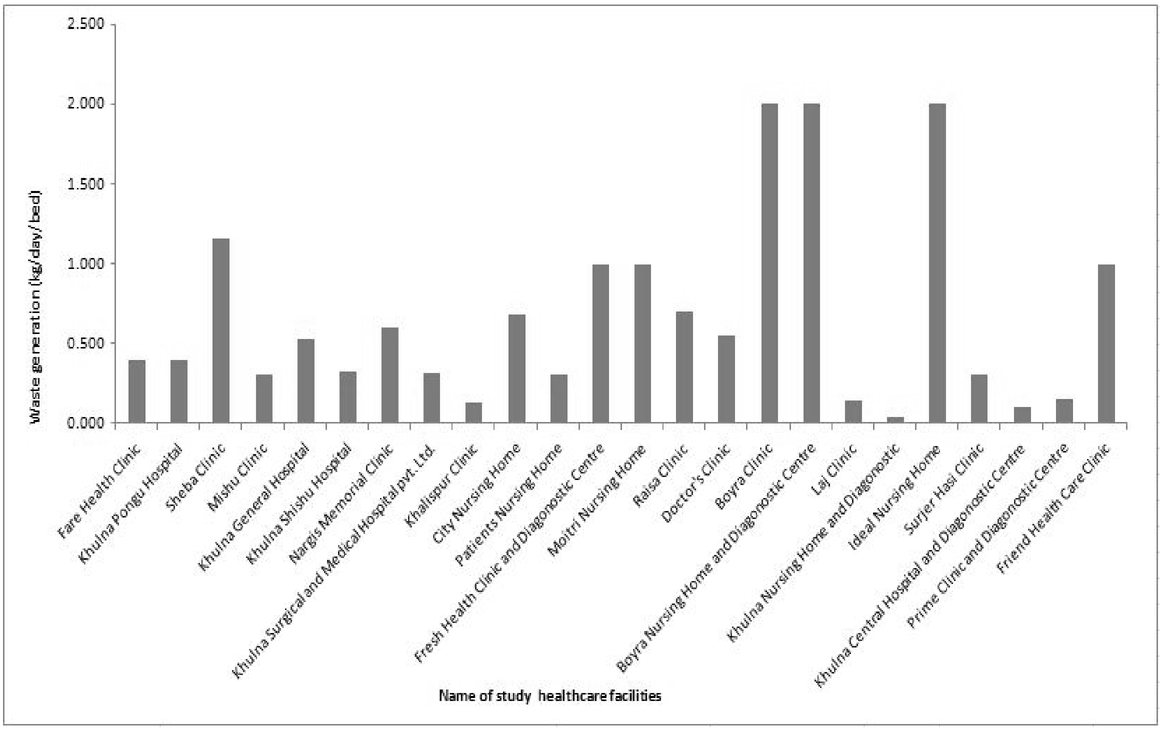

Figure 3: Hazardous waste generation in 24 HCEs in kg/day/bed in Khulna city

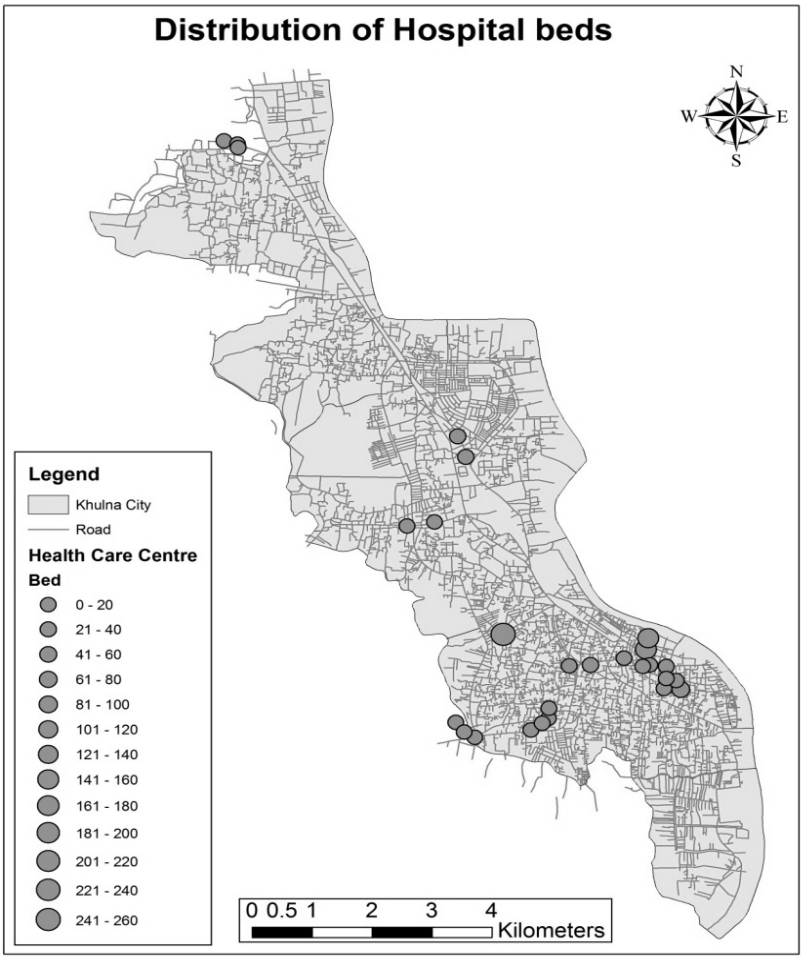

Figure 4: Spatial distribution of hospital beds

The map of Figure 4 is showing spatial distribution of hospital bed in the health care centre. Here the circles are showing number of beds in arrange in ascending format from smaller to larger size. Here most of the hospital's beds are within 0 to 20 ranges. Highest number of beds is 250 in Khulna Surgical and Medical Hospital pvt. Ltd. and lowest is 10 commonly seen in many health care centers. Generally, analysis of bed distribution of HCEs in the study area assists to identify the waste generation rates in kg per day per bed in Khulna City. According to the number of beds in HCEs, it can be assumed the possibility of waste generation in that area and planned a safe system of waste disposal.

In contrast, the spatial distribution of Infectious waste in kg per day of the selected health care centre is provided in the map in Figure 5. Here the rectangles are showing the quantity of infectious waste in arrange in ascending format from smaller to larger size. Also, major quantity of infectious waste ranges within 0.1 to 0.8 kg per day. Highest quantity of infectious waste is 9.33 kg per day in Khulna General Hospital, while lowest is 0.1 kg per day commonly seen in many health care centers. Infectious waste can be considered as hazardous waste. Therefore, this figure analyzes the generation of infectious waste in HCEs in Khulna city as it can be determined a suitable treatment option of infectious waste.

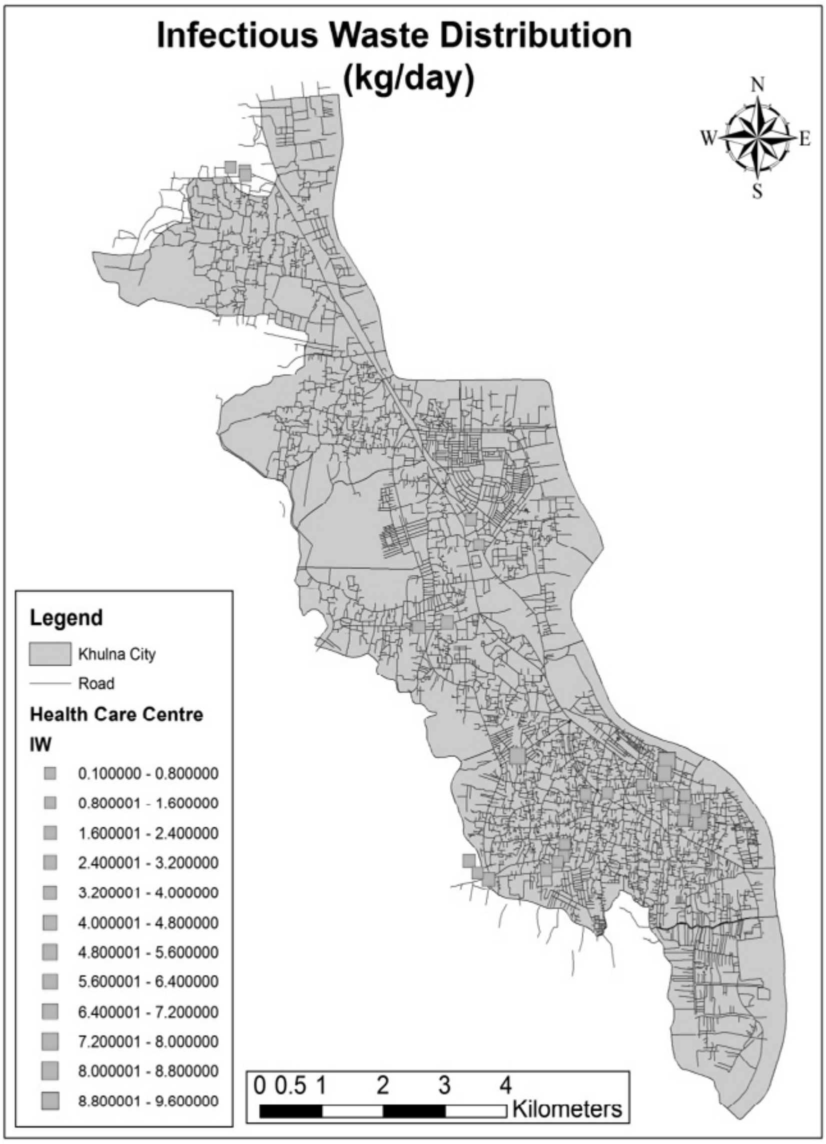

Figure 5: Spatial distribution of infectious Waste

The map of Figure 6 is showing spatial distribution of liquid waste in kg per day in the health care centre. At this point the triangles are showing the quantity of liquid waste in arrange in ascending format from smaller to larger size. Here major quantity of liquid waste ranges within 0.69 to 6.06 kg per day. Highest quantity of liquid waste is 54.67 kg per day in Khulna Surgical and Medical Hospital pvt. Ltd., while lowest is 0.69 kg per day in

Khulna Central Hospital and Diagnostics Centre. Liquid waste can be considered as non sharp waste as well as hazardous waste. So, this GIS map in Figure 6 analyzes the generation of liquid waste in HCEs in Khulna city as it can be determined a suitable treatment option of liquid waste.

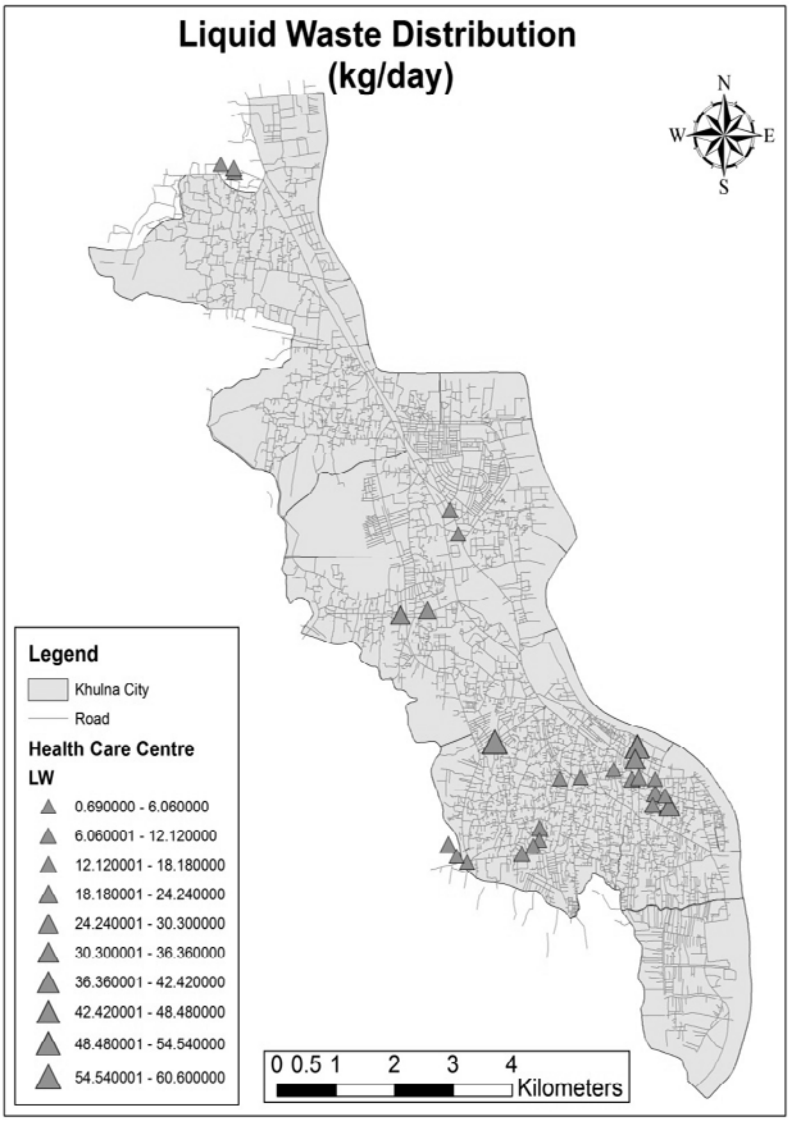

Figure 6: Spatial distribution of liquid waste

Here, it is interesting to note that spatial distribution of plastic waste in kg per day of selected health care centre was investigated and provided in Figure 7. Here the circles are showing the quantity of plastic waste in arrange in ascending format from smaller to larger size. Here major quantity of plastic waste ranges within 0.17 to 1.0 kg per day. Highest quantity of plastic waste is 13.83 kg per day in Khulna Surgical and Medical Hospital pvt. Ltd. and lowest is 0.17 kg per day commonly seen in many health care centers. Plastic waste cannot be considered as hazardous waste. For this it can be collected in a separate plastic container and sometimes it can be made an effort to reuse. Therefore, this GIS map in Figure 7 analyzes the generation of plastic waste in HCEs in Khulna city as it can be determined a suitable treatment option of plastic waste. Also, this analysis makes a contribution in waste management cost in HCEs.

The map of Figure 8 is showing spatial distribution of Sharp waste in kg per day in the health care centre. Here the rectangles are showing the quantity of sharp waste in arrange in ascending format from smaller to larger size. Here major quantity of sharp waste ranges within 0.04 to 0.3 kg per day. Highest quantity of sharp waste is 3.5 kg per day in Khulna General Hospital along with Khulna Surgical and Medical Hospital pvt. Ltd. and lowest is 0.04 kg per day commonly seen in many health care centers. Sharp waste can be considered as hazardous waste and also it can be collected in a separate plastic container for destroying this type of waste fully. So, this GIS map based analysis in Figure 8 illustrates the generation of sharp waste in HCEs in Khulna city as it can be determined a suitable treatment option of sharp waste. Also, this analysis makes a contribution in waste management cost in HCEs.

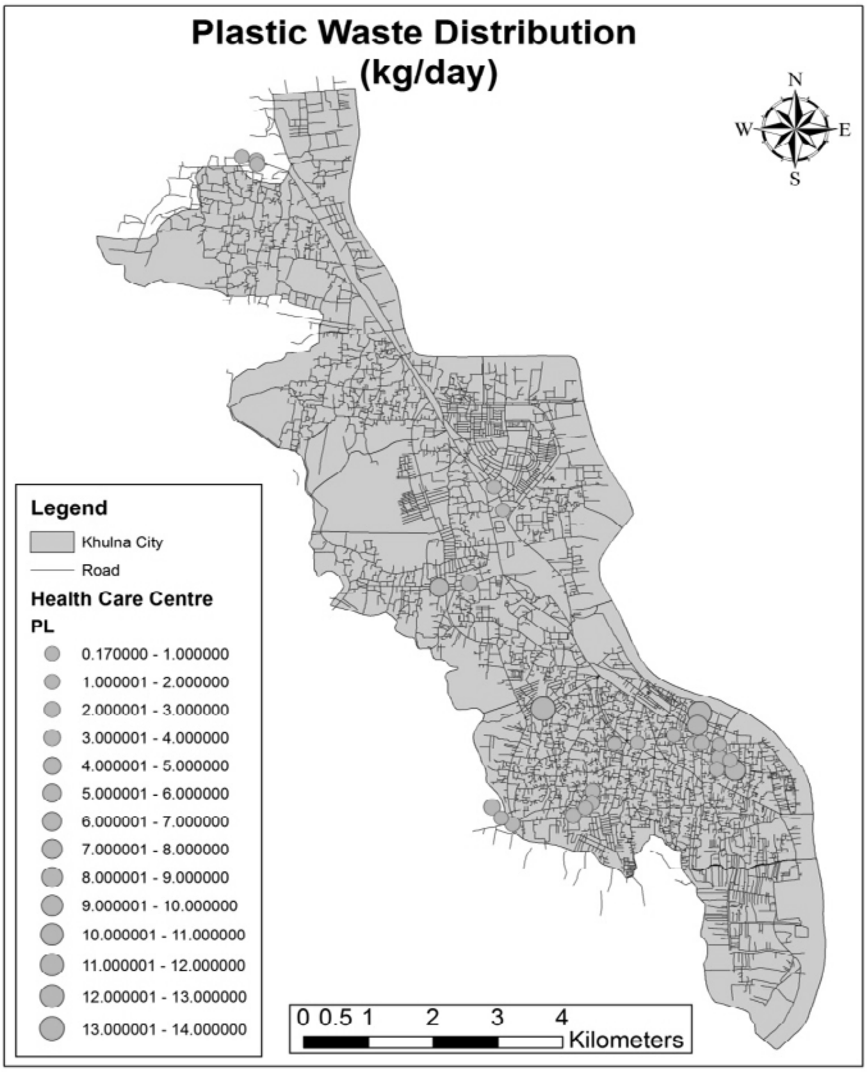

Figure 7: Spatial distribution of plastic waste

The map of Figure 9 is showing spatial distribution of total waste in kg per day in the health care centre. Here the rectangles are showing the quantity of total waste in arrange in ascending format from smaller to larger size. Here major quantity of total waste ranges within 0 to 20 kg per day. Highest quantity of total waste is 80 kg per day in Khulna General Hospital along with Khulna Surgical and Medical Hospital pvt. Ltd. and lowest is 1 kg per day commonly seen in many health care centers. Analysis of total waste distribution indentifies the total quantity of HCEs waste of entire city and separates the different categories of HCEs waste. So, this GIS map based analysis in Figure 9 illustrates the generation of total waste in HCEs in Khulna city as it can be determined a suitable treatment option of HCEs waste. Also, this analysis makes a contribution in waste management cost in HCEs.

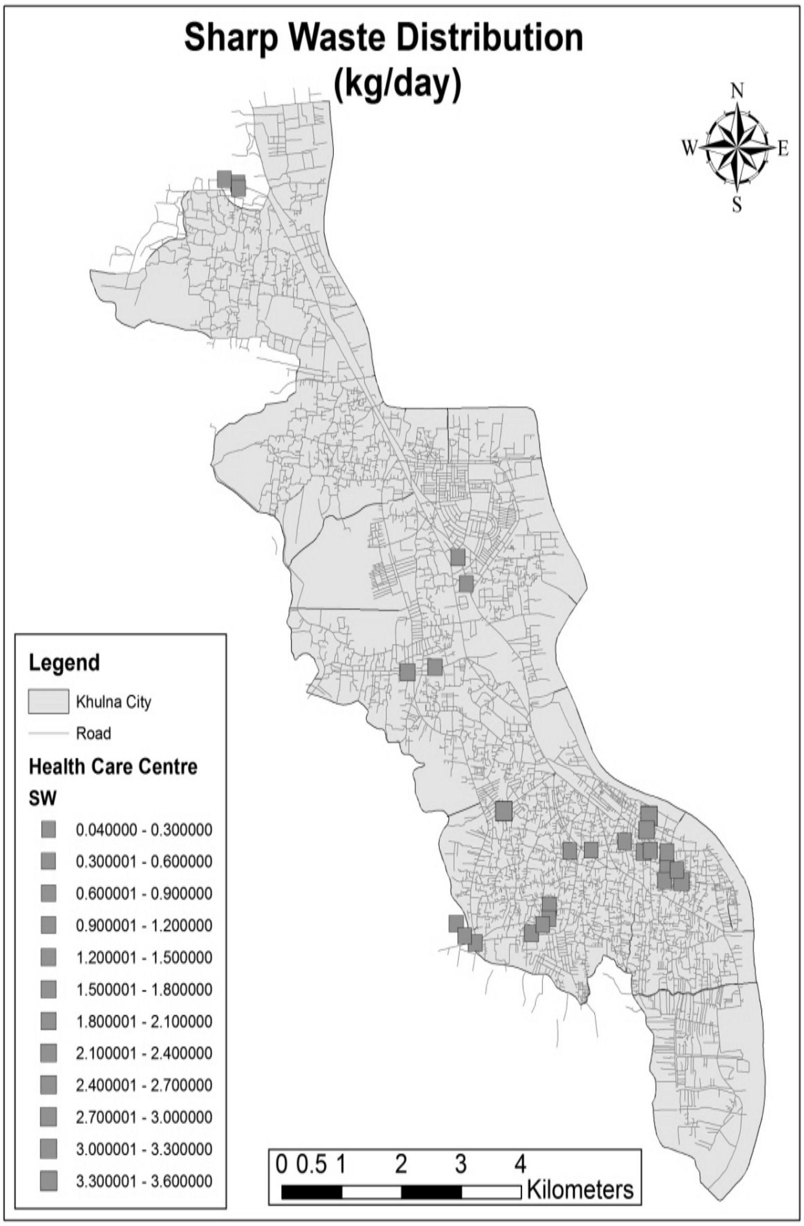

Figure 8: Spatial distribution of sharp waste

The map of Figure 10 is showing spatial distribution of waste management costs in taka per month in the health care centre. Here the circles are showing the quantity of waste management costs in arrange in ascending format from smaller to larger size. Here major amount of waste management costs ranges within 300 to 500 taka per month. Highest amount of waste management costs is 1200 taka per month in Khulna Surgical and Medical Hospital pvt. Ltd. Along with Khalishpur Clinic and lowest is 300 taka per month commonly seen in many health care centers. HCEs waste management costs depends upon the route distance of collection site of HCEs waste and disposal site of waste. Different government or non-government NGO s collects the HCEs waste and they have to be paid a certain amount for waste collection in HCEs. So, this GIS map based analysis can be determined a suitable treatment option of HCEs waste.

The components of clinical waste generation rate are provided in Figure 11. Liquid waste shows the dominant component as 327.47 kg per day, while the minimum is sharp waste presenting 21.13 kg per bed per day on the chart. Others are Plastic waste and Infectious waste showing 83.77 and 55.13 kg per day, respectively.

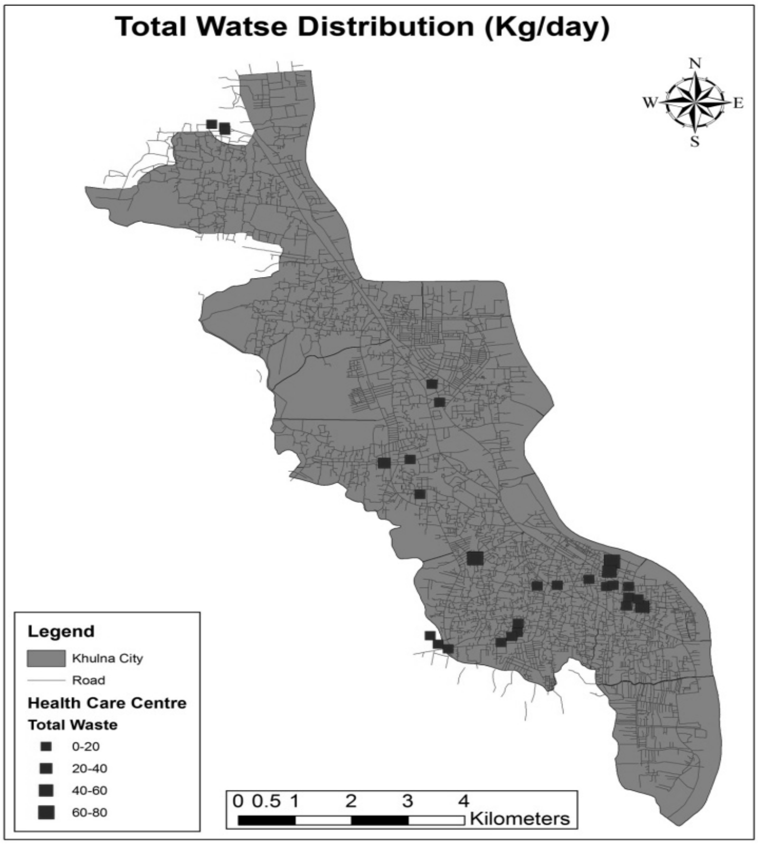

Figure 9: Spatial distribution of total waste

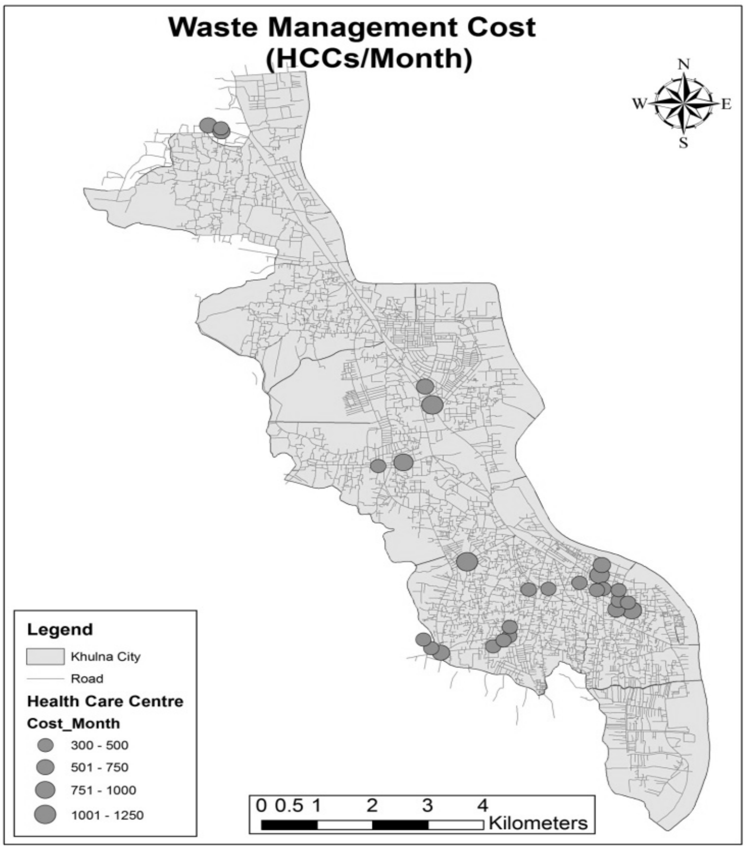

Figure 10: Spatial distribution of waste management costs

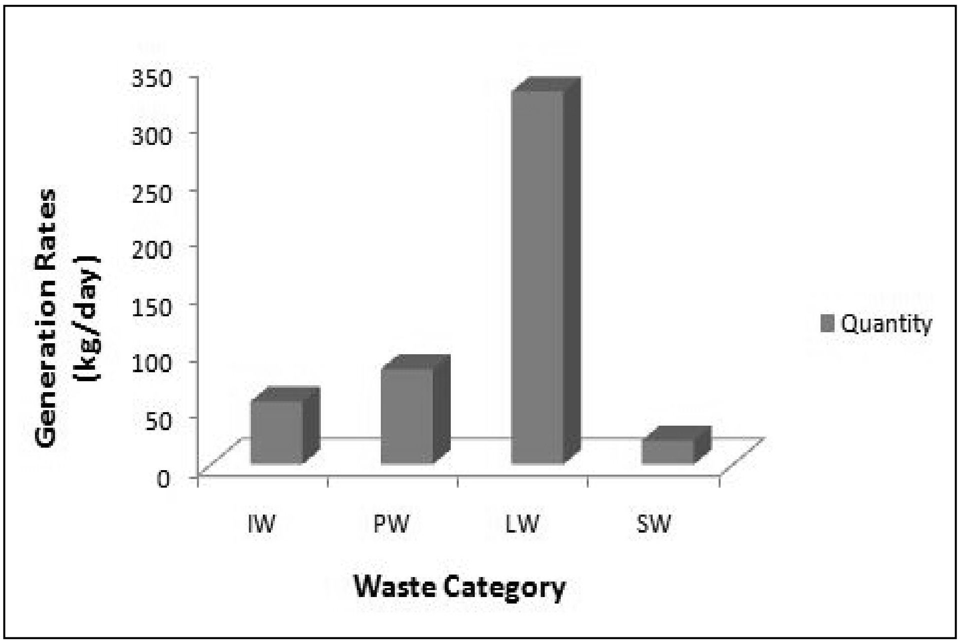

Figure 11: Comparison of component in clinical waste generation.

4. CONCLUSIONS

From the study the average total hazardous hospital waste generation rate was found to be 0.676 kg per bed per day with component wastes' generation rates were 0.06, 0.12, 0.003 and 0.50 kg per bed per day for infectious, plastic, sharp and liquid waste, respectively. Result reveals that liquid waste shows the dominant component as 327.47 kg per day, while the minimum is sharp waste presenting 21.13 kg per bed per day as well as plastic waste and infectious waste showing 83.77 and 55.13 kg per day, respectively. Based on survey, it is very apparent that hospital facilities and waste management in Khulna city is not at all environment friendly, rather degrading the environment with numbers of potential hazards. Moreover, the unhygienic and unsafe disposal of hospital waste in Khulna city poses a serious health hazard to the professionals, workers, and waste collectors, persons involved in recycling and mass people in general. There is lack of proper laws and regulations which specifically can regulate bio-medical waste management. Every HCEs should spend a fixed portion of its monthly or annual budget for waste management because this phenomena is not anyhow less important than treatment. Training must be provided to those who are actually involved with the management of hazardous hospital waste, National level plans and policies are required in this issue. Through this study the estimated clinical waste rate can be used to assess the amount of health care waste generated daily in Khulna city and to design the treatment plant. Finally, it can be concluded that GIS can be used to assess waste generation, generation variation, and collection and transportation routes.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STUDY ON REUSING POSSIBILITIES OF TEXTILE WASH WATER DIRECTLY FOR PRETREATMENT (SCOURING AND BLEACHING) OF COTTON GOODS

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ABSTRACT

Textile dyeing and washing industries in Bangladesh are extracting ground water and releasing to the surface water after use, which creates depletion of water resources eventually threatens our existence. The dyeing industries do not need water as pure as for drinking purpose. The object of the project is to use textile wash water for the pretreatment process of textile goods, instead of using fresh water. The different sources of wash water from yarn dyeing factories are used. Absorbency was found satisfactory for almost all the wash water samples. But, Before Dyeing Rinse (BDR) is the most suitable and highly acceptable than all other selected wash water. It was found the best performing in respect of visual assessment, absorbency, weight loss, color difference, shade matching performance and all the fastness tests. Finally the BDR is selected as the best suited wash water for reusing. The scouring, bleaching and dyeing performances of samples scoured-bleached with BDR wash water is tested against Fresh water samples from the same factory. It is envisaged that, the BDR wash water can be directly used for the scouring bleaching purpose of cotton yarn fabric. For a dyeing mill of 25 ton capacity total saving of water would be 70,000 m^3 per year.

Key words: *Waste water, reuse, scouring and bleaching, dyeing, water saving*.

1. INTRODUCTION

Foreign earning from textile sector rose up to more than 75% in 2005-06 and these industries are providing employment, increasing local incomes, and earning foreign exchange for the country. It is reported that there are around 1500-2000 textile dyeing and washing units at the industrial zone of Bangladesh and few hundreds are in the pipeline. Most of these industries are extracting ground water and releasing it to the surface water directly, mostly without treatment. They are releasing wastes directly into rivers, although the effluents contain pollutants of 10 to 100 times the allowable levels permissible for human health (Diffuse Pollution Conference, 2003). As a result, the surface water body is now vulnerable, aquatic life is endangered and users are facing serious health problems due to contamination by hazardous chemicals from those industries. Wet processing industry uses water in its various steps. Huge amount of water is consumed in dyeing, washing and printing industries from the very beginning of pretreatment stage to finishing. More than 110 liters of water is used to process 1 kg of cotton fabric with reactive dyes in batch process. The water consumption of an average sized textile mill having capacity only 8 tons/day is about 1.6 million liters per day (Choudhury, 2006). The characteristics of the rinse water are variable in different steps in whole batch process. In some stages, it is nearly clear like fresh water and at the post dyeing stages it is colored.

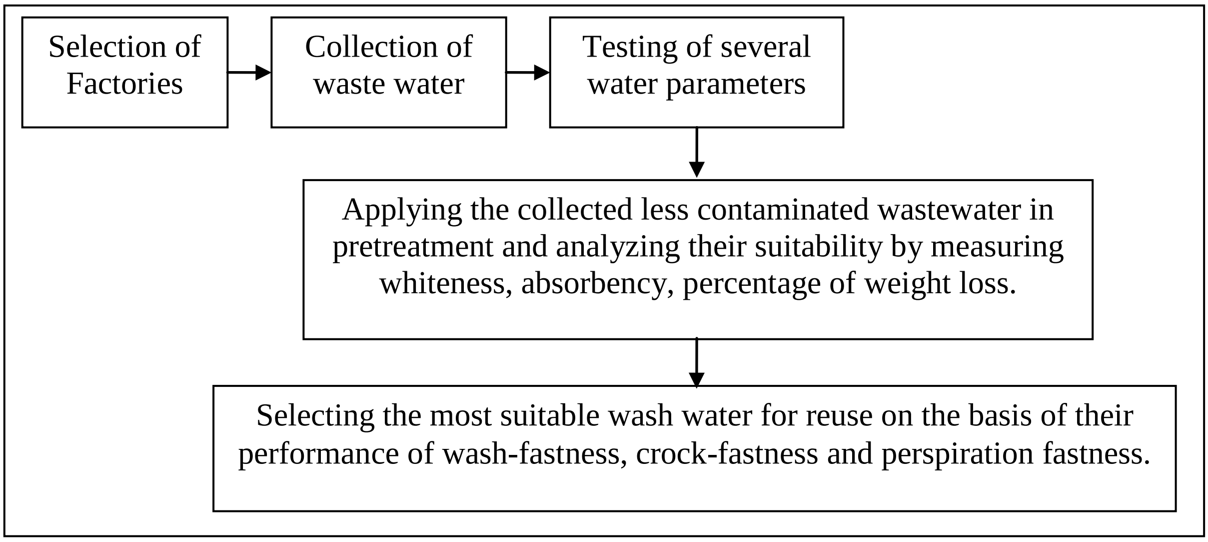
The continuous withdrawal of ground water is responsible for the depletion of our underground water resource and this continuous lowering of the underground water level is threatening our existence. Nowadays, many experts are raising awareness among the government and the general people for the upcoming danger of underground water level lowering problem. But the solutions are not available for us to reduce the daily huge water consumption rate. As the industries do not need water as pure as for drinking purpose, they can reuse their waste water. At present there is little information available where they tried to reuse the textile wastewater for dyeing purpose and could not succeed. But no literature is found regarding reusing of wash water for pretreatment purpose. This research project has illustrated the possibilities of this type of reusing in textile sector as this sector is drawing the maximum amount of water per day. Being a poverty driven country, we should consider both the prosperity of the textile sector as well as the water source and water quality of the country. So an appropriate, cost effective water recycling method could be an option for reducing the pollution level and water withdrawal rate. The main source of water in Bangladesh is the underground water. The increasing demand of water for domestic and industrial purpose creates a large pressure on underground water. The object of the project is to use textile wash water for the pretreatment process of textile goods, instead of using fresh water for the same purpose and thus to reduce the daily fresh water consumption.

The outcome of the research work is directly applicable in the wet processing section (cotton yarn fabric) of the textile industries. The day is coming soon when the underground water sources will not be available for the industrial uses and the factory owners must search for the alternatives.

2. MATERIALS AND METHODS

The raw materials used are grey knitted fabric (Single Jersey) and different chemicals as required in the textile wet processing industries. Major equipments used are Laboratory Dyeing Machine (Sandolab), Woven dryer, Color matching cabinet, Electronic Balance, Data Color and Crocking Meter.

Methodology



Selection of factories:
Factories are selected from different areas of Gazipur districts of Bangladesh which are producing cotton goods by Yarn Dyeing. BOD_5, COD, TSS, TDS, pH, color and hardness were measured of these collected wastewater samples. The results are presented in Table-1.

Table:1 Test results of collected water samples.

Water types	pH	Hardness ($CaCO_3$) (mg/l)	TSS (mg/l)	TDS (mg/l)	BOD_5 (mg/l)	COD (mg/l)	Color (Pt-Co)
Standard for Drinking Water (ECR'97)	6.5-8.5	200-500	10	1000	0.2	4	15
Standard for Textile dyeing	7 – 8						
Yarn Fresh water	7.3	18	NT	132	0	0	NT
DUET fresh water	7.1	140	0	180	0	0	NT
Yarn BDR	7.8	42	10	488	120	287	94

NT - Not Traceable

Selection of Less contaminated wastewater in cotton dyeing cycle:
Less contaminated wastewater means the bath drained with a very few amount of contaminants or almost no contaminants. The following steps of cotton dyeing cycle are considered as comparatively less contaminated wastewater producing steps:

1. Water wasted by ***demineralization*** (Demi.) process.
2. Water wasted by ***rinse after peroxide killing*** (AKR) process.
3. Water wasted by ***neutralization*** (A.W) process
4. Water wasted by ***rinse before dyeing*** (BDR) process.
5. Water wasted by ***rinse before finishing*** (BFR) process.

Other types of water used are: Mixture of various types of selected wastewater ***mixed water*** (MW) and Factory ***fresh water*** (FW).

Comparison between FW Samples with Different Wastewater Samples:
The suitability of different wastewater have determined on the basis of performance testing. The following quality measures are considered:

Measurement of weight loss (Hossain, 2009a)**:**
The weight loss is measured to determine the scouring-bleaching effect. It is measured in percentage by the following formula-

$$\text{Weight loss} = \frac{\text{Grey weight - Bleached weight}}{\text{Grey weight}} \times 100$$

If the weight loss is above 8% it means excess loss of weight or fiber damaged and if it is below 4% it means not scoured properly.

The absorbency test (Spot/drop test) (Hossain, 2009a)**:**
Drop test and spot test:
Drop of 1% Direct Red dyes solution is placed on the fabric surface from a height of 2.5cm with the help of pipette. A stop watch is started as soon as the drop falls on the fabric and stopped no sooner the image of the reflected light disappears at the edge of the drop i.e. the water drop is completely absorbed by the fabric. This is termed as 'drop absorbency time'. A drop absorbency of about 5 sec is generally considered satisfactory for well-prepared cellulosic materials. From the drop test effect it is easy to understand about the absorbency achieved by the fabric, which is usually the first object of scouring and bleaching. Again, by observing the shape of fallen drop the amount and evenness of scouring can be evaluated which is termed as spot test.

Reflectance value of Whiteness (Hossain, 2009a, Choudhury, 2006)**:**
The whiteness of FW sample and the different selected wastewater samples are measured on the basis of reflectance values in percentage. The standard value of the reflectance is 75% to 85% for bleached good [2]. The values are compared in a chart with the reflectance of FW sample to find out whether the residual chemicals present in different less contaminated wastewater can largely hamper the whiteness or its effect is within acceptable limit.

CMC ΔE values of different less contaminated wastewater's Scoured & Bleached sample with Fresh Water Samples.
The color difference CMC ΔE (Color matching committee ΔE) is the difference between the Standard and the sample in the space. CMC ΔE [9] is calculated by the following formula-

$$\Delta E_{CMC} = \sqrt{\left(\frac{\Delta L*}{l \times S_l}\right)^2 + \left(\frac{\Delta C_{ab}*}{c \times S_c}\right)^2 + \left(\frac{\Delta H_{ab}*}{S_H}\right)^2}$$

Where, l & c is the weighting factor and S_l, S_c, S_H are the coordinates factor.

[N.B.: CMC P/F means, if CMC ΔE>1 is known as fail or higher difference and if CMC ΔE<1 it will indicate very minor difference or pass.]

Here, L*= The lightness coordinate, same as L*a*b*

c* = The chroma coordinates, the perpendicular distance from the lightness axis (more distance being more chroma)

H*= The hue angle, expressed in degrees, with 0^o being a location on the +a* axis, then continuing to 90^o for the b* axis, 180^o for –a*, 270^o for –b* and back to $360^o=0^o$.

In the present study CMC ΔE for whiteness is measured by taking FW as standard and other wash water samples as batch. Dyeing performance for shade matching is also determined by applying the above mentioned procedure.

Measuring of Wash Fastness (Hossain, 2009b)**:**
Wash fastness tested in three laboratories namely NITTRAD, Apex Spinning and Knitting Mills Ltd. and Textile Laboratory, DUET by following ISO 105 C06 testing method. The contrast is compared between the treated and untreated sample with the Changing Grey Scale and staining of color in the adjacent multifibre fabric with the Staining Grey Scale. This assessment is done in a color matching cabinet under standard lighting of D65.

Measuring of Colorfastness to Rubbing (Hossain, 2009b)**:**
Rubbing fastness is measured at Textile Lab of DUET through ISO 105 X12, 1993; BS EN ISO 105 X12, 1995 testing method. The contrast between the untreated and treated white rubbing cloth with the Staining Grey Scale measured and rated 1 to 5. This assessment is done in a color matching cabinet under standard lighting of D65.

Color fastness to Perspiration (Hossain, 2009b)**:**
Perspiration fastness is tested in two laboratories namely NITTRAD and Apex Spinning and Knitting Mills Ltd. by applying ISO 105 E04, 1994; BS EN ISO 105 E04, 1996 testing method. The color change and staining of each test specimen is rated under standard lighting D65 using Grey Scale for Color Change and Color Staining.

3. RESULTS AND DISCUSSION

The present study was undertaken to explore the possibility of reusing textile wash water in the same process for the same purpose without or with minor treatment. For this, different types of wash water samples were collected regularly from the selected factories and analysed. Five tests per sample per collection were performed and average values were recorded. Absorbency was found satisfactory for almost all the wash water samples. One example is presented in this section. Other test results are presented through charts and table.

Shade card for absorbency, whiteness and dyeing performance
The shade card is a proof that sample processed with the wash water representing proper absorbency, proper whiteness and proper dyeing. A typical scanned copy of spot/drop test, whiteness and white color difference test and color difference test shade card is shown in Annexure. In respect to spot test performance, it can be said that the selected wash waters are showing quite satisfactory absorbency.

Graphical presentations of Scouring-bleaching and Dyeing performance
The performances for Weight loss (%), Whiteness (Reflectance %), and Color difference (ΔE) are presented through figure 1 to figure 7 and discussion is made with the help of appropriate charts and data tables.

Graphs on Weight Loss:

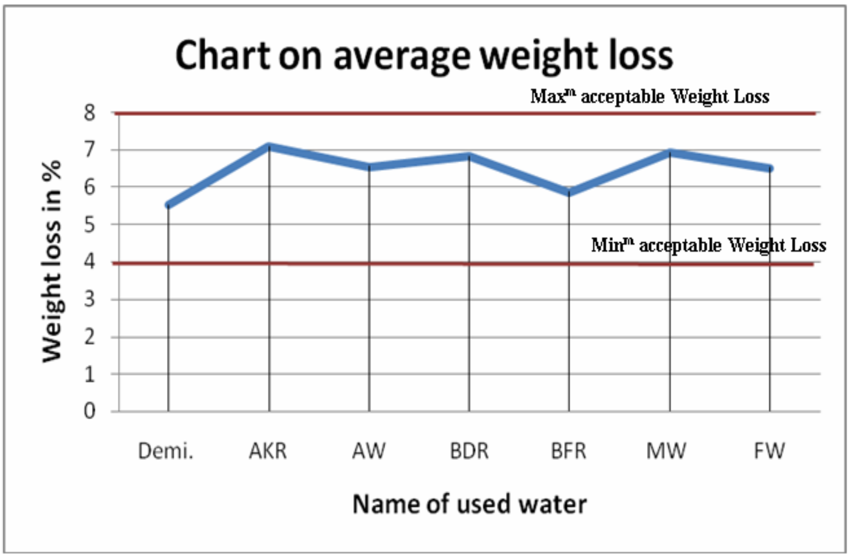


Figure 1: Comparison of average weight loss of sample pretreated in wash water against FW collected from Yarn Dyeing industry.

Graphs on Reflectance:

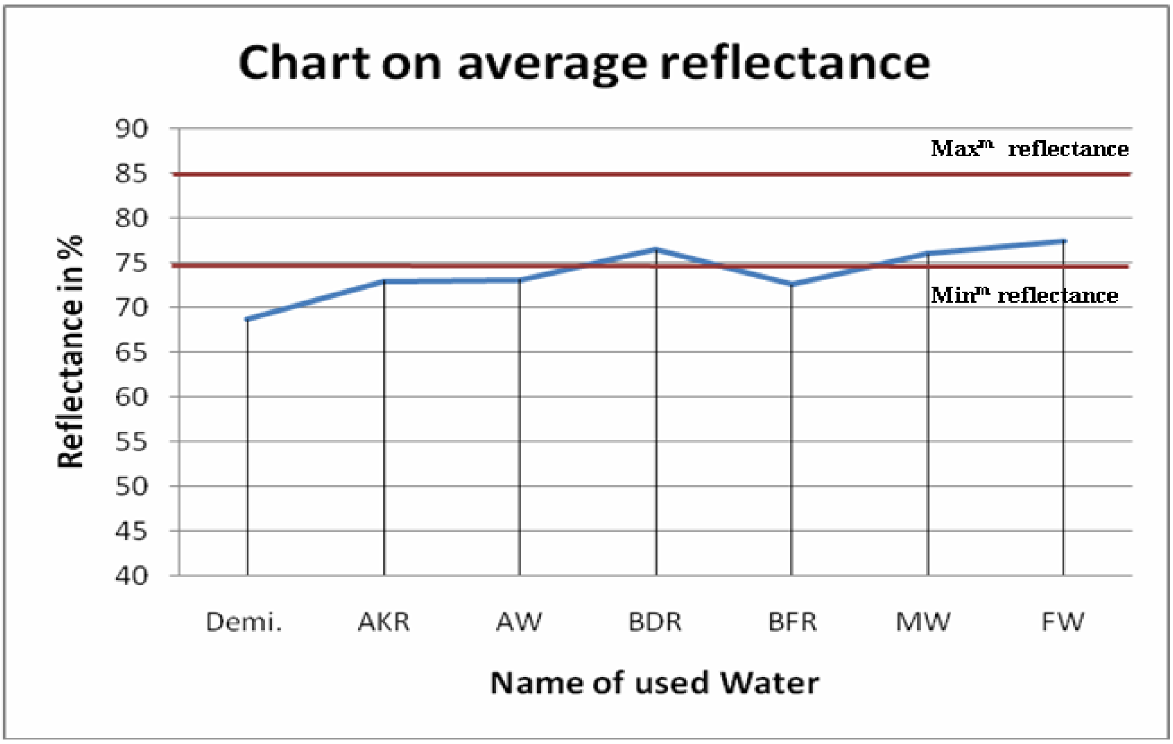


Figure 2: Comparison of average reflectance of whiteness of sample pretreated in wash water against FW collected from Yarn Dyeing industry.

Graphs on CMC ΔE of Whiteness Difference-

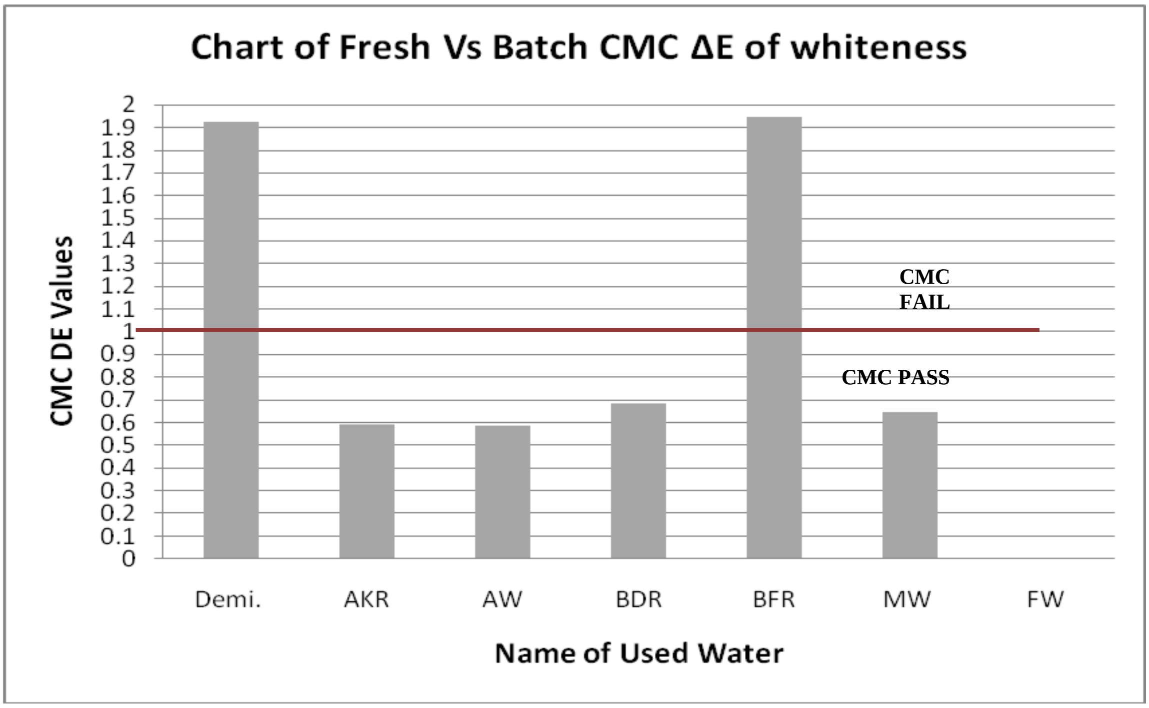


Figure 3: Comparison of average CMC ΔE of whiteness of sample pretreated in wash water against FW collected from Yarn Dyeing industry.

Graphs on CMC ∆E of Color Difference-

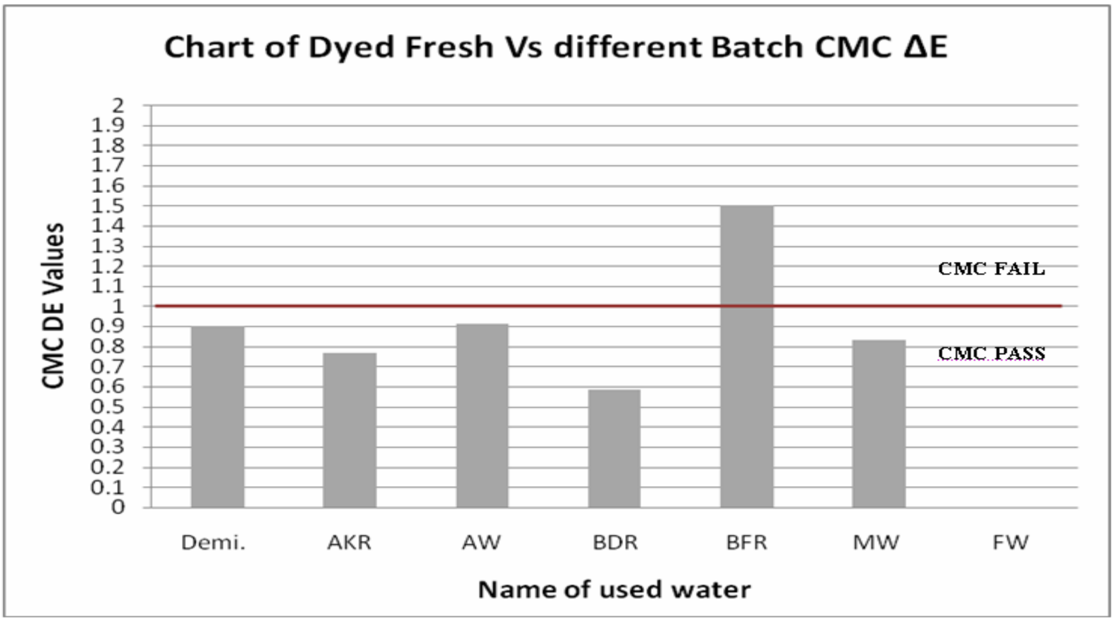


Figure 4: Comparison of average CMC ∆E of color difference of sample pretreated in wash water against FW collected from Yarn Dyeing industry.

Discussion of results and selection of the best reusable wash water

Based on the results presented in the earlier section, following discussions could be derived:

i) Demi. (Demineralization Water):

Demineralization produces bad odor after a few days of storing due to fermentation of the cellulose and other particles. Also the reflectance (68%) showed least value in comparison with the other wash water treated sample, so rejected.

ii) AKR (After Peroxide Killer Rinse):

The fabric sample treated with AKR given better result. It may contain some Peroxide Killing agent which is a risk factor for bleaching. So, this water needs more cautious handling during reusing. Weight loss is high, good whiteness, moderate shade matching performance.

iii) AW (Acid Water, neutralization bath water):

The fabric sample treated with AW found quite satisfactory for reusing. It is same like BDR and the shade matching performances are also very good. This water can be the perfect alternative for BDR where the knit dyeing factories use dye bath enzyme in the absence of BDR. But, if BDR is available, it is surely preferable than AW.

iv) BDR (Before Dyeing Rinse):

BDR is the most suitable and highly acceptable than the all other selected wash water, due to the presence of least amount of contamination. It has been found the best performing in respect of visual assessment, water quality analysis, absorbency, weight loss, color difference, shade matching performance and all the fastness tests.

v) BFR (Before Finish Rinse):

BFR is also the less contaminated as BDR, but it contains faint color after dark shade dyeing (Black, Navy etc). The presence of color has a great shading effect on the pretreated fabric. It has been found that required reflectance is not achieved and CMC∆E values in comparison with for both in whiteness difference and color difference become failed. In the assessment it did not give any good result for all parameters tested. Thus BFR has been rejected considering the above cases.

vi) MW (Mixed water):

MW is the even mixture of all less contaminated wash water. MW will provide better result if BFR and Demi water is absent in the mixture. But surely it is not as clear as BDR and also not capable to show the best whiteness and shade.

On the basis of above charts and discussions, finally the BDR is selected as the best suited wash water for reusing.

The performance of BDR:

The scouring, bleaching and dyeing performances of samples scoured-bleached with BDR wash water is tested against Fresh water samples from the same factory. The performances are shown through charts and tables below. Here the wash water samples are collected from different factories at different dates. “Collections” in the charts means the different collection dates. Data given are the average values of five tests for each individual type of wash water samples for each factory at each date.

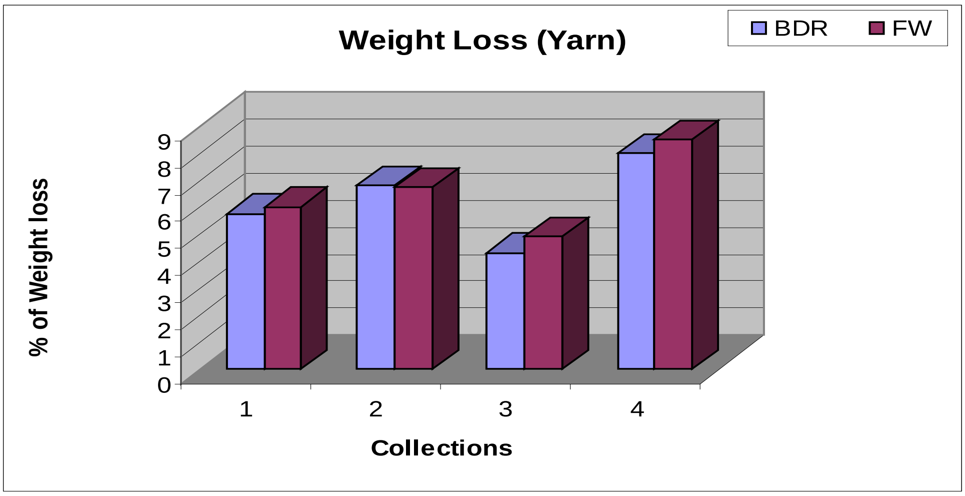


Figure 5: Weight loss performance of BDR and FW sample (Yarn).

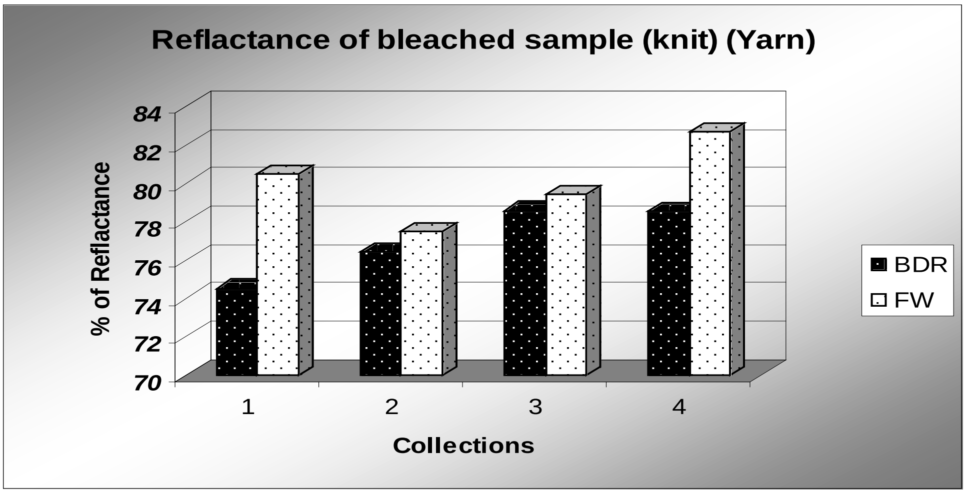


Figure 6: Reflectance value of BDR and FW for bleached sample (Yarn)

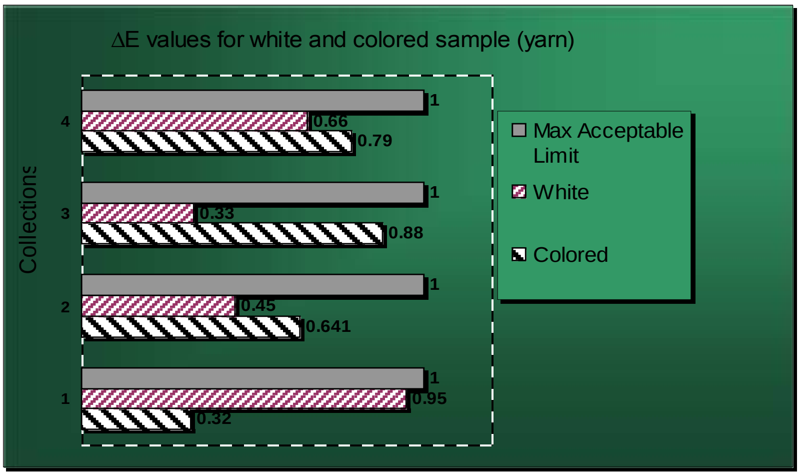


Figure 7: ΔE values for white and colored sample (yarn)

The graphs showed that the weight loss of the Scoured-bleached samples with Before Dyeing Rinse (BDR) wash water is similar to the Factory-Fresh Water (FW) samples for yarn dyeing. The reflectance values are not as good as the fresh water samples but it is quite acceptable. Again the ΔE values for the white samples and dyed samples are very good. Almost all the ΔE values lie below maximum acceptable limit i.e., 1. The Table-2 below showed the fastness rating for a dyed sample which was scoured and bleached with BDR wash water. Here all the fastness ratings are within the acceptable limits.

Table– 2: The fastness rating of dyed sample scoured-bleached with BDR wash water

Different Fastness Rating								
Collections	Yarn Dyeing							
	Rubbing		Washing		Perspiration			
	CIC	CIS	DR	WR	ACID		ALKALI	
					CIC	CIS	CIC	CIS
1	4.5	4.5	5.0	4.5	4.5	4.5	4.5	4.5
2	4.5	4.5	5.0	4.5	4.5	4.5	4.5	4.5
3	4.5	4.5	5.0	4.5	4.5	4.5	4.5	4.5
4	4.5	4.5	5.0	4.5	4.5	4.5	4.5	4.5

Water Saving:
The other objective of the present work was to reduce the load on underground water and also to reduce the load on Effluent Treatment Plant (ETP) eventually saving environment. A quantitative explanation of the above possibility is given below:

Name of the dyeing factory: TEXEUROPE (BD) LTD-GAZIPUR (25 tons/day), M**:**L = 1**:**8

Calculations:
A dyeing machine (capacity 500 kgs.) will use 500 X 8 = 4000 liters of water for each bath. Thus BDR will also need 4000 liter of water, which can be saved for 500 kgs fabric (0.5 ton). That is, 8000 liters of water can be saved for dyeing 1 ton fabric.

Thus, for TEXEUROPE (BD) LTD., GAZIPUR, total saving of water per day = (25X8000) 200000 liters and per year amounting 70 million liters or 70,000 m^3 for one dyeing factory.

4. CONCLUSIONS

In batch wise dyeing of cotton yarn or knit fabric with reactive dyes, the most clear, less contaminated used water, from the pretreatment processes before taking the dye bath water, is reusable for the scouring and hydrogen peroxide bleaching bath/bathes of next batch/batches with satisfactory results in respect of absorbency, weight loss and whiteness. However, the shade matching performance is very good for deep shade, good for medium shades and good to satisfactory for the light shades. The rubbing, wash, and perspiration fastness is very good.
So, it can be easily said that the Before Dyeing Rinse (BDR) wash water can be directly used for the scouring bleaching purpose of cotton yarn fabric. Also water saving of 8000 liters per day could be achieved for dyeing 1 ton fabric. In the present work, only one step is selected for reusing purpose. But there were several other steps from which one can reuse water without any hesitation, specially for deep shades.

Scope of further research:
To save this huge amount of fresh water per day, further research is needed to suggest the necessary dyeing machine modifications. Presently, the establishment of water act is underway and this is the high time to think about underground water pumping cost so that the industrialists would consider the price of water.

Beside this dyeing machine modification, the reusability of these wash water like BDR, AW, ASR, AKR in different other steps like soaping and after dyeing rinsing is a vital issue for further research. Also, further study could be undertaken to investigate the possibility of reusing any of these rinsing water in the agricultural or irrigation purposes or even for recharge of the underground water. .

ACKNOWLEDGEMENT

The present research work was carried out under the financial support of International Training Network (ITN), BUET, Bangladesh. The authors are gratefully acknowledged to the authority of DUET and ITN BUET for their all out support in completing the work.

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Annexure:

Figure I: Sample Card Showing Comparison of Whiteness.

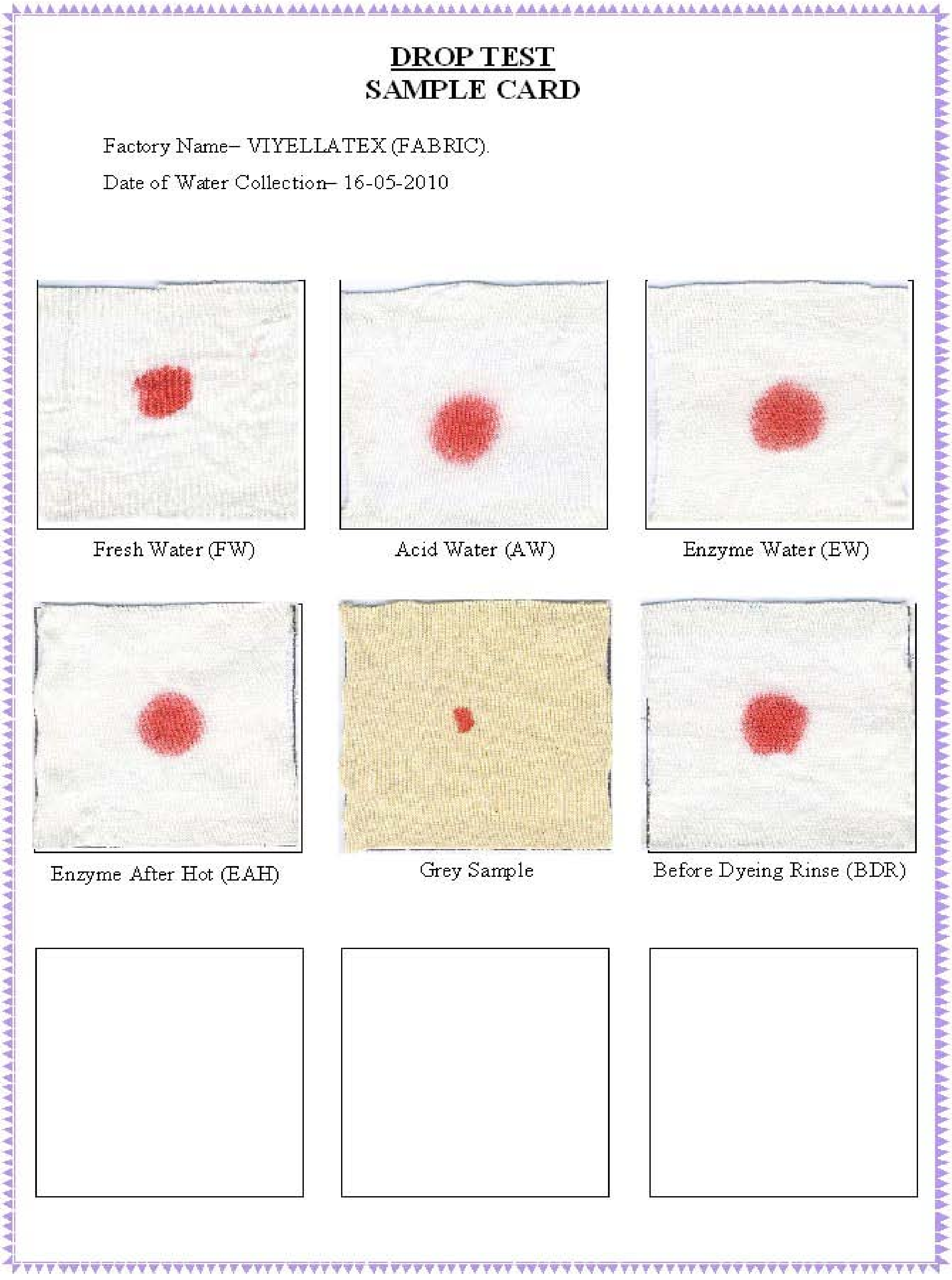

Figure II: Drop/Spot test Sample Card

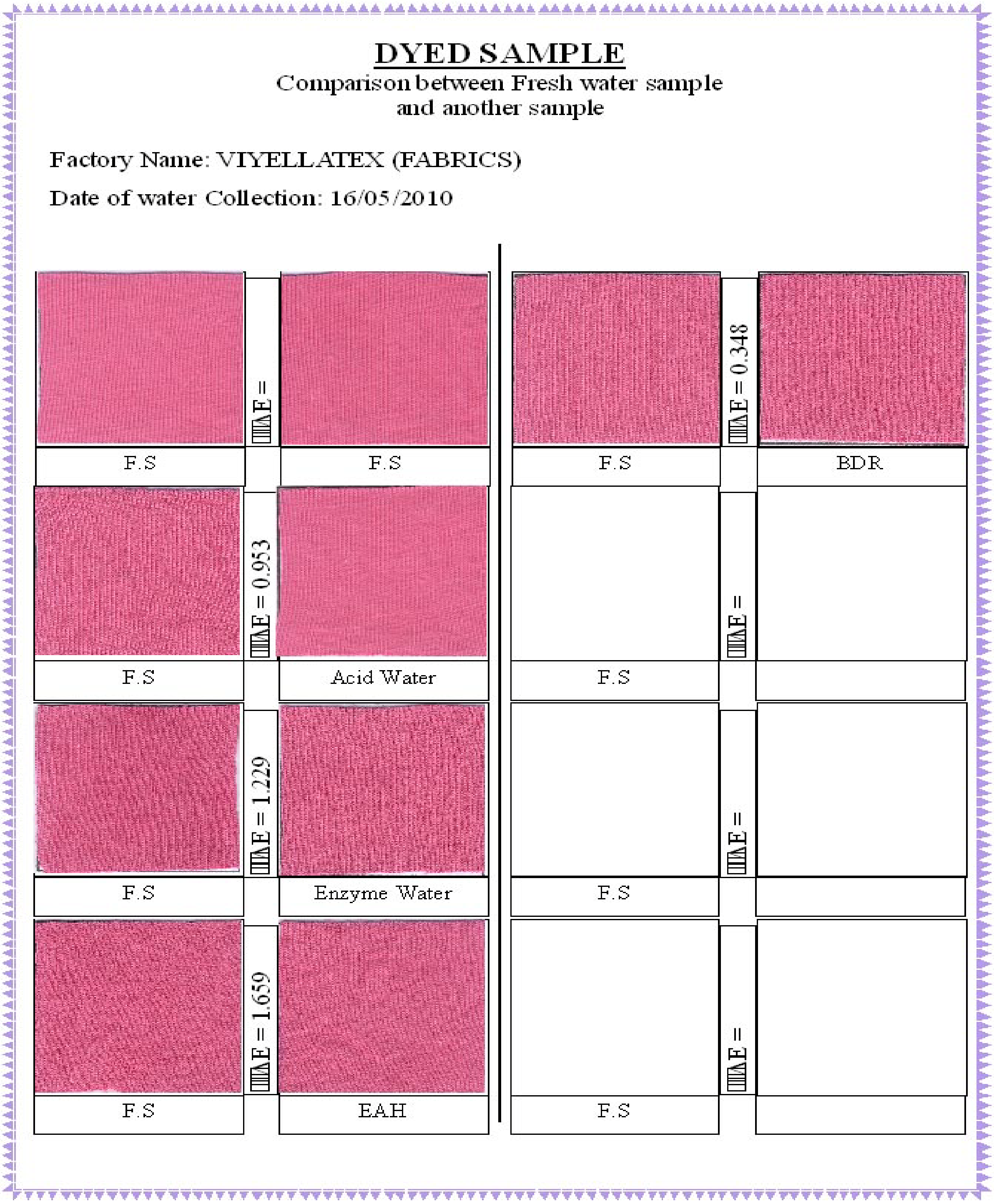

Figure III: Sample Card Comaring Dyeing Performance

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

METHYLENE BLUE ADSORPTION BY RICE HUSK ASH

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ABSTRACT

Methylene blue is a heterocyclic aromatic chemical compound extensively used for the production of permanent dye in the various textile industries of Bangladesh. It is responsible to impart the color in the rivers where textile water is directly discharged without any treatment. It resists the sunlight penetration into the water and thus creates a great threat to the water lives. Adsorption is a very popular method for the removal of color from water but the available adsorbent materials are very much costly. Activated Carbon produced from the rice husk can be effectively used for the removal of Methylene blue from the color solution. But the residue of the rice husk activated carbon may also be used for adsorption of color from a solution. In this study it is shown that a residue which is obtained from the production of activated carbon from rice husk called rice husk ash (RHA) can be used for the removal of Methylene blue from the color solution in an almost equally effective way as like as rice husk activated carbon. Both rice husk activated carbon and rice husk ash may be the much more preferable materials than the available materials which are widely used for the treatment of textile effluent in terms of low cost of production, economy and efficiency. From the experiments it is shown that at 45^0C temperature 3gm rice husk ash can remove the 99% Methylene blue from the 100mL water in 50 minutes. While for the rice husk carbon it takes 40^0C temperature and 3.0gm to remove the 99% Methylene blue from the 100mL water in 60 minutes. For the same criteria that is 45^0C temperature and 50minutes contact time in case of commercial grade activated carbon the removal efficiency is lower than that of both rice husk activated carbon and rice husk ash. In case of rice husk activated carbon and rice husk ash the color removal efficiency is 99% while in case of commercial grade activated carbon in crushed state the color removal efficiency is 35%.

Key Words: *Methylene blue, Heterocyclic aromatic chemical, Adsorption, Activated carbon, Rice husk.*

1. INTRODUCTION

Textile operations are extremely water and energy intensive and textile effluents contain high concentrations of salts, total suspended solids, color, chemical oxygen demand (COD), nutrients(nitrogen and phosphorous), and toxic compounds, such as surfactants, heavy metal, and chlorinated organic compounds(Correia et al. 1994). In additions, effluents created by methylene blue which is used as a permanent synthetic dye compound is very much esthetically offensive, can interfere with UV disinfection of wastewaters and interrupt photosynthesis in receiving waters. The chemical structure of methylene blue shown in figure 1 and the projected area of the methylene blue molecule has been given as 135 A^2 (Kipling and Wilson, 1960; Haul, 1966).

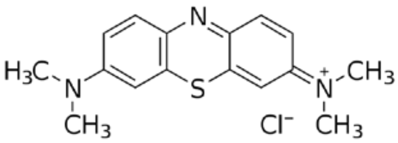

Figure 1: Chemical Structure of Methylene Blue

If water bodies receiving such waste water containing methylene blue without proper treatment then adverse environmental impacts can happen on contact with this wastewater, like allergic dermatitis, skin irritation and mutation etc. to humans since Methylene blue is carcinogenic and highly toxic to humans. Methylene blue resist rubbing, light exposure, and oxidizing or reducing agents when it is mixed with water to produce dye and are therefore also highly resistant to conventional wastewater treatment processes, such as aerobic biodegradation. Direct discharge of dyes containing effluents into municipal environment may cause the formation of toxic carcinogenic breakdown products. The highest rates of toxicity were found amongst basic and diazo direct dyes

(Lata et al., 2007; Wang et al., 2008a). Research on advanced treatment of textile effluents has focused on a variety of technologies including ozonation (Perkowski et al. 1996; Aplin and Waite 2000), chemical reduction (Cook 1996; Laszlo 1997), electrochemical oxidation (Lin and Peng 1994; Gutierrez and Crespi 1999; Hao et al. 2000), anaerobic biological treatment (Delée et al. 1998; Beydilli et al. 1998), Fenton's reagent (Ince and Tezcanli 1999), granular activated carbon (GAC) adsorption (Nicolet and Rott 1999), and coagulation (Tunay et al. 1996). But all these technologies are so much expensive for a country like Bangladesh.

Rice husk ash is an agricultural residue abundantly available in rice producing countries. Rice husk is generally not recommended as cattle feed since its cellulose and other sugar contents are low. Industries use rice husk as fuel in boilers and for power generation. Rice Husk Ash is a substance which is obtained during the production of Rice Husk activated carbon. The rice husk ash which is used in this study was obtained from the burning of rice husk at a temperature 86^0C in a furnace made of 1.00 mm MS steel sheet and has a diameter and height 35.56 cm and 50.8 cm respectively with two 7.62 cm × 15.24 cm openings at the bottom side. (Mahmudur Rahman, 2011). It works as an excellent adsorbent material due to its highly micro porous structure. The Rice Husk Ash is one of lignocelluloses material which consist of lignin, cellulose and xylon which apart of biomass material with high content of carbon and silica.

The aim of this study was to investigate the efficiency of rice husk ash for the treatment of methylene blue. It is observed that the ability of rice husk ash to remove color from the solution of methylene blue at the optimum condition is almost 99%. The optimum treatment conditions that are temperature, contact time and amount of dose were fixed by the regular interval of trial process and finally the favorable condition was verified by the Freundlich adsorption isotherm. For the same treatment temperature and contact time the industrial grade activated carbon had the 35% color removal efficiency from the Methylene blue solution in the crushed state (Mahmudur Rahman, 2011)

2. METHODOLOGY

The mechanism which is involved for the color removal of methylene blue solution by rice husk ash is adsorption. In this study, method of temperature effect, contact time and amount of dose on the efficiency of rice husk ash has been experimentally investigated. The methods are given in below in a precise form

2.1 Characterization of RHA

Rice Husk Ash was characterized by determining its moisture contents and volatile solid contents. The characterization results are shown in Table-1 and Table-2

Table-1: Characterization chart of Rice Husk Ash

	Sample-1	Sample-2	Sample-3	Average
Moisture Content (%)	5.63	5.24	5.32	5.396
Volatile Solid Content (%)	1.80	0.47	0.42	0.896
Ash Content (%)	98.84	98.80	98.95	98.863

2.2 Preparation of Standard Solution

Standard solution of methylene blue was prepared by mixing 20mg methylene blue with 1000mL distilled water. The amount of methylene blue in the standard solution was fixed to the 20mg which gave the color 185Pt-Co by "Hach (Spectrophotometer)".

2.3 Preparation of Batches for fixing the Ambient Conditions

The ambient treatment conditions are temperature, contact time and dose. Five batches of standard methylene blue solution were prepared by taking 100mL Methylene blue solution in five conical flasks as shown in figure 2 with a trial dose of Rice Husk Ash started from 0.5g.

Figure 2: Preparation of Batches with a trial dose of Rice Husk Ash.

2.4 Shaking of Prepared Batches

The prepared batches with the trial dose of RHA were set for shaking in the orbital shaking machine as shown in figure 3. In all of the trials the shaking speed was set at 200rpm and the shaking was done for 5minutes. The effect of contact time was considered after the 5 minutes except the room temperature (30^0C).

Figure 3: Shaking of Prepared Batches.

2.5 Application of Heat

The batches were put into the “Philip-Harris” water bath as shown in figure 4 with a trial temperature started from the room temperature (30^0C).

Figure 4: Application of heat with a trial temperature.

2.6 Filtration of the Batches and determination of Color

After completion of the heating process the batches were allowed to filtrate through the "Whatman" filter paper and the filtrated samples were collected in beakers as shown in figure 5. Color of the filtrated samples was determined by the "Hach Spectrophotometer".

Figure 5: |Filtration of the Batches

2.7 Continuation of the Trial Process

The whole trial process was continued until finding the optimum treatment conditions as well as temperature, contact time and amount of dose of the Rice Husk Ash.

3. RESULTS AND DISCUSSIONS

The experiments were continued until fixing the highest removal efficiency for the effect of temperature, contact time and the amount of dose of Rice Husk Ash. The results are shown in below

3.1 Temperature Study

Starting from the room temperature the experiments for fixing the ambient temperature were carried out for three different doses of Rice Husk Ash and for three different contact times.

Table 2: Temperature effect on the efficiency of rice husk ash

		Dose 2g Contact time 50min.	Dose 3g Contact time 60min.	Dose 4g Contact time 70min.
Temperature (^{0}C)	Raw Color (Pt-Co)	Average % Efficiency	Average % Efficiency	Average % Efficiency
30	185	54.4865	55.6757	58.1622
35	185	58.3784	60.973	71.5676
40	185	72.4324	78.1622	80.1081
45	185	81.1892	88.5405	84.4324
50	185	68	87.027	82.3784
55	185	56.5405	74.4865	66.3784
60	185	49.0811	60.1081	57.4054
65	185	41.1892	49.0811	46.1622
70	185	32.6486	42.7027	38.5946
80	185	20.3243	32.8649	33.5135

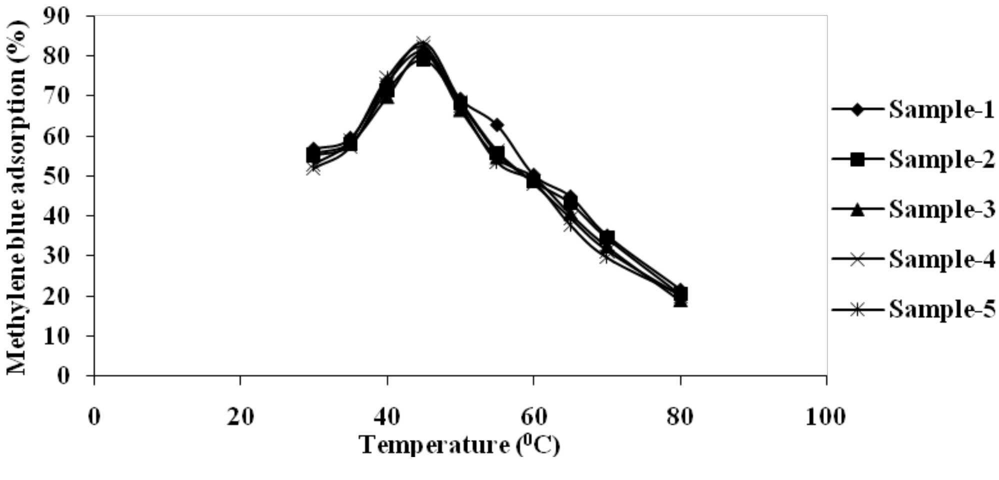

Figure 6: Temperature Effect on the Efficiency of Rice Husk Ash for 50 min. contact time and 2g dose.

In all three cases with the increasing of heat the removal efficiency was increased and at 45^{0}C temperature maximum color removal efficiency was found as shown in figure 6- figure 8 After 45^{0}C temperature in all three cases the efficiency started to decrease with a considerable rate. Table 2 is showing the results of the temperature effect on RHA

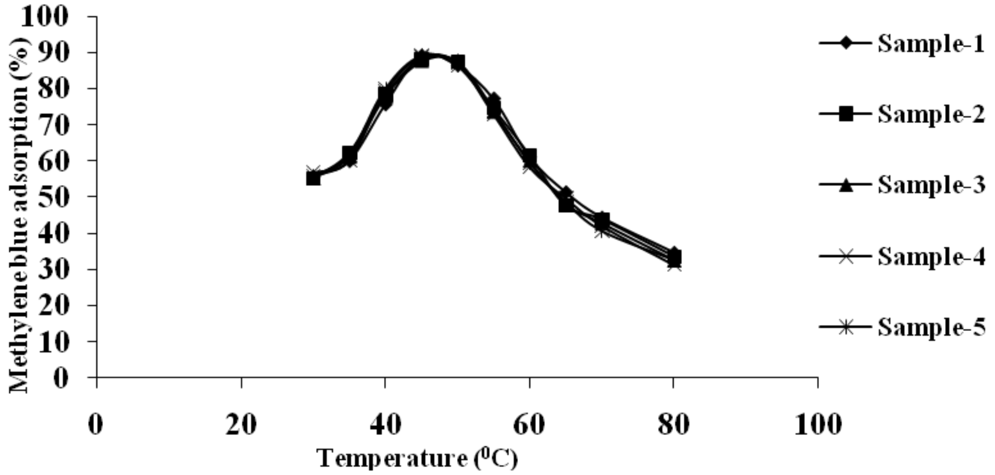

Figure 7: Temperature Effect on the Efficiency of Rice Husk Ash for 60 min. contact time and 3g dose

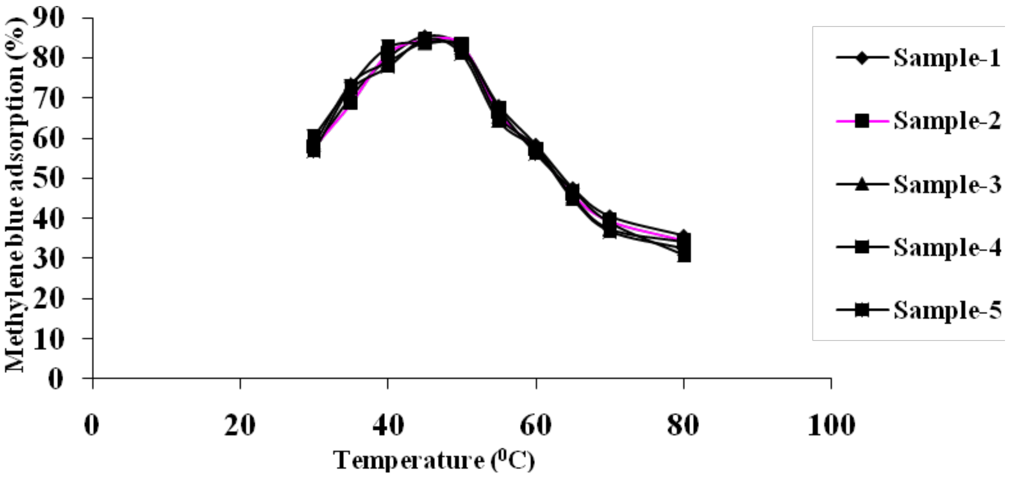

Figure 8: Temperature Effect on the Efficiency of Rice Husk Ash for 70 min. contact time and 4g dose

3.2 Contact Time Study

For the contact time study the temperature was fixed at 45^0C. Started from 0.5 minute the experiments were continued to 80 minutes for three different doses and contact time. Results are shown in Table 3. In all three cases it was observed that Rice Husk Ash had the ability to hold the color for 50 minutes and after 50 minutes it started to release the color. The color removal efficiency with the increase of contact time has shown in figure 9-figure 11.

Table 3: Contact time effect on the efficiency of rice husk ash

			Dose 7g	Dose 8g	Dose 9g
Temperature (°C)	Contact Time	Raw Color	Average % Efficiency	Average % Efficiency	Average % Efficiency
45	5	185	20.5405	33.2973	50.4865
	10	185	31.5676	47.3514	59.027
	15	185	36.7568	62.5946	64.5405
	20	185	41.2973	68.8649	66.9189
	25	185	47.7838	78.7027	70.9189
	30	185	56	85.2973	76.1081
	35	185	62.8108	87.8919	85.5135
	40	185	69.8378	92.6486	92.2162
	45	185	90.3784	95.8919	99.1351
	50	185	97.9459	99.2432	100
	55	185	92	97.0811	97.6216
	60	185	79.027	90.5946	90.9189
	65	185	63.3514	78.0541	79.8919
	70	185	46.1622	67.6757	71.4595
	75	185	37.5135	45.9459	55.4595
	80	185	31.1351	40.1081	46.1622

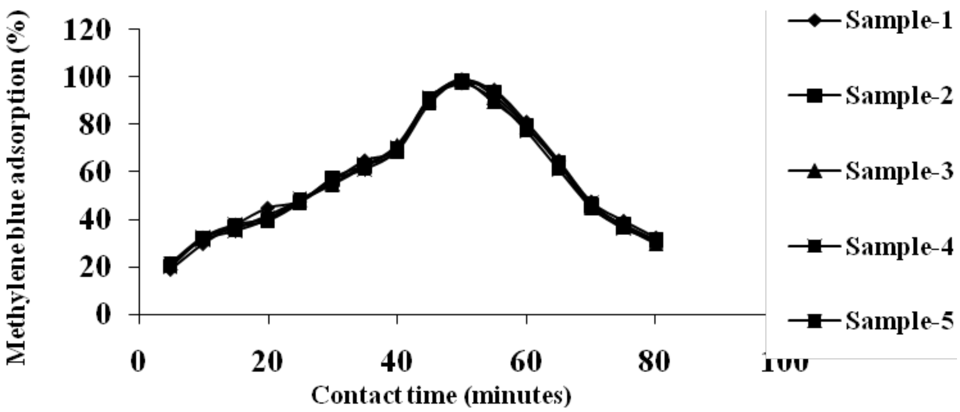

Figure 9: Effect of contact time on the efficiency of the Rice Husk Ash at 45^{0}C and for the dose 7g

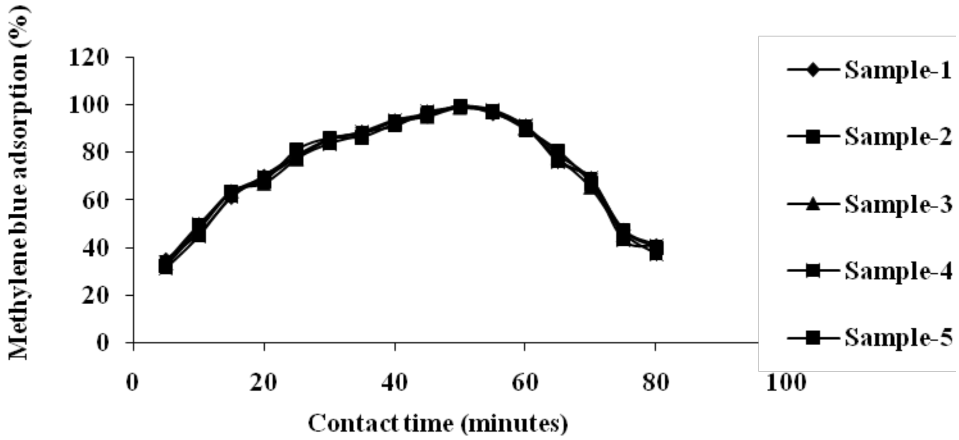

Figure 10: Effect of contact time on the efficiency of the Rice Husk Ash at 45^0C and for the dose 8g

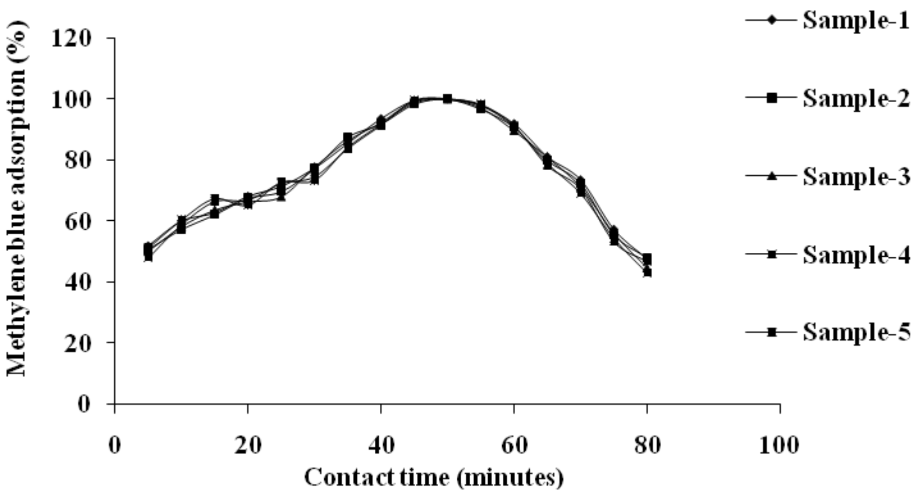

Figure 11: Effect of contact time on the efficiency of the Rice Husk Ash at 45^0C and for the dose 9g

3.3 Dose Study

Dose study of Rice Husk Ash was started from 0.5g. During the entire dose study temperature and contact time of the experiments were fixed to 45^0C and 50minutes respectively as shown in figure 12. The results have shown in Table 4. With the increase of the amount of dose the removal efficiency gradually increased and at 3g the efficiency is enough to reduce the color into the accepted range (Pt-Co). Beyond 3g the removal efficiency increased in a slow rate and at 7g the removal efficiency is almost 100 percent.

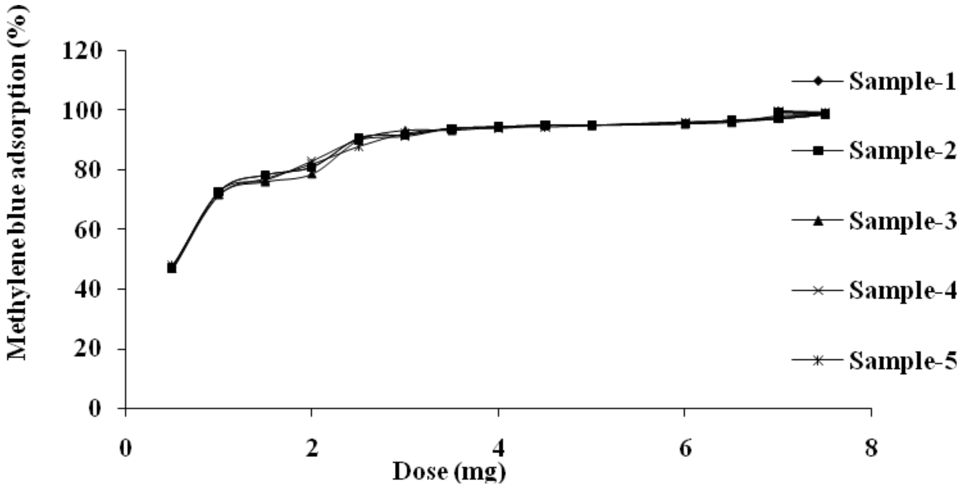

Figure 12: Effect of amount of dose on the efficiency of the Rice Husk Ash

Table 4: Dose effect on the efficiency of rice husk ash

Dose	Contact time (min.)	Temperature (^{0}C)	Raw color (Pt-Co)	Average % Efficiency
0.5	50	45	185	47.45946
1	50	45	185	72.54054
1.5	50	45	185	77.40541
2	50	45	185	81.2973
2.5	50	45	185	90.05405
3	50	45	185	92.21622
3.5	50	45	185	93.94595
4	50	45	185	94.48649
4.5	50	45	185	95.02703
5	50	45	185	95.13514
6	50	45	185	95.89189
6.5	50	45	185	96.54054
7	50	45	185	97.62162
7.5	50	45	185	99.02703
7	50	45	185	99.67568

3.4 Freundlich Adsorption Isotherm

The optimum treatment conditions were verified by the Freundlich adsorption isotherm. Adsorption of methylene blue by Rice Husk Ash is the physical adsorption of liquid-solid phase. The Freundlich isotherms represented by model x/m = K ($Ce^{1/n}$). Freundlich adsorption isotherm represents the relationship between the amounts of color adsorbed per unit mass of the adsorbent (x/m) band the concentration of the color remaining in the solution after adsorption is complete (Ce). K and n are constant representing the adsorption capacity and intensity of adsorption, respectively. The plot of log (x/m) versus log (Ce) was found to be linear. According to figure13 with R^2 value 0.916. The adsorption intensity (n) was found to be 1.84. The value of n was greater than 1 which signified the adsorption conditions are quite favorable.

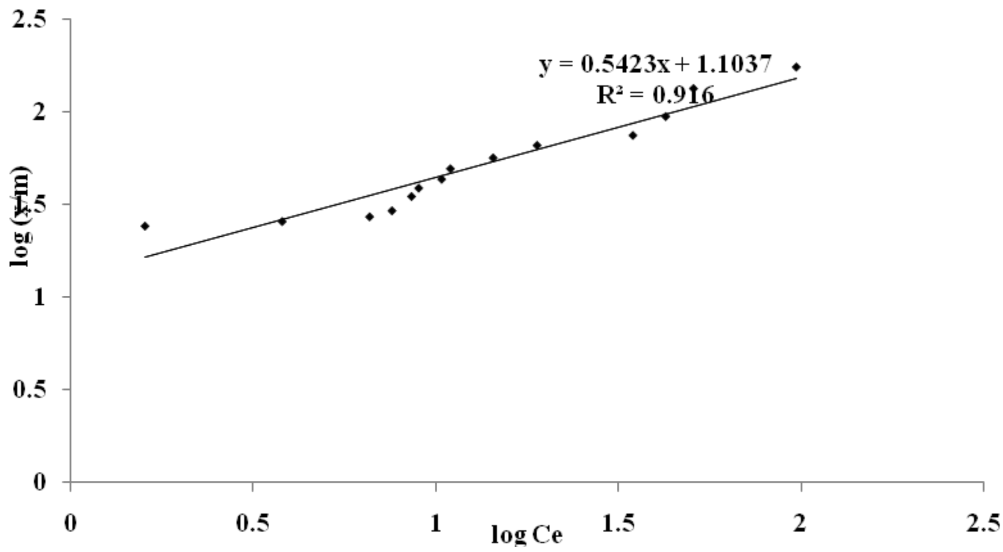

Figure 13: Freundlich Adsorption Isotherm

4. CONCLUSION

Methylene blue is a synthetic dye which is widely used by the textile industries and with presence of water makes an effluent of objectionable color for the regular photosynthesis. Available industrial activated carbons can only remove color from the Methylene blue solution with an efficiency of 35% (Mahmudur Rahman, 2011). This study has showed that Rice Husk Ash has the efficiency quite higher than the industrial carbon. But the efficiency of Rice Husk Ash is completely depends on the conditions that significantly influence the efficiency of adsorption that is temperature and contact time. At 45^0C temperature and 50 minutes contact time 3g Rice Husk Ash has the ability to remove the color from methylene blue solution with an efficiency that is almost 100 percent. Additionally, this research contributes to our understanding of how to sustainably manage wastes and byproduct materials and has the potential to provide several important environmental and economic benefits.

ACKNOWLEDGEMENTS

This research was funded by the Department of Civil Engineering, Khulna University of Engineering & Technology. The writers would like to thank Mr. Sayed Ahsan Ali, Asst. Technical Officer, and Mr. Sarder Shahidul Alam, Principal Laboratory Assistant, Environmental Engineering Laboratory, Department of Civil Engineering, Kuet.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

E-WASTE MANAGEMENT IN SYLHET CITY

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ABSTRACT

Bangladesh is a developing country where development activities are going on to boost up the advancement and progress in different sectors. To keep pace with development as the population growth is increasing day by day the solid waste is being increased as well. The increasing of e-wastes is getting upward like other solid wastes as the technological development is resulting in the increment of the waste. Sylhet City is one of the important cities in Bangladesh where school, college, universities, community institutions, commercial establishments, and offices are being increased for fulfillment of growing needs. Obviously e-waste is also being increased in these sectors. A study has been carried out on television, refrigerator, cell phone, computer in Sylhet by civil and environmental engineering department of Shahjalal University of Science and Technology which brought out the percentage of wastes of these e-products. In this study, it has been found out that 43% refrigerators, 25% televisions, 24% computers and 8% cell phones are generated in Sylhet city every year. The paper summarizes the findings of the study and ends with some guidelines towards appropriate e-waste management thereby leading to a pollution free environment.

1. INTRODUCTION

The technology boom of the 1990s gave us e-mail and e-commerce. The latest word to acquire the ‘e’ prefix has a more dubious distinction of waste. Electronic Waste which commonly referred to as ‘e-waste’ is the new by-product of the InfoTech society. And no, it does not describe junk e-mails, but a more real and physical waste in the form of old, discarded, end-of-life electronic. As electronics have become more reasonably priced, the volume of electronics in society has increased exponentially, as a larger section of the society is able to purchase them. As affordability has increased, so has the replacement rate, with the speed of technological innovation offering more functionalities, smaller sizes and newer designs. Given the time lag between the purchase of the product and its end-of-life means that products purchased five, seven or even ten years ago, are being discarded now. Already, the quantities of electronic waste discarded annually have been creating disposal problems for municipalities, especially in the developed world for some time. The technology boom of the 90s is being reflected in an e-waste boom now and even developing countries are waking up to the problem. E-waste management is more complex than household solid waste for municipal waste systems because of the high concentration of toxic compounds prevalent in these appliances. Their disposal requires special treatment to prevent the leakage and dissipation of toxins into the environment. The substantial expense involved as a result, is forcing governments to look for solutions to manage e-waste more effectively in cost and environmental impact**.**

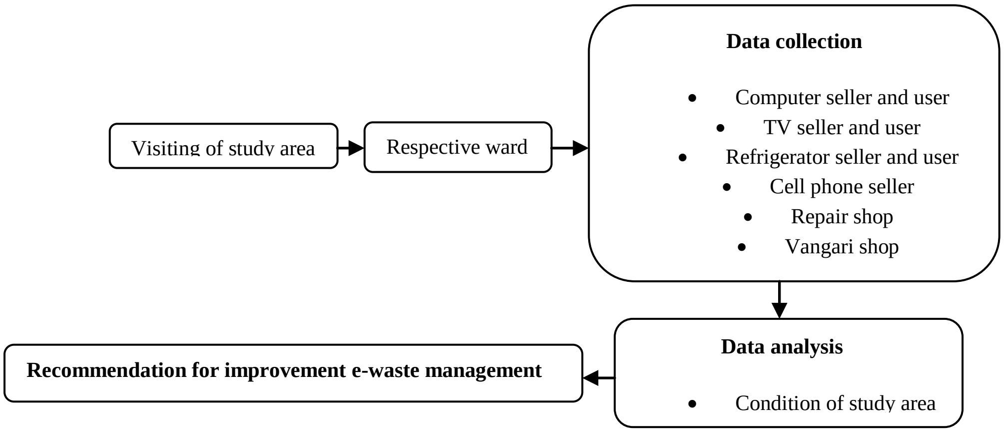
2. SOURCE OF E-WASTE:

The main generators of electrical and electronic waste are government institutions and the public and private sector, accounting for almost 70 percent of the total waste. The contribution from individual households, currently relatively small, is also likely to grow appreciably in future. Manufacturers of components and assemblers are another important source of e-waste generation in the country. However, it is extremely difficult to capture the exact quantity of waste generation by this group.

The import of e-waste, which is illegal, is another major source and preliminary estimates do point that the quantity being brought in is very significant. This takes place both in a legal as well as quasi-legal way, since e-waste is either misclassified as ‘metal scrap’ or imported as second hand or ‘end-of-life’ goods, which soon become waste**.**

3. METHODOLOGY

The amount of replaced or recycled or reused e-waste is obtained by field survey. Data's are collected from five vangari and repair shop in Sylhet City. The amount of reused and replaced and recycled product is obtained by interviewing.



4. RESULTS AND CALCULATION

General: In the study, after going through market best electronics company, based on product specific field to find out on which product of which company the methods would be applied.

- **Television Seller and User:** In this survey, the data was collected from different dealer shop. Here the amount of different type of TV selling was investigated. The maximum selling and using of TV brand is Sony 54% and 58% as shown in figure 1 and 2.

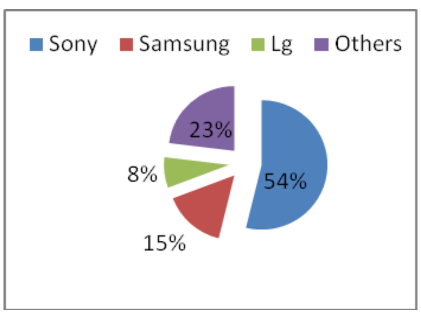


Figure 1: TV selling brand wise

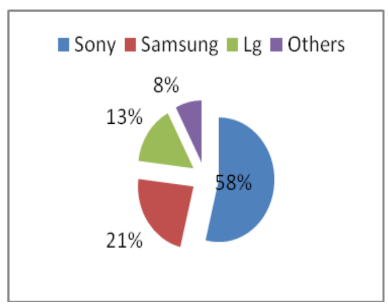


Figure 2: TV user brand wise

Cell Phone User: In this survey, the data was collected from different dealer shop. Here the amount of different type of cell phone selling was investigated. The maximum selling and using of cell phone brand is Nokia 55%, Symphony 20%, Maximum 15% and the other user is 10% as shown in figure 3.

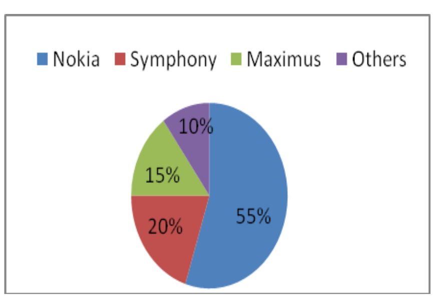


Figure 3: Percentage of cell phone user

Refrigerator Seller and User: In this survey, the data was collected from different dealer shop. In here the amount of different type of refrigerator selling was investigated. The maximum selling and using of selling refrigerator brand is Samsung 46% and 49% figures 4 and 5.

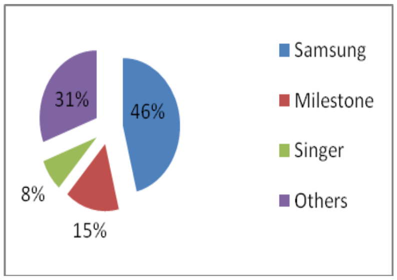


Figure 4: Refrigerator selling brand wise

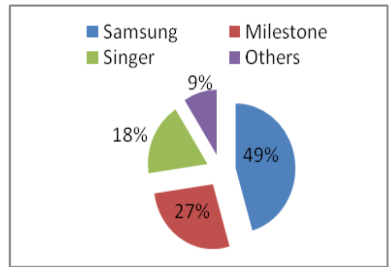


Figure 5: Refrigerator Uses brand wise

Computer Seller: In this survey, the data was collected from different dealer shop. Here the amount of different type of Computer parts selling, monthly repairing, sell to vangari shop and warranty was investigated. The maximum selling of computer parts is keyboard. The amount is 1363 Pcs. The minimum selling of computer parts is hard disk. The amount is 252 Pcs. Yearly selling of different computer parts is shown in figure 6.

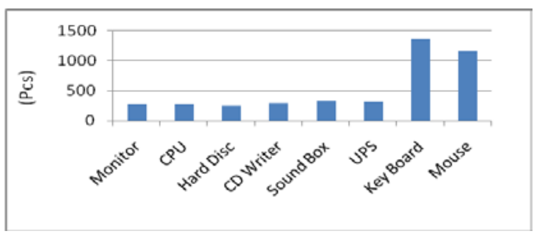


Figure 6: Yearly selling of computer parts

➢ **Repairing of E-Products:**

Computer parts repairing: Every year many pieces of computer parts come for repairing from users. The maximum repairing parts is processor; the amount of repairing processor of CPU is 96 Pcs. The minimum repairing parts is UPS, the amount is 16 Pcs. Key board and mouse are not repairable. Scenario of yearly repair of computer parts in repair shops is presented in figure 7.

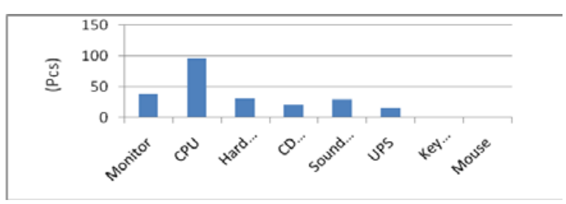


Figure 7: Scenario of computer parts repairing yearly

Television repairing: The study concentrated to about 20 dealer shop that repairs old televisions. Every year 39 Pcs TV come for repairing. The maximum repair is for picture tube, remote, antenna. Every year, 22 Pcs of television is damaged. Scenario of television repair and damage is cited in table 1 while percentile representation of repair and selling of television to vangari shop is presented in figure 8.

Table 1: Television repair and damage

No. of selling	Repair per year	Damage yearly	Warranty
275 Pcs	74 Pcs	22 Pcs	5 Years

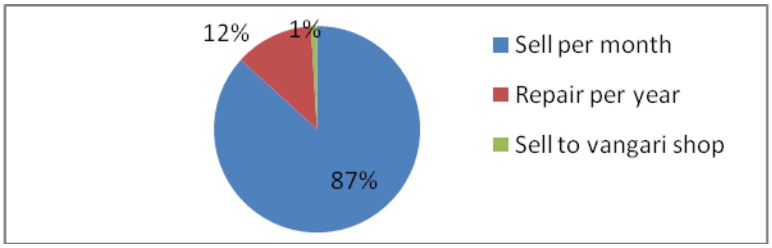


Figure 8: Television repair and selling into vangari shop

Refrigerator repairing: Every year 24 Pcs refrigerator come for repairing. The maximum repair is for compressor and casing. Yearly 19 Pcs are totally damaged. Scenario of refrigerator repair and damage is cited in table 2 while percentile representation of repair and selling of refrigerator to vangari shop is presented in figure 9 .

Table 2: Refrigerator repair and damage

No. of selling Pcs	Repair per year Pcs	Damage per year Pcs	Warranty Years
340	24	19	3

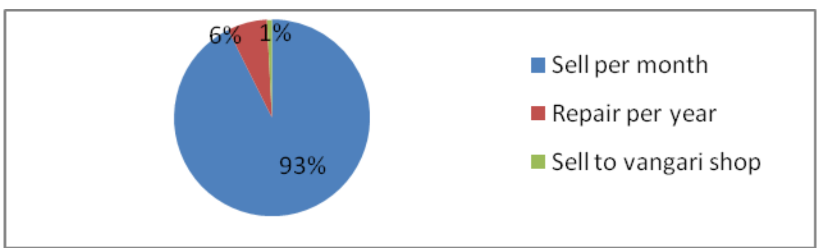


Figure 9: Amount of repairing and selling refrigerator

Cell phone repairing: Every year 2912 Pcs cell phone sell into the dealer shop, 450 Pcs are repairing monthly and 369 Pcs sell to vangari shop as shown in table 3. All phones warranty is one year.

Table 3: Cell phone sell, repair and sell to vangari shop

Monthly sell Pcs	Repair per month Pcs	Sell to vangari shop Pcs	Warranty (Years)
2912	450	369	1

- **Sell to Vangari Shop:**

Computer Waste: Every year maximum 23% waste comes from hard disk and minimum 8% waste come from sound box. The other waste is 13% CD writer, 18% UPS and monitor, 20% CPU waste come from computer as shown in figure 10 .

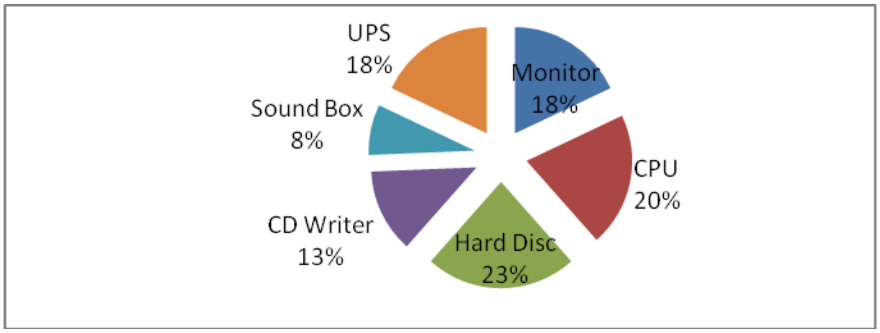


Figure 10: Percentage of computer parts waste

Calculation of Computer Waste:

Computer Waste

(Survey on 31 Sellers and Users)

For 15 inch Samsung CRT monitor

Weight = 12.3 Kg

UPS weight = 5 Kg

Let, the other computer parts weight = 5 Kg

No. of damage CRT monitor = 13 Pcs.

No. of damage UPS = 15 Pcs.

No. of other parts damage = averagely 12 Pcs.

$\therefore$ Total waste for Computer = $\{(13 \times 12.3) + (15 \times 5) + (12 \times 5)\}\ Kg$

$= 294.9\ Kg$

$\approx 295\ Kg$

Television Waste: In repair shop, 74 Pcs of TV comes for repair of which 22 Pcs of TV are totally damaged as shown in table 4. Every year 29% TV are repaired while 71% parts of damaged TV are sold in vangari shop as shown in figure 11.

Table 4: Amount of repair and damage of Television

Repair into shop Pcs	Totally damage Pcs
74	22

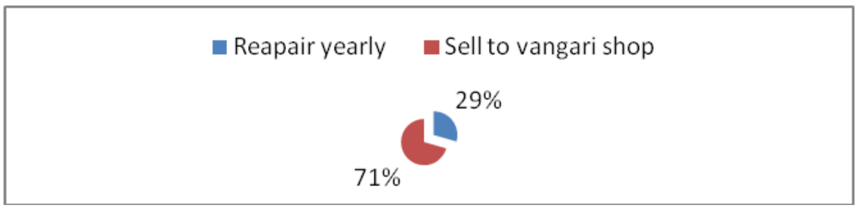


Figure11: Percentage of TV selling into vangari shop

Calculation of TV Waste:

Television Waste

For Sony 21 inch

Weight = 14 Kg

No. of damage TVs = 22 Pcs.

∴ Total TV waste per year = $(22\times14)\ Kg = 308\ Kg$

Refrigerator Waste:

In repair shop, 24 Pcs of refrigerators comes for repair of which 19pcs are totally damaged as shown in table 5. Every year 31% TV are repaired while 69% parts of damaged refrigerators are sold in vangari shop as shown in figure 12.

Table 5: Totally damage refrigerator into vangari shop

Repair into shop Pcs	Totally damage Pcs
24	19

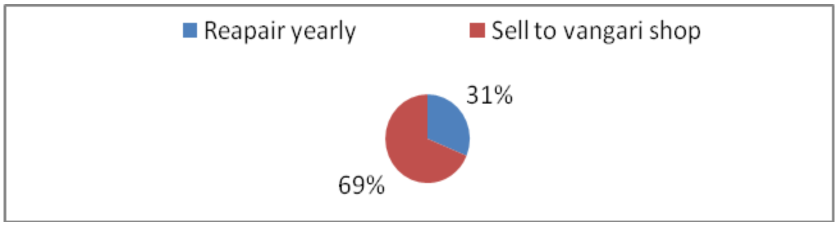


Figure 12: Percentage of refrigerator selling into vangari shop

Calculation of Refrigerator Waste:

Refrigerator Waste

For Samsung 8 sft

Weight = 28 Kg

No. of damage refrigerator = 19 Pcs

∴ Total refrigerator waste/year = $(19 \times 28) Kg = 532\ kg$

Cell phone Waste: In repair shop, 450 pcs of cell phone comes for repair of which 369 pcs are totally damaged as shown in table 6. Every month 12% cell phone is repaired while 78% parts of damaged cell phone are sold in vangari shop as shown in figure 13.

Table 6: Totally damage cell phone into vangari shop

Repair into shop Pcs	Totally damage Pcs
450	369

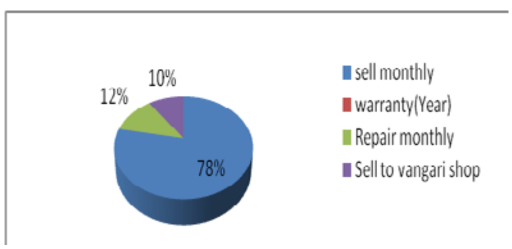


Figure 13: Percentage of cell phone selling into vangari shop

Calculation of Cell phone Waste:

Cell phone Waste

For Nokia 1200 model

Weight = 0.25 Kg

No. of damage cell phone = 369 Pcs.

∴ Total waste for cell phone/year = (369×0.25) Kg = 92.25 Kg

Total Waste Calculation:

Total waste = (295 + 308 + 532 + 92.25) kg = 1227.25 kg $\approx 1228kg$

Percentage of E-Waste in Sylhet City: From calculation the maximum 43% e-waste of Sylhet city come from refrigerator. All other e-waste like 25% TV, 24% computer, 8% cell phone waste generate in Sylhet city every year as shown in figure 14.

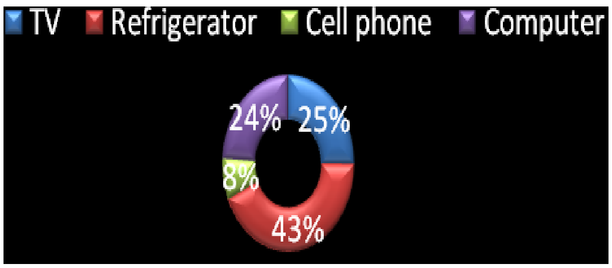


Figure 14: Presenting E-waste Condition in Sylhet City

Adverse Health Effect of E-Product:

Graphical pie presentation in figure shows how e-product affects on users eyes and ears. There are 64% people facing eye and hearing problem. 12% people are not facing any health effect and 24% peoples are not being understood with the problem at all as shown in figure 15.

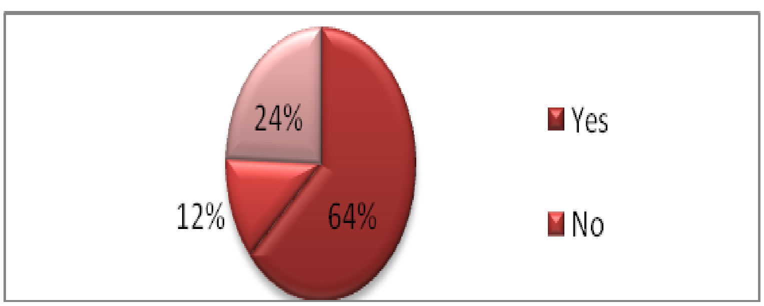


Figure 15: Health effect of e-products

Proper E-Waste Management System:

Based on the study result whole e-waste management process in Sylhet City is summarized in the flow diagram of figure 16:

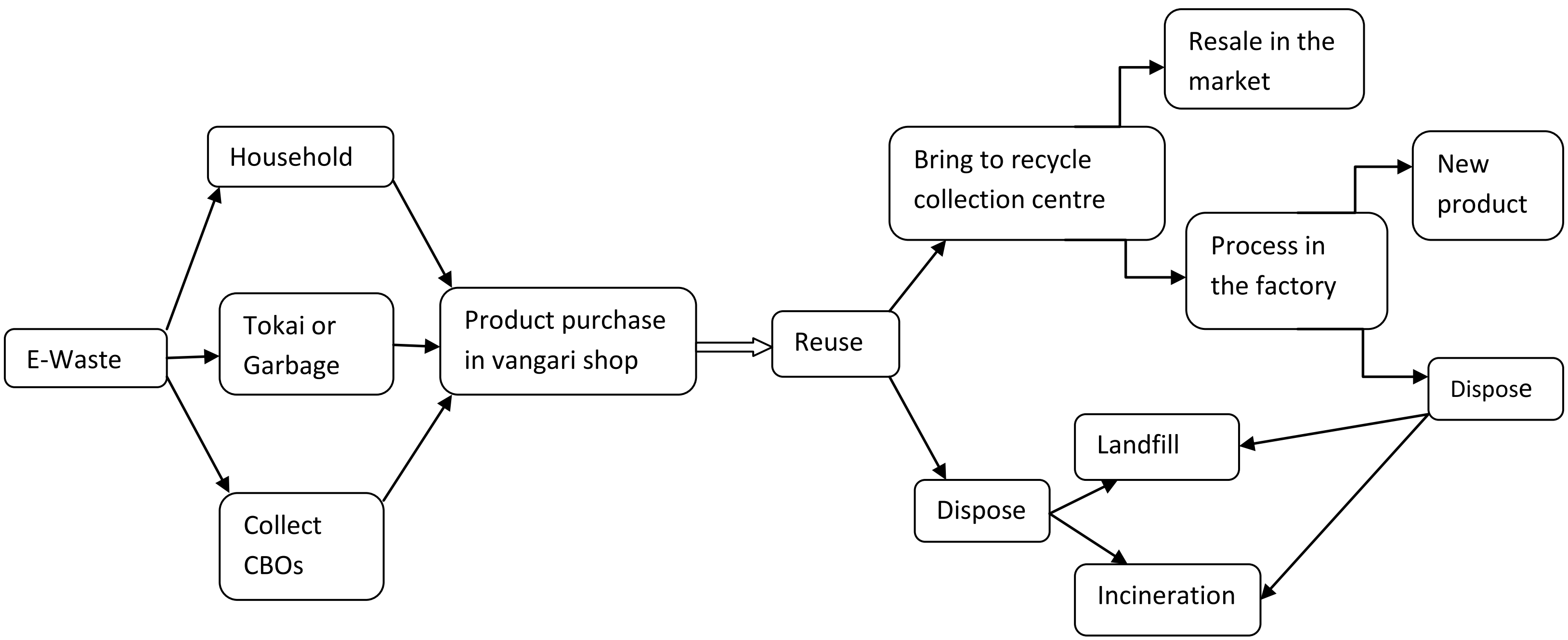


Figure 16: Flow diagram for proper e-waste management

5. CONCLUSION

E-waste management is one of the important concerns for proper urbanization and better environment. The main source of e-waste in Sylhet city is increasing exponentially. This is due to lack of public awareness, public behavior, improper urbanization etc. This is to ensure that the environmental objectives are met and all activities relating to the implementation of the study are carried out in an environmentally sustainable manner.

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DEVELOPMENT OF A SIMPLE CERAMIC MEMBRANE FILTER FOR WASTEWATER TREATMENT

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ABSTRACT

In this study a low cost and simple type ceramic membrane was innovated for concurrent wastewater treatment and reuse. As the processes are still costly in terms of price of membrane, maintenance cost and energy consumption. Locally available and cheap materials (clay soil and rice bran) were used as ingredients to decline the membrane cost in this study. Mixing ratio of the ingredients were 80% of clay soil and 20 % of rice bran on weight basis. The ceramic membrane was submerged inside reactor to formulate as ceramic membrane filter. For this research study 3 numbers of lab-scale Ceramic Membrane Reactors were set up. The average Turbidity Removal efficiency of all Reactors was 95.34%. The result shows that about 88.45% Color removal was achieved by the system. The average SS removal efficiency was 95.25% by all Reactors which demonstrates that the removal of SS was very efficient by Ceramic Filter. About 81.55% COD removal was achieved by Ceramic Membrane Filter. The Ceramic filter has great potential in removing biodegrading organic pollutants from wastewater. As the Ceramic Membrane was made by locally available materials the technology was inexpensive. Therefore, the technology is suitable and can be adapted in developing countries for wastewater treatment and reuse.

Keywords: *Ceramic Membrane, Filter, Wastewater, Treatment*

1. INTRODUCTION

The demand for water is increasing with the increasing industrialization, urbanization and the diversification of lifestyles (Kim and Cho 1993). In addition to this, there have been problems relating to the increased amount of wastewater also, since aside from a minor quantity, most consumed water is transformed into wastewater. Wastewater is water that has been used and must be treated before it is released into another water body, so that it does not cause further pollution of water sources (SDWF Report 2007) or any water that is no longer wanted, as no further benefits can be derived out of it, is termed as wastewater (Hasan and Nakajima 2010). Generally there are four types of wastewater such as Domestic wastewater, Municipal wastewater, Industrial wastewater and Storm wastewater. Domestic wastewater can be divided into Black waster and Grey water. Generally greywater is divided in four greywater categories based on its origin: bathroom, laundry, kitchen and mixed origin (Morel 2005). Greywater includes all household wastewater except toilet waste. It can be a valuable water resource, and an increasing number of householders are recycling greywater for a variety of purposes. However, care must be taken with this practice as it can carry health and environmental risks (Environmental Health Unit Report 2003). In Bangladesh, the greywater situation in terms of treatment and recycling is the most horrible. Most of the people think that greywater is not so much polluted like blackwater and so not only rural parts but also in urban area there is no treatment options for greywater. And greywater recycling is still unbelievable. Generally different scenarios are prevailed in urban and rural areas of Bangladesh about greywater (Hasan and Nakajima 2010). Membrane bioreactor (MBR) technology is characterized as a combination of biological wastewater treatment (WWT) and membrane separation, by which biomass can retained in the system without conventional gravity sedimentation (Itokawa 2009). The Membrane Bioreactor is a simple, but very effective combination of the activated sludge treatment process and the membrane filtration process (Operator Notebook Report 2001). The MBR process is an emerging advanced wastewater treatment technology that has been successfully applied at an ever increasing number of locations around the world (Chapman et al. 2003). Membrane bioreactor (MBR) is an improvement of the 100-year old CASP (Conventional Activated Sludge Processes), where the traditional secondary clarifier is replaced by a membrane unit for the separation of treated water from the mixed solution in the bioreactor (Xing et al. 2000). The major goal of this study is to carry out initiative for the improvement of the further treatment of wastewater in developing countries by using an innovative, low cost and simple technology. For achieving the goal following successive objectives were fixed

1) To apply Ceramic Membrane in a Reactor for wastewater treatment. 2) To determine the quality of effluent from Ceramic Filter. 3) To observe the efficiency in flux of the developed Ceramic Filter. 4) To monitor the overall operation and maintenance performances of the Reactor.

2. METHODOLOGY

2.1 Manufacture of Ceramic Membrane Module

For making the Ceramic Membrane Reactor, cheap, easily and locally available materials (rice bran and clay soil) were chosen for wastewater treatment. About 80% clay soil and 20% rice bran was used for manufacture of ceramic membrane. Locally collected clay soil samples were dried and grind by hammer. The dried and grind clay soil was sieved by 0.5 mm mesh. The rice burn was also dried and sieved by 1 mm mesh. Mixing ratio of the ingredients were 80% of clay soil and 20 % of rice bran on weight basis. This mixing proportion of the materials was selected by quantifying pore volume, pore size, compressive strength and filtration efficiency of several ceramic bars prepared with different ratios of the ingredients. Details about the selection of ingredients proportion were described in the next section. To make the dough manually, dried ingredients were mixed homogeneously and then sufficient amount of water was used. Water of 400-500 ml was used with the dry homogeneous-mixed ingredients of 800 gm for making one membrane module. In the end, a hollow cylindrical shape was manually made with the dough as 10 cm height with 10 cm outer diameter and 6 cm inner diameter with one side opened (Figure 1). To make preferred shape, a wooden dice and PVC pipe of 10cm ht with 10cm outer diameter cut vertically in symmetrical were used. The membrane was then kept for 24 hrs for natural dry at room temperature, then oven dried at 105°C for 24 hrs and finally burnt in a muffle furnace in the laboratory. The temperature in muffle furnace was increased from room temperature to 900°C and kept this temperature for 2 hrs. After that Temperature was decreased gradually as the firing was terminated. After the termination of firing the membrane modules were kept into the kiln for overnight and take out from the kiln in next morning.

2.2 Experimental Set-up

For this research study 3 numbers of lab-scale Ceramic Membrane Reactors were set up as named R-1, R-2, and R-3. All the three Reactors were as cylindrical column shaped with 14 cm inner diameter and made by PVC pipe. The inner diameter of all the Reactors was same as 14 cm and heights was same as about 140 cm. In each reactor, Membrane Module was directly submerged inside the Reactor. The Membrane was placed on a PVC plate and was made water tight by using Silica glue to prevent infiltration of water, tilting and floating. The PVC plate with Membrane was then placed at the end of PVC pipe. The Reactors were filled with raw wastewater. The wastewater used in this study was actual wastewater rather than synthetic Wastewater. The wastewater was collected from a drain near Dr. M. A Rashid Hall in KUET. The raw wastewater used as feed composed of effluent water from septic tank and bathing water from the residential Hall. The concentration of various ingredients of raw wastewater was different. The intermittent flow of influent was applied in this study. The objective was to measure influent and effluent characteristics of the raw wastewater. For this purpose various water quality parameter was determined to measure the performance of Ceramic Membrane Reactor. The wastewater was fed into each Reactor from the feed tank by peristaltic pump. The water was aerated from the beneath of membrane module through a diffuser, so that rising air bubbles can provide the membrane surface with more shear stress, which is effective for removing attached sludge out of membrane, and to mix the mixed liquor in the Reactor and also to maintain an aerobic environment for the normal growth of activated sludge. Intermittent aeration system was set up as 4 hr aeration per day by using the blower. The three Reactors R-1, R-2 and R-3 were under aerobic condition. Permeate from the Reactors was collected during the aeration period through outlet by gravitational pressure. The water was sampled every three days per week. The parameters analyzed were the level of Turbidity, Color, TS, TDS, SS, Fe and COD in the Reactor and Flow rate. Figure 3 shows the schematic view of the system, displays photographs of the membrane media position in the Reactor and the CMR under continuous operation in the laboratory. The systems were monitored by measurement of permeate flux, pH and DO. Sludge retention time (SRT) was infinitive as there was no sludge wastage except for sampling during the operation. The Run time of Reactor-1, Reactor-2 and Reactor-3 were 215 days. Within this time period Reactor-1 and Reactor-2 were clogged after 129 days from 29-12-10 to 04-05-11. Reactor-3 was clogged after 65 days and after cleaning it was clogged again at 129 days. A cake layer formation was found on all Ceramic Membrane surfaces because of the deposition of the floe sludge. The membrane was then cleaned physically by removing the accumulated sludge with the help of water, knife and soft spongy brush. There was no need of chemical washing or change of the membrane.

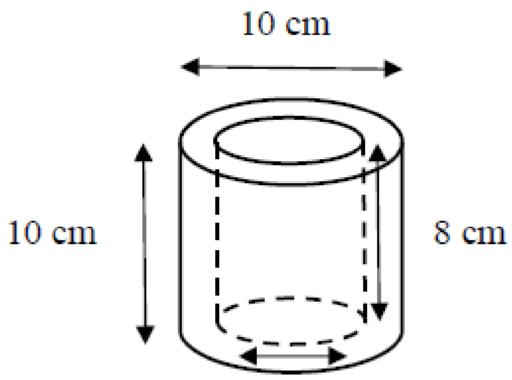

Figure 1: Dimension of Ceramic Membrane

Figure 2: Ceramic Membrane after burn

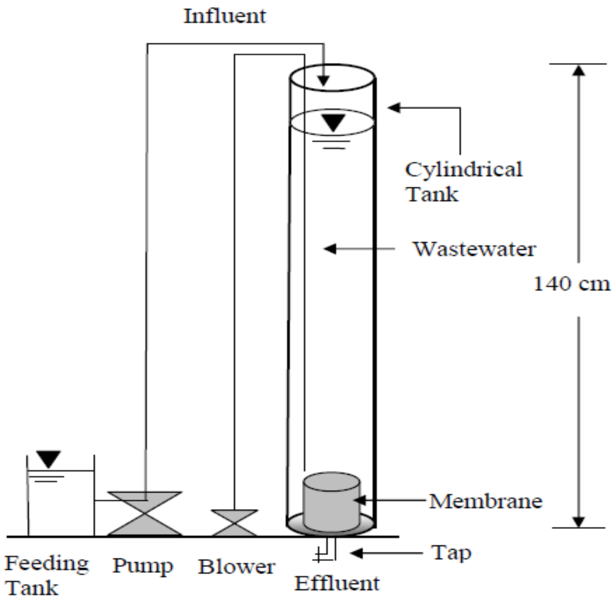

Figure 3: Schematic representation of the system

3. RESULTS AND DISCUSSION

3.1 Operating Parameters (pH & DO)

From laboratory test it was found that the average DO in influent wastewater was 3.47 mg/l. The average effluent DO by Reactor-1, Ractor-2 and Reactor-3 were 5.42 mg/l, 5.42 mg/l and 5.47 mg/l, respectively. During the accumulation of effluent water into the bucket it has been found that the effluent water was exposed to air and the value of DO increase because of aeration. The standard value of DO is 4.5 to 8 for discharging the wastewater into land water, public sewer and on irrigated land. The average effluent DO by all Reactors was 5.44 mg/l which is within the standard value. In water supply, pH is very important as the organism involved in treatment processes operate within a certain pH range. The pH value in influent was within the range of 6.68 to 8.80 and the average pH value in influent was 7.97. The average pH value in Reactor-1, Reactor-2 and Reactor-3 were 7.99, 7.92 and 7.91 respectively. The pH of all Reactors was stable within the range of 7-8 as shown in Figure: 4.

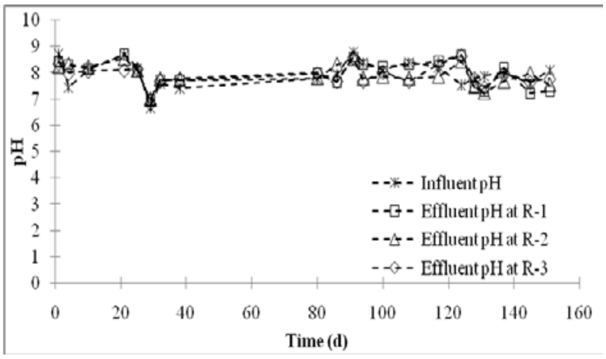

Figure 4: Range of pH in CMBR

3.2 Turbidity Removal

Turbidity is one of the basic concerns in the treatment of any types of wastewater. From laboratory test it was found that average Turbidity in influent was 602.85 NTU with minimum and maximum value of 139 NTU and 3930 NTU respectively. After treatment the effluent showed very low Turbidity as the average Turbidity of all Reactors was 13.55 NTU. Significant Turbidity removal was achieved by Ceramic Filter. Turbidity removal efficiency of Reactor-1, Reactor-2 and Reactor-3 were 95.31%, 95.49% and 95.21% respectively. The average Turbidity Removal efficiency of all Reactors was 95.34%. It is worth to state that an excellent performance for Turbidity removal was observed for all Runs.

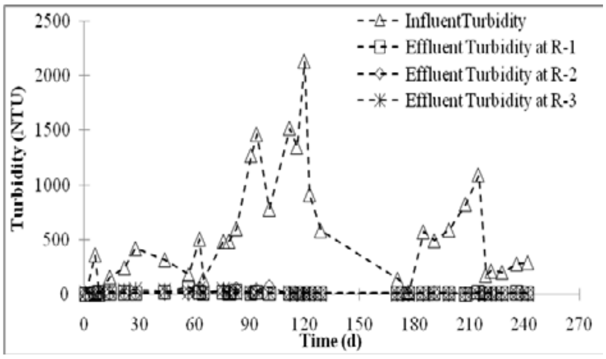

Figure 5: Turbidity in Influent and Effluent

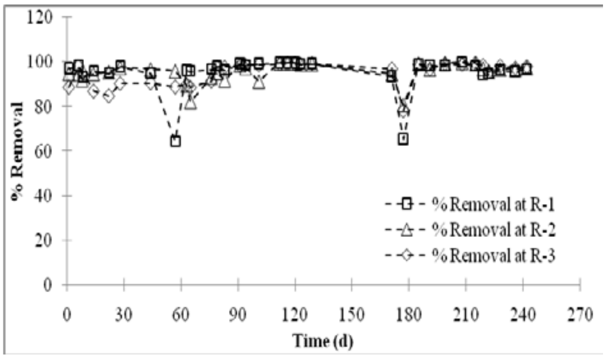

Figure 6: Turbidity Removal Efficiency

3.3 Color Removal

From laboratory test it was found that the average Color in influent was 2294 Pt-Co with minimum and maximum value of 148 Pt-Co and 8130 Pt-Co respectively. Color reduced significantly by Ceramic Filter as average effluent Color of Reactor-1, Reactor-2 and Reactor-3 among 32 samples were 138.79 Pt-Co, 184.47 Pt-Co and 142.91 Pt-Co respectively. Significant Color removal efficiency was achieved by Ceramic Filter. Removal efficiency of Reactor-1, Reactor-2 and Reactor-3 were 90.10%, 87.48% and 87.78% respectively. The result emphasized that about 88.45% Color removal was achieved by the system.

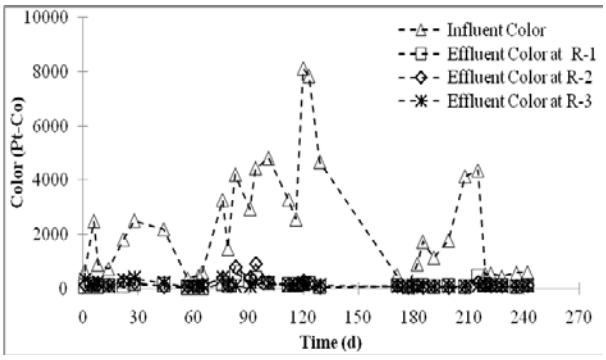

Figure 7: Color in Influent and Effluent

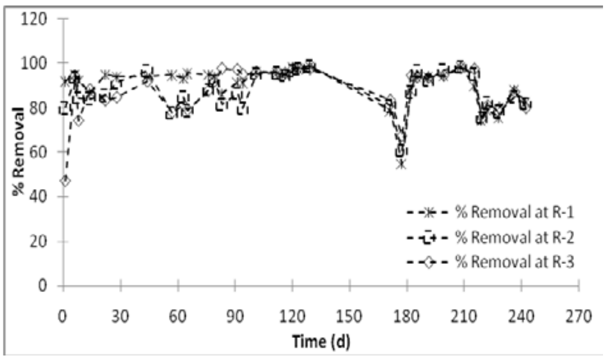

Figure 8: Color Removal Efficiency

3.4 COD Removal

Disposal of wastewater containing high COD to receiving water bodies might cause oxygen depletion that will have harmful effects to living resources like fishes, or eventually make the environment anaerobic. Therefore, its removal is given more focus in any wastewater treatment facility. Through the study it was found that the average COD of influent wastewater was 572.91 mg/l with minimum and maximum value of 92 mg/l and 1568 mg/l respectively. After treatment by Ceramic Filter the effluent showed very low COD as the average value of COD in effluent of Reactor-1, Reactor-2 and Reactor-3 were 68.40 mg/l, 84.91 mg/l and 78.36 mg/l respectively during the operation period. The average effluent COD value by all Reactors was 77.22 mg/l.

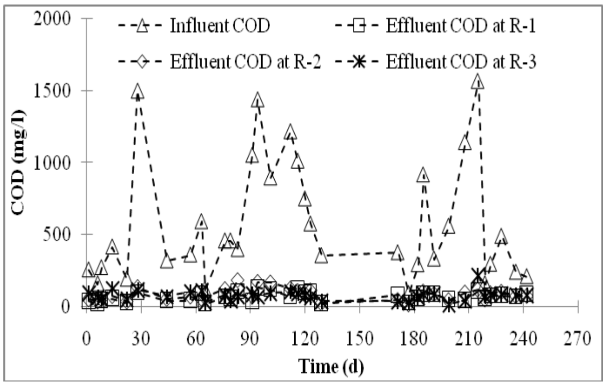

Figure 9: COD in Influent and Effluent

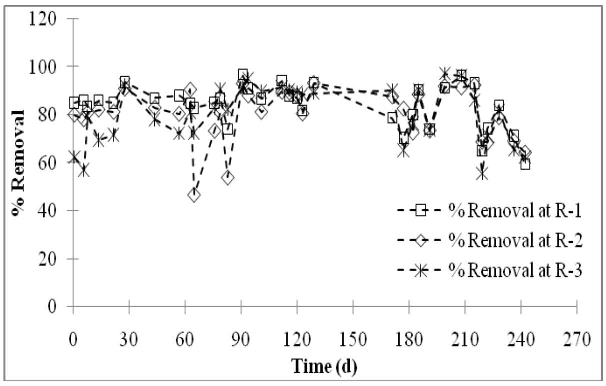

Figure 10: COD Removal Efficiency

3.5 TS, TDS, SS Removal:

The TS, TDS and SS are important concern in the treatment of wastewater. Its disposal causes clogging of sewerage system and also its disposal to water bodies have harmful effects to living resources. It has been found that the average TS in influent was 3329 mg/l with minimum and maximum value of 1790 mg/l and 5189 mg/l, respectively. The Average TS in effluent of Reactor-1, Reactor-2 and Reactor-3 were 2040 mg/l, 2094 mg/l and 2123 mg/l, respectively. The standard value of TS and TDS for discharging the wastewater into land water, public sewer and on irrigated land is 2100 mg/l. The average TS and TDS in effluent of all Reactors were 2085 mg/l and 2058 mg/l, respectively which are below the standard value. The average value of SS in influent was 1011.34 mg/l with minimum and maximum value of 28 mg/l and 3080 mg/l respectively. After treatment remarkable SS removal was achieved by Ceramic Filter as the average effluent SS of Reactor-1, Reactor-2 and Reactor-3 were 22.95 mg/l, 32.79 mg/l and 28.03 mg/l respectively. TS removal efficiency of Reactor-1, Reactor-2 and Reactor-3 were 37.16%, 35.25% and 35.09% respectively and TDS removal efficiency were 12.81%, 9.77% and 9.73% respectively. The data also reveals that the SS removal efficiency of Reactor-1, Reactor-2 and Reactor-3 were 95.61%, 94.59% and 95.56% respectively. The average SS removal efficiency was 95.25% by all Reactors.

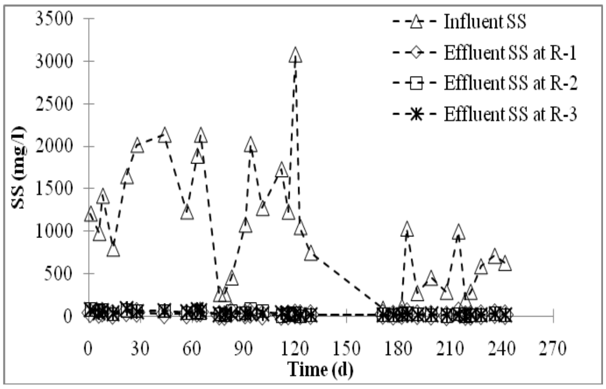

Figure 11: SS in Influent and Effluent

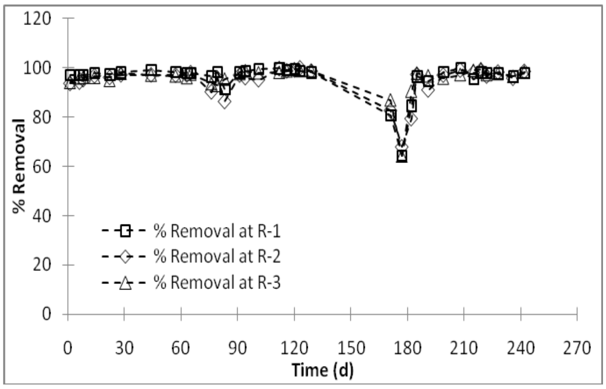

Figure 12: SS Removal Efficiency

3.6 Fe Removal

In this study, it was found that significant Fe removal was achieved by Ceramic Filter. The average concentration of Iron in influent wastewater was 0.2 mg/l which was very low because the wastewater used as influent in Reactor was surface water. As the concentration of Iron in Influent water was very low, the data collection was carried out up to 129 days. The average concentration of Iron (Fe) in effluent water was 0.02 mg/l with minimum and maximum value of 0.0 mg/l and 0.06 mg/l respectively. The concentration of Fe in influent and effluent water is shown in Figure 13.The Iron removal efficiency of Reactor-1, Reactor-2 and Reactor-3 were 93.99%, 88.98% and 95.1% respectively and the average removal efficiency of all Reactors was 92.69%. Figure 14 shows the Fe removal efficiency of all Reactors

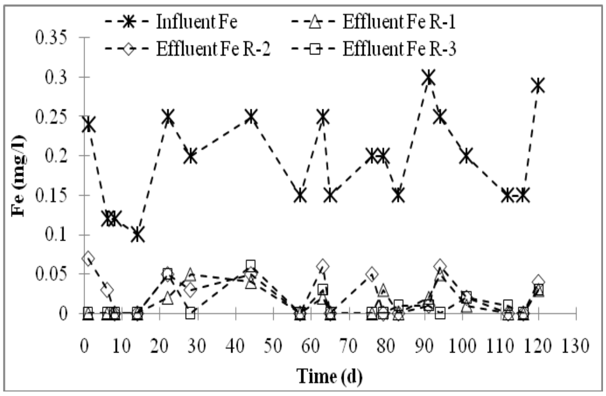

Figure 13: Concentration of Iron in Influent and Effluent

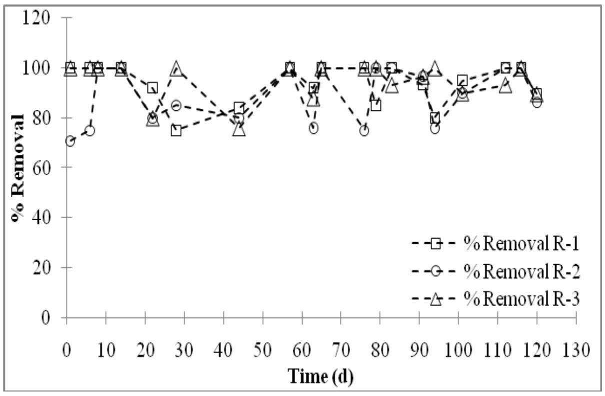

Figure 14: Iron Removal Efficiency

3.7 Flux Performance

The fluxes of different Reactors were determined by simple gravitational filtration method. Initial fluxes at R-1, R-2 and R-3 Reactor were found 100 ml/min 90 ml/min and 39 ml/min respectively. The Run time of Reactor-1 Reactor-2 and Reactor-3 were 242 days from 29-12-10 to 24-08-11. Within this time period Reactor-1 and Reactor-2 were clogged after 129 days. But Reactor-3 was clogged after 65 days and after cleaning it was clogged again at 129 days. Figure 15 demonstrates the flux details of all Reactors.

As can be seen, Flux decreased gradually throughout the time period. The Flux of Reactor-1, Reactor-2 and Reactor-3 before clogging were 0.5 ml/min, 0.25 ml/min and 0.1ml/min respectively. It is noticeable that the membrane was not totally clogged rather its flow rate was very low. As blockage of the pores of the membranes by solid particles was removed by cleaning, sharp rise of the Flow rate occurred after 65 days and 171 days as shown in Figure 15. A sharp decrease of the flux can occur due to non uniform air distribution into the membrane tank caused the total blockage of the gap between the membranes by solid particles (TS,TDS,SS), leading to reduction of the membrane flow capacity.

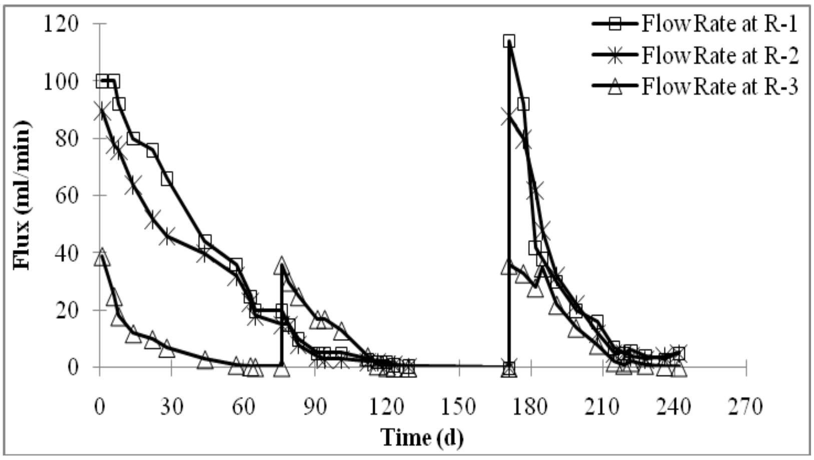

Figure 15: Variation of Flux

3.8 BOD Removal

From previous study it has been found that BOD concentration of synthetic wastewater used during the operation period was 5000 mg/l but effluent BOD was lower than 5 mg/l which meant, about 99.9% of BOD was removed (Hasan and Nakajima 2010). In this study actual wastewater was used as feed and it has been found that among six samples, the average BOD concentration of raw wastewater used during the operation period was 5.33 mg/l but average effluent BOD was about 0.96 mg/l which means that about 82% of BOD removal was achieved by Ceramic Filter. Overall, it can be concluded that the Ceramic Filter has great potential in removing biodegrading organic pollutants from wastewater.

3.9 Other Findings

The total amount of wastewater used for all Reactors is about 550.8 liters (183.6 Liters for each Reactor). After 129 days of operation about 113 liters of wastewater was used for each Reactor. After 242 days of operation about 70.2 liters of water was used for each Reactor.

Table 1: Overall condition of Reactor-3 before Clogging

Days	No of Sample	Water Used (Liters)	Total Solids in Influent (Kg)
At 65 days	10	54	0.21
At 129 days	11	60	0.20
At 242 days	13	70	0.21

From the above Table it has been found that the amount of Total solids was about 200 gm when the Reactor-3 was clogged. So clogging depends on the amount of Total Solids used as feed.
By considering similar situation of Reactor-1 and Reactor-2 before and after Clogging, it has been found that Reactor-1 and Reactor-2 was clogged and cleaned at 129 days. After 242 days when all Reactors were cleaned again then the Flow rate of R-1 and R-2 were 5 ml/min and 5 ml/min respectively. Table 4-9: shows the situation of Reactor-1 and Reactor-2 at similar flow rate before and after clogging.

Table 2: Overall condition of Reactor-1 and Reactor-3 at similar flow rate (about 5 ml/min)

No of days	Total Sample	Flow rate at R-1 (ml/min)	Flow Rate at R-2 (ml/min)	Water Used (L)	TS in Influent (Kg)
Before Clogging at 91 days	14	5	4	75.6	0.25
After clogging at 242 days	13	5	5	70.2	0.2

From the study it has been found that the flow rate of Reactor-1 and Reactor-2 at 91 days was about 5 ml/min (before clogging at 129 days) and the flow rate of Reactor-1 and Reactor-2 at 242 days was also about 5 ml/min (after clogging at 129 days). At 91 days the Total Solids in Influent used as feed was 250 gm and at 242 days the Total Solids in Influent used as feed was 200 gm. So the amount of Total Solids was very similar when the flow rate was about 5 ml/min for both Reactors before and after clogging. The total amount of water used as feed before and after clogging were 75 liter and 70.2 liter respectively (when the flow rate was about 5 ml/min). So Flow rate and Clogging of Ceramic Filter depend on the amount of Total Solids used as feed.

4. CONCLUSIONS

A low cost and simple type ceramic membrane was innovated for concurrent wastewater treatment and reuse especially for wastewater treatment in this study. This simple type Ceramic Filter process was investigated from the laboratory experiments and it can be concluded that: Activated sludge can be easily separated through this Technology. Through this process sufficient amount of flux was obtained for the case of wastewater treatment.
Turbidity removed significantly by Ceramic Membrane Filter. The average Turbidity Removal efficiency of all Reactors was 95.34%. It is worth to state that an excellent performance for Turbidity removal was observed for all Runs. The result shows that about 88.45% of Color removal was achieved by the system. Turbidity and Color removal efficiency was excellent that’s why the filtered water is more acceptable by the people as the aesthetic appearance of the water is good. Removal efficiency of organic matters in terms of COD was very much satisfactory in the case of wastewater treatment. About 81.55% of COD removal was achieved by Ceramic Membrane Filter. From the results, it can be concluded that the Ceramic Membrane Filter has great potential in

removing biodegrading organic pollutants from wastewater. This indicates that the Ceramic Membrane is able to retain the organic content from wastewater. The average SS removal efficiency was 95.25% by all Reactors which demonstrates that the removal of SS was very efficient by Ceramic Filter. The Total Solid removal was not satisfactory by the system as the average Total Solid Removal efficiency was 35.83%. Iron removed significantly by the system. The average Iron removal efficiency of all Reactors was 92.69%. The WHO guideline suggests that the concentration of Iron should be less than 0.3 mg/l in drinking water. The average concentration of Iron in effluent water was found 0.02 mg/l by Ceramic Filter which is well below even drinking water standard. So the Ceramic Membrane Filter is very efficient to remove Iron from wastewater. Physical cleaning of the membrane was much simple and it was easy to remove the cake layer to reclaim the membrane. The run time of Ceramic Filter was very good. The longer and maximum run time of Ceramic Filter was 129 days. The quality of effluent water was excellent as the effluent water was clear colored and odor- free. It was found that high removal efficiency of organic content was obtained that could be made it suitable for wastewater reuse. As the Ceramic Membrane was made by locally available materials the technology was inexpensive. Therefore the technology is suitable and can be adapted in developing countries for wastewater treatment and reuse. The total cost for setup of Ceramic Membrane Reactor in the laboratory was about 16500 Taka only.

ACKNOWLEDGEMENTS

The author expresses his praise to Almighty God, who helped to complete paper successfully and highly grateful to my teachers and friends for their sincere cooperation and providing valuable information.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

WATER QUALITY MONITORING IN THE VICINITY OF LEATHER INDUSTRIES IN BANGLADESH: A CASE STUDY

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ABSTRACT

Leather industry plays an important role in Bangladeshi Economy due to its large potential for employment, growth and export. At the same time, it poses serious environmental threats by discharging liquid effluents and solid wastes directly into surrounding low lying areas without proper treatment. In a study of 2001, relatively high concentrations of Chromium, Lead and Sulfide was identified in the ground water in Hazaribagh area but in another study of 2008 there was no sign of any contamination. This paper describes a recent survey conducted in 2010 whose major objective was to monitor the water quality of selected DWASA deep tubewells of Hazaribagh area for the presence of contamination by tannery wastewater and to identify the trend of change of water quality. A high concentration of chemical oxygen demand (COD) (from 7.9 to 8.5 mg/l) was observed in the month of April in all wells of Hazaribagh. Sulfide concentrations in some of the samples were found to be excessively high, varying from 0.0-10.0 µg/l. Relatively higher concentration of iron (0.0 to 1.2 mg/l) and ammonia (0.0 to 1.272 mg/l) were detected in some water samples. EC concentration was higher for all the wells (300 µg/cm to 500 µg/cm). The groundwater resources at Hazaribagh area that have been sampled in this investigation mostly satisfy the WHO and Bangladesh drinking water standard for the parameters tested in the study. But some elevated levels of COD, sulfide and lead have been detected which may be a matter of concern.

Keywords: *Tannery effluent, water quality, groundwater, absorption, contamination.*

1. INTRODUCTION

Hazaribagh is a densely populated area of Dhaka city where the main leather tanning industries of Bangladesh are situated. There are about 200 leather processing industries in Hazaribagh area. In a tannery, different operations require large volume of water for the purpose of washing in each step. The average volume of water required for the processing of one ton of hide is about 50 m^3 (WHO, 1982).Waste water from tannery industry contains chromium, dissolved lime, hydrogen sulfide, dyes, oils, organic matter and suspended solids. The industries are discharging their solid wastes and liquid effluents containing rotten flesh, fat, blood and skin, toxic chemicals etc. directly into low lying areas and water bodies without proper treatment. So, there is an obvious risk of percolation of leachate, which may affect the ground water quality.

In the flood plain between the Buriganga river and Hazaribagh area, a dike was constructed in 1988. Between the dike and Hazaribagh area there are stagnant ponds. The tannery wastewater is discharged into the ponds without proper treatment. Finally this wastewater is discharged into Buriganga river.(Ganesh, Ali). The tannery wastewater retains in the stagnant ponds for prolonged periods of time. This allows the dissolved constituents from the wastewater to percolate into the subsurface. This can contaminate the precious groundwater resource. Thus water quality of the rivers deteriorates increasingly and these pose a significant threat to our limited water supply by changing taste and odor, growth of aquatic weeds, aquatic life and wild life.

In Hazaribagh area, groundwater is extracted through a network of 8 deep tube wells and distributed to the dwellers through distribution system. Any contamination of ground water would endanger not only the water supply network of this particular area but also the entire water supply system in Dhaka city. Groundwater samples were collected from 7 deep tubewells of DWASA Zone-2 (Hazaribagh area) and have been analyzed for a wide range of parameters including chromium, iron, lead, manganese, pH, chloride, sulfide, ammonia, COD and total dissolved solids. This study aims to analyze the present status of surface water pollution and a statistical comparison of the quality of water in different seasons at different locations of Hazaribagh area.

2. METHODOLOGY

The area of Hazaribagh is situated in the south west side of Dhaka City. It is a densely populated area of Dhaka, where about 185 leather processing industries are located in a congested area of only 70 acres. The tannery industries are operating and discharging solid and liquid wastes directly to the low-lying areas, river and natural canals without proper treatment. Tanneries in the Hazaribagh area discharge some 21,600 cubic meters of liquid wastes everyday. The contaminated low-lying land area is about 25 hectors and this zone is categorized as Red Zone according to Department of Environment (DoE), Bangladesh. Around 20,000 peoples are presently living in the slums in this area, under extremely densely populated and unhygienic conditions.

2.1 Study Area

There are about 40 deep tubewells in DWASA Zone 2. Among them 8 is located at Hazaribagh area. In this study, samples were collected from 7 stations.

Table 1: DWASA Zone-2 pump locations

Well No.	Pump Location
1	Shishu Park Pump
2	BDR No.2 Pump (near BDR Gate No.1)
3	Hazaribagh -3 (near Phoenix Textile)
4	Hazaribagh -4 (Baddanagar Water Tank)
5	Hazaribagh -5 (near Leather Technology College)
6	Gajamahal Pump
7	Kalunagar Pump

2.2 Collection and Analysis of Samples

Groundwater samples were collected from 7 DTW stations during the period of January 2010 to September 2010 covering dry and wet periods. To determine the extent of pollution of the surrounding rivers of Dhaka city, various water quality parameters were monitored and a detailed field survey has been conducted within the study area. The sampling cycle and collection dates are shown in the table:

Table 2: Sample collection cycle

Sampling Cycle	Date of Sampling
1	06-01-10 and 15-01-10
2	11-02-10
3	17-04-10
4	29-05-10
5	30-07-10
6	30-09-10

Sampling was conducted between 10 a.m. and 6 p.m. based on weather conditions. For collecting the samples, some fresh plastic containers were used. The capacity of the containers was 1 to 1.5 liters. At first, the water was allowed to flow freely from the source for a few minutes. Then the containers were washed three times with the pump water and then it was filled with that water. After filling it, the cap of the container was closed and sealed to prevent it from contamination. The container was filled fully so that no air bubble remains in the sample. After that, the containers were marked properly with waterproof level including all the information required by the laboratory. Bottles were protected from contamination during transport to and from the sampling site. Ammonia, Conductivity, pH and Sulfide were measured immediately. The rest of the samples were transferred to refrigerator as soon as possible to test the other parameters next day.

2.3 Analysis of Water Samples

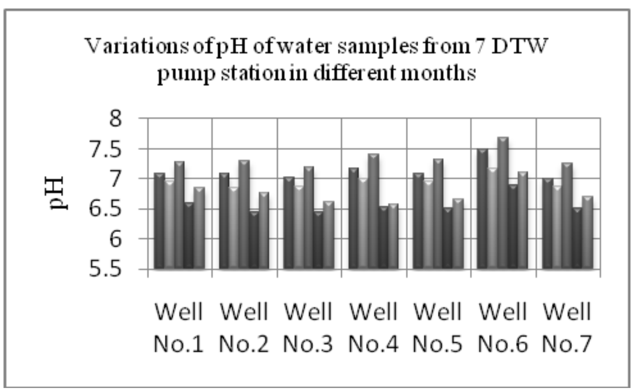
The water samples collected from the 7 pump stations of DWASA Zone 2 were analyzed for pH, Chemical Oxygen Demand (COD), Ammonia, Sulfide, Electrical Conductivity (EC), Chromium, Lead, Iron, Chloride and Total Dissolved Solid (TDS). Temperature measurements were also carried out. pH of the water sample was measured using a pH meter (HACH). TDS and COD were determined by volumetric methods following Standard Methods. Iron concentration was determined using potassium permanganate agent. Iron concentration was measured using the thiocyanate cholorimetric method. Ammonia was determined using Nessler method. Conductivity was measured using a conductivity meter. Chloride concentration was identified using silver nitrate titremetric method. Chromium concentrations in the water samples were measured with an atomic absorption (flame) spectrophotometer (Shimadzu Corporation Model number AA 6800). Lead was also measured with the atomic absorption spectrophotometer.

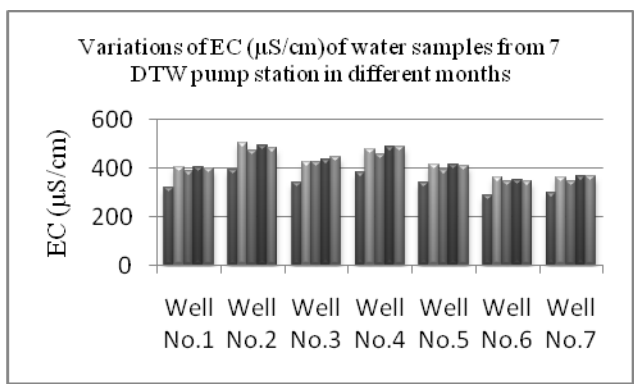
3. RESULTS AND DISCUSSION

To evaluate the risk of ground water contamination due to presence of contaminated lands at Hazaribagh, groundwater samples were collected from the seven tubewells of DWASA located in Hazaribagh area. Water samples were tested for physical, chemical and heavy metals concentration to assess the suitability of water for drinking.

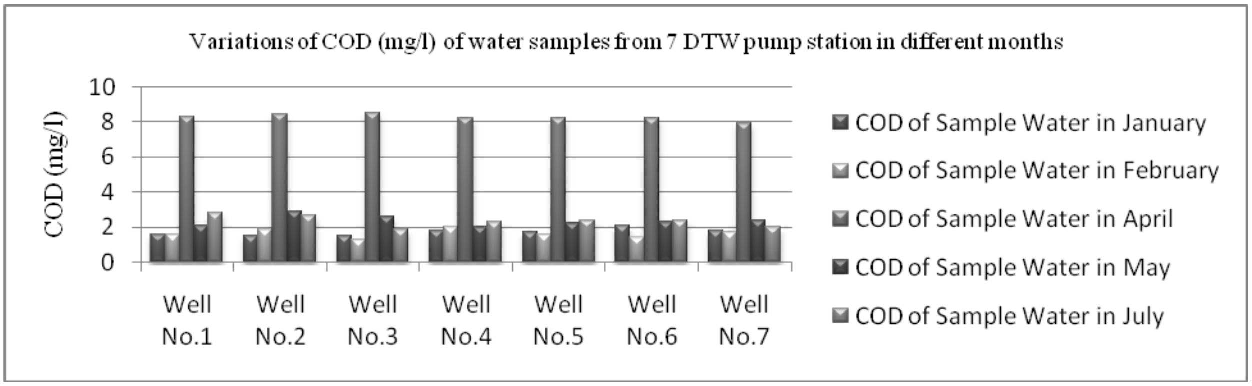
3.1 Characteristics of Groundwater

The groundwater samples have pH within 6.20-7.49 which lies within the Standard limit of WHO and ECR'97. pH in water samples from 7 DTW pump stations were found to follow similar pattern during testing period. For all the well, maximum pH was recorded in the month of April while the minimum during September. In January pH of samples vary within a narrow range from7.00-7.49. During the months of February, April, May, July and September these ranges were 6.85-7.16, 7.18-7.67, 6.45-6.89, 6.57-7.10 and 6.20-6.38 respectively. pH values of Well 6 appear to be higher (averaging close to or below 7.50) compares to the six tube wells where average values varied from about 6.74-7.84. Electric conductivity (EC) of the collected sample follows a more or less common increasing trend from January to September. Samples of well 6 has got the lowest average EC (340 µS/cm) while highest average EC (486 µS/cm) was recorded for well no 2. Average EC of other well ranges from 364µS/cm to 423µS/cm. As expected, conductivity values followed the trend observed for TDS and chloride.





Total Dissolved Solids was recorded for the month of May, July and September only. TDS data shows a typical trend in all the observed well which is closely similar to that of chloride value. TDS values vary from 250mg/l to 420mg/l. TDS was also calculated from EC data by the standard relationship TDS (mg/l) = 0.67* TDS (µS/cm). Most of the results are very close to the analysis result except a few differs abruptly. Chemical oxygen demand of water samples satisfy the maximum limit of COD in drinking water except the samples collected in April. COD value of water samples vary from 1.3 mg/l to 2.8 mg/l for rest of the months whereas in April the value was found to be within 7.9 mg/l to 8.5 mg/l. since this value is abnormally high and found to be within normal range in the next month the COD value of April can be ignored. As the COD values are low ground water of the area can be considered to be free of organic load.

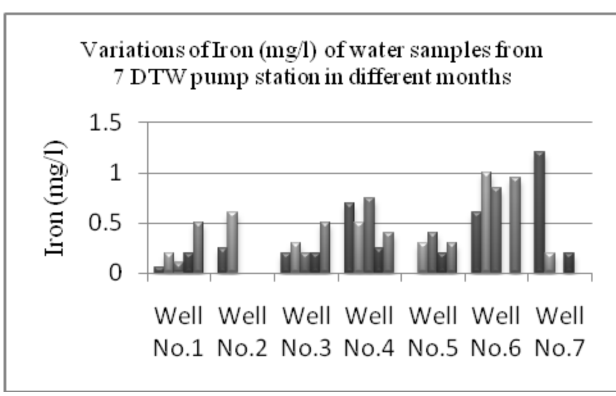


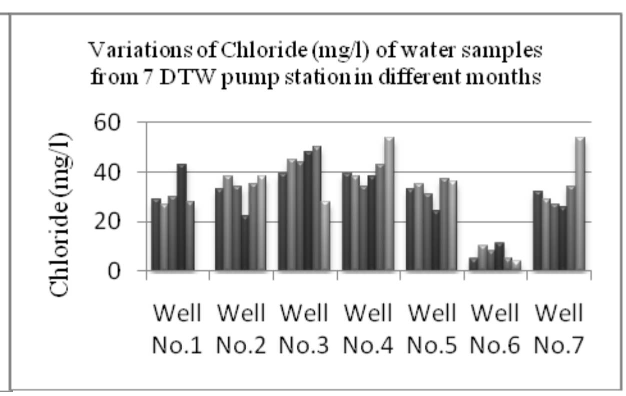
Chloride concentration of water samples from 7 DTW pump stations were found to vary from 4mg/l to 54mg/l. All values satisfy the Bangladesh Standard as well as the WHO guideline value for drinking water. Since tannery waste contains high concentration of chloride, higher concentration of chloride was expected in the water samples. However although no clear well wise trend could be observed, chloride concentration of samples (averaging close to 7 mg/l) from well 6 are clearly lower than those of other wells. Chloride concentrations of water samples from other wells are relatively high, averaging from 32mg/l to 42 mg/l. Iron concentrations was found to be vary from 0.1mg/l-1.2 mg/l. no common trend was observed in iron among the wells but in well 2 iron was detected only in three samples out of 6.one sample of well 6 exceeds the Bangladesh standard for drinking water while 3 samples of well 3 are very near to the standard limit.

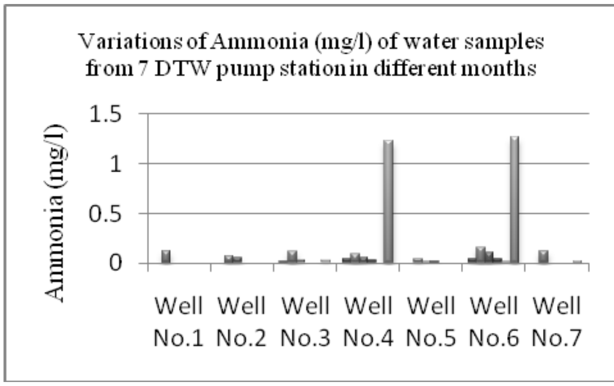
The concentration of sulfide in drinking water should be zero according to ECR and WHO guideline. But it was found from the test results that concentration of sulfide in 9 water sample exceeds the drinking water standard.

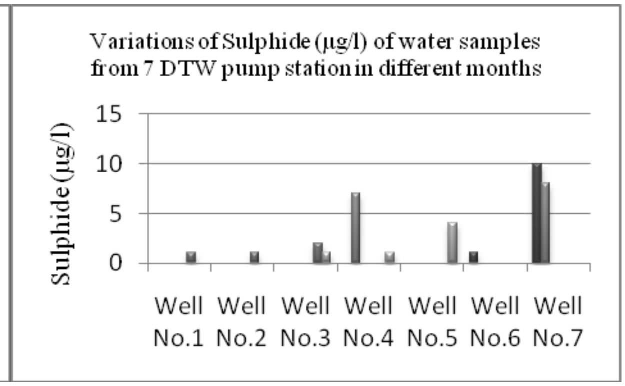
Significant amount of sulfide was detected randomly in almost all the wells once or twice except well no 5. Since tannery wastewater contains high concentration of sulfide, presence of sulfide in groundwater at the periphery of Hazaribagh could indicate the groundwater in this zone might be contaminated by leaching of tannery wastes into the subsurface. However, as no other contaminants (like chloride) seem to have leached out high sulfide concentration could also be of natural origin. Ammonia has been detected in samples from all well. Although no clear trend could be observed, ammonia of water samples of well 4 and well 6 are marginally higher than those of other wells .in fact one water sample from well 6 and one from well 4 exceeded the standard for Bangladesh drinking water. The lowest concentration was recorded 0.12 mg/l (well no 1) which is higher than the detection limit 0.017mg/l.

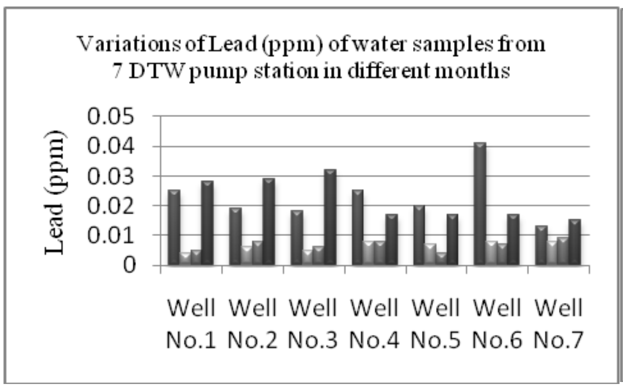
Lead concentrations in water samples from 7 DTW pump stations were found to vary from 0.004mg/l to 0.041 mg/l. lead has been detected in every sample. All satisfy the Bangladesh standard as well as the WHO guideline value for drinking water. For heavy metal analysis sample should be acidified during collection which was not done here. This may affect the result since heavy metal tends to accumulate on the wall of the container. Chromium concentration (vary from 0.001mg/l to 0.005mg/l) in none of the water samples exceeded the Bangladesh standards as well as WHO standard.

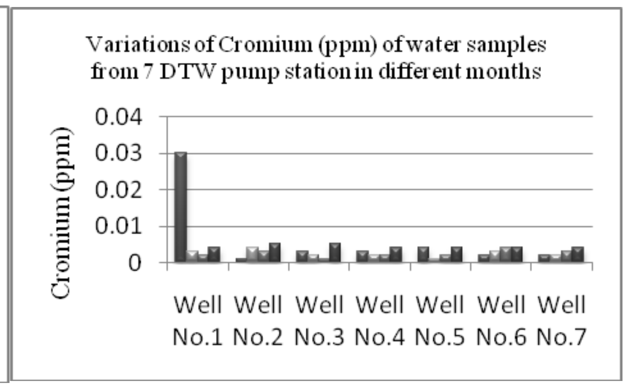






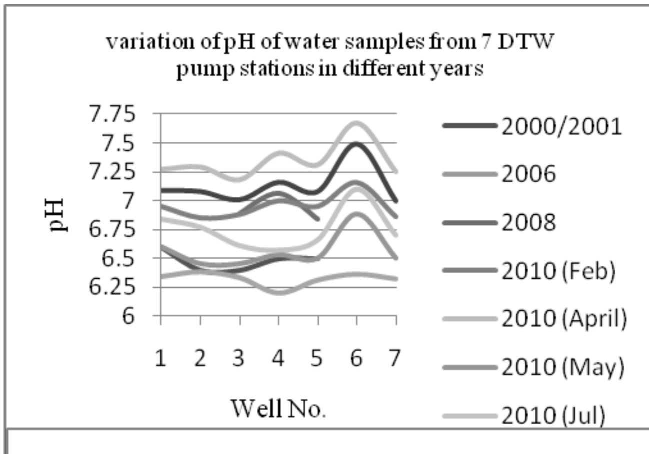


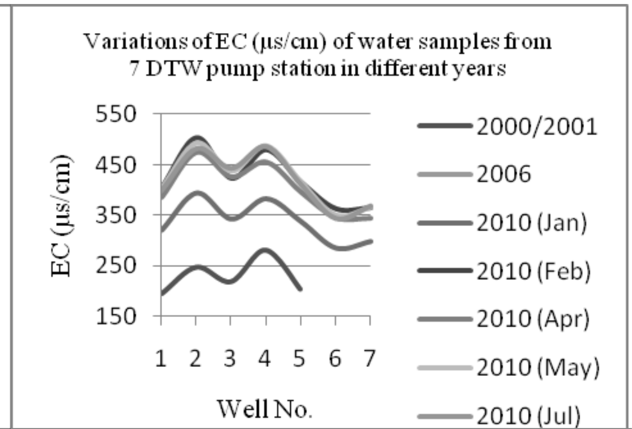


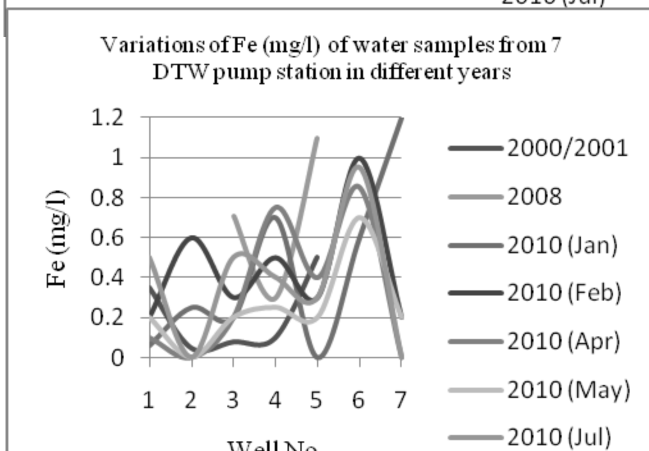


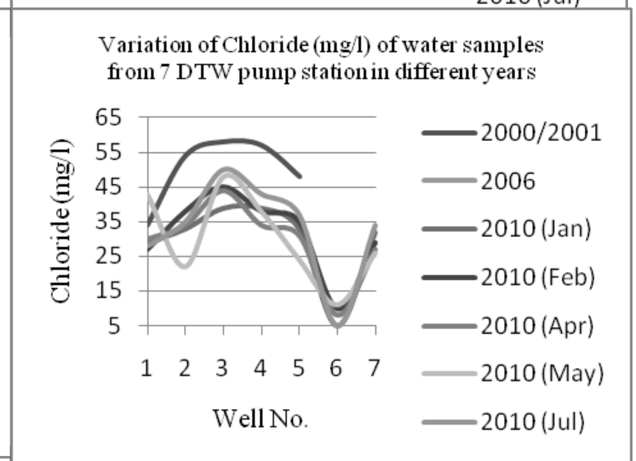
3.2 Comparison with Previous Results

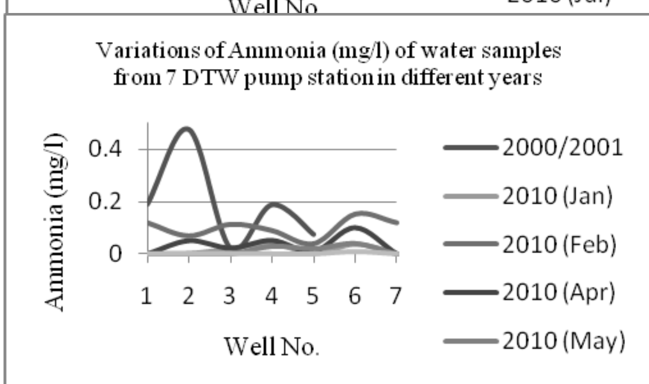
Available previous groundwater quality data of Hazaribagh area were reviewed and summarized for any evidence of trends occurring within the last 10 years. The review included the water quality data of DTW pump station collected by Saha (2001) and data from EU-ASIA ECO PROGRAMME interim report (2008). Some limited data of DWASA (2006) was also evaluated. pH of Well No 1 varied from 6.34-7.27 in 2010 which was 6.6 and 6.94 in the year of 2000 and 2008 respectively. For rest of the wells pH seems to vary within normal range for the last ten years. Electric Conductivity (EC) seems to increase as the value of EC was 194µS/cm-281µS/cm in 2000 while in 2010 EC of water samples were recorded 321- 568µS/cm. Since water samples in different year were not collected at the same season this variation may be due to seasonal change. Total Dissolved Solids of the selected wells varied from 291mg/l-359mg/l in 2000. In 2006 During 2010 TDS value ranges from 250mg/l-420 mg/l. the lowest TDS was observed in water samples of well no 1 in both 2000 and 2010. Most of the parameters vary within a small range except chromium concentration. Study by Saha (2001) found significantly higher chromium concentration (0.002mg/l-0.12mg/l) in the groundwater of Hazaribagh area. Water samples were collected in two phase. Chromium concentration in all water samples collected in June 2001 was lower than those in water sample collected in March 2000. In both case Cr^{3+} concentration exceeded the drinking water quality standard of Bangladesh as well as WHO. But during 2006-2007 no Cr was found in the deep tube wells of Hazaribagh area as all values were below detection limit of 0.007 mg/l (EU-ASIA ECO PROGRAMME interim report, 2008). Present analysis shows that chromium is present in the groundwater at an acceptable limit. The study has been focused to identify the present status of the groundwater quality of Hazaribagh area. Water samples were analyzed for detailed characterization. The analysis results indicate that the groundwater resources at Hazaribagh area that have been sampled in this investigation mostly satisfy the WHO and Bangladesh drinking water standard for the parameters tested in the study. But some elevated levels of COD, sulfide and lead have been detected which may be a matter of concern.

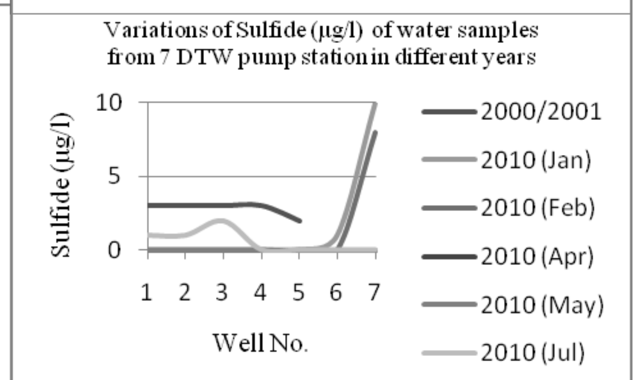

variation of pH of water samples from 7 DTW pump stations in different years
pH
7.75
7.5
7.25
7
6.75
6.5
6.25
6
1 2 3 4 5 6 7
Well No.
2000/2001
2006
2008
2010 (Feb)
2010 (April)
2010 (May)
2010 (Jul)

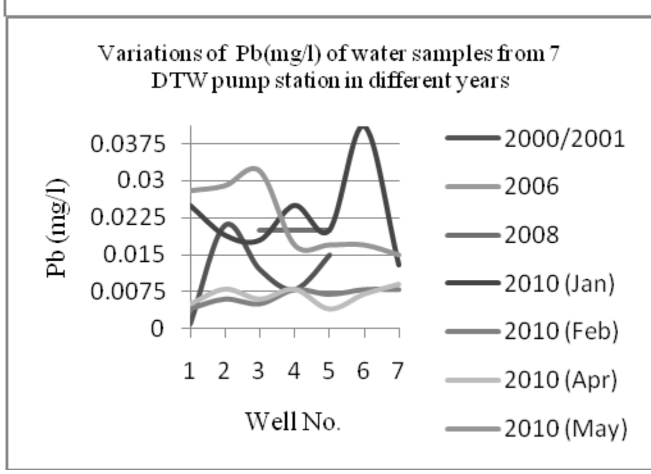

Variations of EC (µs/cm) of water samples from 7 DTW pump station in different years
EC (µs/cm)
550
450
350
250
150
1 2 3 4 5 6 7
Well No.
2000/2001
2006
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)
2010 (Jul)

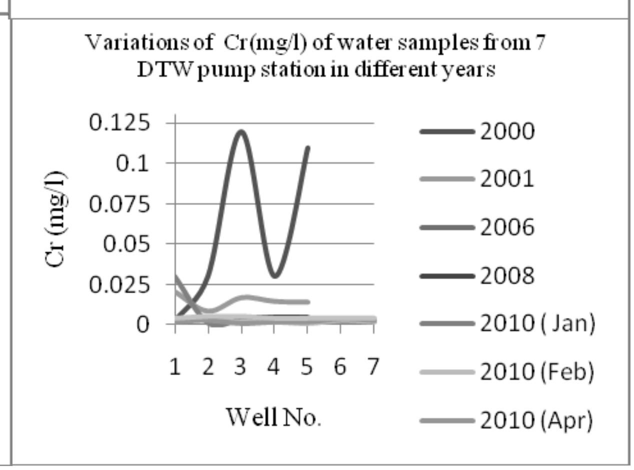

Variations of Fe (mg/l) of water samples from 7 DTW pump station in different years
Fe (mg/l)
1.2
1
0.8
0.6
0.4
0.2
0
1 2 3 4 5 6 7
Well No.
2000/2001
2008
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)
2010 (Jul)


Variation of Chloride (mg/l) of water samples from 7 DTW pump station in different years
Chloride (mg/l)
65
55
45
35
25
15
5
1 2 3 4 5 6 7
Well No.
2000/2001
2006
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)
2010 (Jul)


Variations of Ammonia (mg/l) of water samples from 7 DTW pump station in different years
Ammonia (mg/l)
0.4
0.2
0
1 2 3 4 5 6 7
Well No.
2000/2001
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)


Variations of Sulfide (µg/l) of water samples from 7 DTW pump station in different years
Sulfide (µg/l)
10
5
0
1 2 3 4 5 6 7
Well No.
2000/2001
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)
2010 (Jul)


Variations of Pb(mg/l) of water samples from 7 DTW pump station in different years
Pb (mg/l)
0.0375
0.03
0.0225
0.015
0.0075
0
1 2 3 4 5 6 7
Well No.
2000/2001
2006
2008
2010 (Jan)
2010 (Feb)
2010 (Apr)
2010 (May)


Variations of Cr(mg/l) of water samples from 7 DTW pump station in different years
Cr (mg/l)
0.125
0.1
0.075
0.05
0.025
0
1 2 3 4 5 6 7
Well No.
2000
2001
2006
2008
2010 (Jan)
2010 (Feb)
2010 (Apr)

4. CONCLUSION

The major objective was to monitor the water quality of selected DWASA deep tubewells of Hazaribagh area for the presence of contamination by tannery wastewater and to compare the present water quality of the tubewells with previous results. In order to assess the quality of groundwater, samples from 7 deep tubewells pump stations of DWASA Zone 2 (located in Hazaribagh area) were collected and analyzed. Some major conclusions have been derived from the analysis.

pH followed a common trend. None of the values exceeded the Standard limit. The maximum pH value was observed in April but the value decreased in May. The minimum value was found in the month of September for all the wells. The parameters were also monitored in 2001, 2006 and 2008. Ten parameters were measured in 2010. Among them Ammonia and Sulfide changed abruptly. These two parameters did not follow any common trend. High value of Ammonia was found in the month of September in two wells. In all wells Sulfide exceeded the Standard value in some of the time which indicates the contamination of the water.

COD values for all the wells were within the range except for the month of April. In April the value exceeded the Standard limit. The change of the value of Iron was very random. No similarity of Iron values has been found in the wells. The variation of Iron was different for different wells and did not follow a common trend. Alarmingly high Chromium and Lead concentrations were detected in 2000. But in our investigation, concentration of lead of only one sample was very close to the standard value set by WHO. Chromium concentration of all sample are bellow the standard limit.

Sometimes the water gets polluted a due to various reasons and mainly due to inflow of sewage into the source. Because of the contamination, the parameters of pure drinking water can be changed. This water can be a great threat to ecosystem though some parameters may not in the deteriorate level but the tannery wastes may cause all kind of water pollution in the near future.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SUSTAINABILITY OPTIONS OF SANITATION PROJECTS IN THE SLUMS OF KHULNA CITY

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ABSTRACT

There are more than 520 slum and squatter settlements in the third largest industrial Khulna City of Bangladesh where more than 20% of total city population of 1.5 million live. There are some specific government organizations (GOs) and NGOs in Khulna City that implemented and still now implementing a number of sanitation projects in the slums since 1985. In spite of implementation of repeated sanitation projects or development initiatives by GOs and NGOs, the sanitation system in the slums could not sustain. Sustainability options are very important components of sanitation and hygiene projects of present days. In the study, communal and shared latrines are found nearly abandoned, unhygienic and in poor condition due to absence or lack of community participation for repairing and maintenance whereas twin pit latrines used mostly by individual families are found good. Some of the projects have shown good results with better performance whereas many projects could not sustain with good results. The paper highlights the sanitation projects in the slums of Khulna city city since 1985, options considered in sanitation project for bringing their sustainability, reasons of insustainability and failure of the projects and recommendations for bringing their sustainability.

Key Words: *Slum, sanitation project, sustainable option, community participation*

1. INTRODUCTION

The success or sustainability of a sanitation project is achieved when it meets its objectives and is maintained by the users over a significant period of time (Ahmed et. al, 2000). A sanitation project is regarded as sustainable when it provides an efficient and reliable service at a level which is desired; can be financed or co-financed by the users with limited but feasible external support and technical assistance; and is used in an efficient and effective way without negatively affecting the environment. Despite huge investments in sanitation during the United Nations International Drinking Water Supply and Sanitation Decade (1981-90), an estimated 2.5 billion people, half of the developing world and more than 35% of the world's population lack access to adequate sanitation (WHO, 2000). The Local Government Division (LGD) of the Ministry of LGRD&C (1998) in its "National Policy for Safe Water Supply & Sanitation 1998" mentioned that it is globally recognized that physical provision of services alone is not a sufficient precondition for sustainability or improvement of health and well being of the people. Greater attention needs to be focused on elements of behavioral changes of users and sustainability through user participation in planning, implementation, management and cost sharing. The need for change within the conventional programs are recognized by the Government and all stakeholders in the sector. The aim to bring about the changes calls for transition from traditional service delivery arrangement. Institutionalization of strategic partnership process between the central and local Government in coordination with other organizations within the civil society is one way of bringing about this change. Regarding community participation in planning and management of sanitation system UNICEF (2000) in its report titled "Sanitation for All" highlighted that community members involved in sanitation programmes from planning to management to sanitation promotion feel a sense of ownership and show a greater willingness to help programmes succeed. Communities should be supported with training as well as access to parts, materials and financing. Programmes planned and implemented exclusively by central authorities often lack the flexibility and diversity required to respond well to local needs. Approaches lacking community participation are rarely sustainable; programmes ideally should involve all sectors of national and local government as well as civil society. The report also highlighted some important sustainability and in-sustainability aspects of sanitation programs in different countries. The report mentioned that it is crucial that waste disposal systems be tailored to the cultural and social setting of each family and community. Latrines should be non-polluting, affordable, user-friendly and simple to construct and maintain. Families themselves are the best judges of what works. In Myanmar, where a UNICEF-assisted sanitation programme offered people a choice of latrines, enthusiasm for the programme was so high that 800,000 families built latrines in just one year. It also mentioned that even the best technology will not work in the long run unless it has been chosen by community members who are given the range of options. Latrines chosen for families rather than by them commonly remain unused or improperly used.

2. OBJECTIVES OF THE STUDY

Objectives of the study are:

- To study the sanitation projects in the slums of Khulna city since 1985
- To find out the options considered in sanitation project for bringing their sustainability
- To find out the reasons of insustainability and failure of the sanitation projects in the slums
- To draw some recommendations of more options for sustainability of sanitation projects

3. METHODOLOGY

The study was conducted during 2008-2009 in three slums of Khulna City, namely Rupsha slum on Government khas land and Christian community land; Montu Colony slum on Bangladesh Railway land; and Kulibagan slum on Ispahani Company private land. Primary data for the study was collected through structured household questionnaire survey, interview with group and committee members of sanitation and related projects, and concerned GO-NGO staffs. A total of 384 households (22%) were randomly selected and surveyed from the slums, where 215 households were from Rupsha slum, 75 households were from Montu Colony slum and 94 households were from Kulibagan slum. Books, project reports and implementation guidelines, journals, newspapers etc. were reviewed as secondary data sources.

4. KHULNA CITY AND STUDY SLUMS

Khulna is the 3rd largest industrial city of Bangladesh. It is a divisional city and acts as regional hub of administrative, institutional, commercial and academic affairs. It is located on the banks of the Rupsha and the Bhairab rivers. The city covers an area of 45.65 square kilometers with a population of near about 1.5 million. There are 520 slums in Khulna city. Most of the slums of Khulna City have established after 1971, the year of independence of Bangladesh. About 73% slums have been established after 1971. Among 73% slums, 15.2% in 1971-75, 15.8% in 1976-80, 12.9% in 1981-85, 10.2% in 1986-90 and the rest 18% in 1991-2005. In the slums of Khulna city 188,442 poor and landless people live. Density of population per acre and per kilometer in the slum area is 538 and 132,988 respectively (CUS, 2006). Following three slums, located in three diverse locations of Khulna city are the study slums:

Rupsha Slum

Rupsha Slum is situated in Ward 22 of Khulna City Corporation. It is 2.5 kilometers on the north-east side from the Central Business District (CBD). It is on the west side of Rupsha embankment of BWDB which is along the western bank of River Rupsha. Total land area of Char Rupsha Slum is 4.0 acre. The strip of land just along the embankment is owned by the Government and the rest area after along the western side is owned by the Christian Baptist Church. The slum was established nearly in 1960s for settling the poor workers of nearby Dada Match Factory, Rupsha; Khulna Shipyard, Labonchara; and water transports namely boats, launches and steamers. Around 5,500 people live in around 1,000 households. The slum is bounded by BWDB embankment and Rupsha River on the east; Caritas and LGED office premises on the west; Rupsha Natun Bazar on the north; and Rupsha bus stand on the south.

Montu Colony Slum

Joragate Montu Colony Slum is situated in 21 Ward of Khulna City Corporation. The land is owned by the Railway Department of the Government of Bangladesh. It is established immediate after the liberation war of 1971. First time a population of around 75,000 lived here but it reduced at around 35,000 after an eviction during the Mujib regime. The slum also faced an eviction during the Zia regime. As the land is taken lease from the Department of Railway by Mr. Montu, a contractor of the area of that time, the area is named as Montu Colony. There are 350 households in the slum with a population of around 1,925. The 5 no. ghat (landing point at river bank) of Bhairab river was famous and busy for shipment of goods from cargo, launches, trawlers and boats to Khulna city and adjacent industrial area. In past, transport, goods landing and industrial laborers lived in this slum. The Montu Colony slum is bounded by Joragate Rail crossing on the west, Koiladepo slum and Bhairab river on the east, KCC Kitchen market (Kutcha bazar) and the road of 7 no. ghat on the north and Railway vacant land and *Natun Bastee* on the south.

Kulibagan Slum

Kulibagan slum is in Ward 3 of Khulna City Corporation. The Slum is located about one kilometer away from Daulatpur traffic island towards north at the western side of Khulna-Jessore highway. Area of the slum is 2.5 acre. Kulibagan is at the southern side of Moheswarpasa Modhodanga road which is originated from the Khulna-Jessore highway towards west. The slums are bounded by Shashibhusan High School at the south, Sadhubagan slum and Urban Primary Health Care Center of KCC at the north, Khulna-Jessore Highway on the east and Daulatpur Police Station on the west. Around 2,390 people live in around 435 households and workers mess of the slum. The land of the slum is owned by Ispahani Group of Industries.

5. SUSTAINABILITY OF SANITATION SYSTEM

The achievement of sanitation coverage is not enough, it must be sustainable to gain the benefit from total sanitation coverage. World Commission for Environment and Development (WCED) has defined sustainable development as "development that meets the needs of the present without compromising the ability of the future generation to meet their own needs". Sanitation improves environment and hence sustainability refers mainly to "functional sustainability" and to some extent to "environmental sustainability" (GOB SACOSAN, 2003).

5.1 Functional Sustainability

Improvement in sanitation is a change in practice and needs people's acceptance and urge to build and sustain. In case of sanitation, proper operation/use and maintenance are most important for sustainability of the system. There are examples in which 100% sanitation coverage achieved in an area or in a pilot project gradually collapsed to disuse (GOB SACOSAN, 2003). The functional sustainability of sanitation facilities are shown in Figure 1.

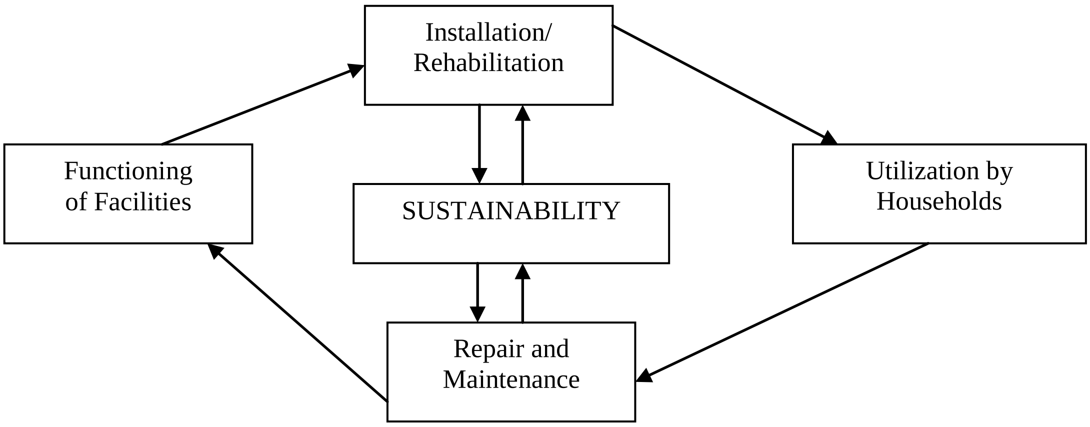

Figure 1: Functional Sustainability of Sanitation Facilities

The key issues of functional sustainability are:

- Installation and replacement of components, when needed
- Proper use of facilities
- Repair and timely maintenance
- Keep the system functional

5.2 Environmental Sustainability

The provision of sanitation is intended to improve environment and quality of life. Hence, there is less possibility that the service provisions cause degradation of the environment. However, improper design and maintenance of sanitation facilities can cause environmental pollution (GOB SACOSAN, 2003). Some of the possible negative impacts of sanitation facilities include:

- Damage/degradation of natural resources by poorly managed facilities
- Groundwater pollution from pit latrines /soak well etc.
- Pollution from disposal of sludge from treatment of water and wastewater
- Pollution from disposal of effluent from treatment plant/ facilities

6. ORGANIZATIONS FOR SANITATION AND SANITATION PROJECTS IN THE SLUMS

Government bodies and NGOs implement some slum improvement programs in pursuit of meeting the needs of the poor slum people in the urban area. The organizations that absolutely work on sanitation are the sanitation organizations. There are some organizations that keep sanitation as one of the major components of their implemented development projects. List of sanitation and related projects implemented in the slums in Khulna City during the period 1985-2009 is enumerated in the Table 1. Once the previous sanitation projects and sanitation components do not sustain, repeated sanitation projects are needed for implementation in the slums.

6.1.Organizations for Sanitation in Khulna City

There are specific government organizations (GOs) and NGOs in Khulna City that implemented and still now implementing some sanitation status improvement projects in the slum areas since 1985.

6.1.1 Government Organizations

LGED, KCC and DPHE have been working in Khulna City for slum status improvement since 1985 through SIP. WASA has also been established in Khulna City in 2008 for providing water and sanitation services in Khulna City. Before establishing KWASA, KCC was responsible for the water supply and sanitation services in Khulna City. All the staff of the Water Supply Department of KCC has been shifted in KWASA. Activities of KWASA at this primary stage are preparation of plans for water supply networks and infrastructures, taking initiatives for collection of arrear water bills and system development etc. Firstly, KWASA will work for water supply and gradually it will work for sanitation and drainage.

6.1.2 Non Government Organization (NGO)

World Vision (WV) mainly works with temporary slums that are of cluster type and poor in drainage, road and sanitation condition on government land which are under threat of eviction. It has started its development interventions in the Khulna City slums around 10 years ago. It provides supports to establish sanitary latrines and construct low cost houses. All the groups of WV are of female. Each group consists of 30 women. Nabolok Parishad, a local NGO has been working in Khulna City with implementation of the ASEH project funded by WaterAid Bangladesh (WAB) since March 2005.

6.1.3 Private Sector Organizations

In Khulna City there are a large number of profit making public sector organizations in the form of contractors, enterprises and shops for production, marketing, installation, maintenance and repairing of both low cost and high cost sanitary latrines. These private organizations also work as supporting organizations for installation, repair and maintenance of water supply systems and facilities.

6.2 Sanitation and Related Projects in the Slums of Khulna City

A number of sanitation and related projects starting with Slum Improvement Project (SIP) since 1985 have been implemented in Khulna City. Brief of these projects with title, donors, implementation period, implementing agency and project area are shown in the Table 1.

Table 1: Sanitation and related projects in the slums in Khulna city

Sl.	Title of the Project	Donor	Period	Implementing Agency	Project Area/KCC Wards
1.	Slum Improvement Project (SIP) Phase I & II	GOB, UNICEF	1985-1995	LGED through KCC	All Wards
2.	Urban Basic Service Delivery Project (UBSDP)	GOB, UNICEF	1994-1996	LGED through KCC	All Wards
3.	Support for Basic Service in Urban Areas Project (SBSUAP) Compt.-Part II	GOB, UNICEF	January 2001-June 2006	LGED through KCC	All Wards
4.	Urban Slums and Fringes Project	GOB, UNICEF	1999/2003-2006	LGED through KCC	19 Wards
5.	Local Partnership for Urban Poverty Alleviation Project (LPUPAP)- Part I	GOB, UNDP, UNCHS	January 2000-June 2006	LGED through KCC	All Wards
6.	Urban Partnership for Poverty Reduction Project (UPPRP)	GOB, UNDP, UN-HABITAT	July 2006-June 2013	LGED through KCC	All Wards
7.	Advancing Sustainable Environmental Health (ASEH)	WaterAid Bangladesh	April 2005-March 2008	Nabolok Parishad	All Wards
8.	Micro-credit and Child Education Sponsorship	USAID	1998 to till now	World Vision	Slums and fringe areas

Source: KCC, DPHE and Nabolok Parishad Office, 2008.

Slum Improvement Project (SIP)

In view to fulfill the government planning and providing facilities to low income people of the country LGED with the help of UNICEF worked for providing basic urban service through the Slum Improvement Project (SIP) since 1985. Various service facilities have been provided to 45,000 families of 4 cities and 21 pourashavas. The facilities include establishment of tube-well, sanitary latrine, construction of footpath, drain, dustbin, street light, primary health care, satellite school, self employment human resources development, IGA skill development training and various package programs for improving basic services (PP LPUPAP, 2001). As a result slum dwellers of 225 slums of 25 municipalities/cities have improved their living condition and environment. With the rapid increase of urban population, demand for urban infrastructure and basic services has also been increased. But the facilities provided through SIP were limited compared to the increased demand. SIP was implemented in almost all the slums of Khulna City during the period 1985-1995. Some tube-wells and sanitary latrines were established in the slums. Households of the study slums could not clearly memorize about the sanitation facilities provided under the SIP. This is due to lack of effective community mobilization and community participation mechanisms in the project activities.

Urban Basic Service Delivery Project (UBSDP)

UNICEF assisted Urban Basic Service Delivery Project (UBSDP) established management and implementation structures in 4 city corporations and 21 pourashavas to address vulnerable population groups. The evaluation of this project done by Center for Urban Studies (CUS) highlighted the importance of implementing activities through local government structures. But there is no other regular structure for any of the local Govt. bodies in Bangladesh to deliver adequate basic services to the urban poor excepting a few slum development staff in Dhaka City Corporation H.Q. The Urban Basic Service Delivery Project is an attempt to bridge the gaps for better coordination among the local Govt. authorities and development partners. The evaluation study recommended continuation of UBSDP and also its possible extension to another phase. CUS also recommended that UNICEF should support on advocacy campaign to strengthen the urban local governments and the civil society to work for improving the quality of life of the urban poor. There were different committees in the project-Central Coordination Committee (CCC), chaired by LGD-Secretary, Project Coordination Committee (PCC), chaired by respective Mayor/Pourashava Chairman and Project Implementation Committee (PIC), chaired by respective Ward Commissioners oversaw and implemented all UNICEF assisted project activities in the urban areas. Under the project 35 UDCs were established in Khulna City to perform basic health care services including Expanded Programme of Immunization (EPI), Vitamin A Capsule (VAC) distribution, growth monitoring, hygiene education, sanitation, IGA & education for children.

Support for Basic Service in Urban Areas Project (SBSUAP)

Support for Basic Service in Urban Areas Project (SBSUAP) was implemented in 206 Urban Development Centers (UDCs) and 21 Pourashavas functioning under UBSDP. The reach of UDCs was increased from 2,000 to 10,000 urban population by involving major NGOs and partners like ADB. Each UDC with existing staff provided basic health care services including Expanded Programme of Immunization (EPI), Vitamin A Capsule (VAC) distribution, growth monitoring, hygiene education, sanitation, IGA & education for children. UBSDP project staff with City Corporations/Pourashavas Health Assistants provided same services in the urban areas. UNICEF closely worked with UNDP which supported LPUPAP of LGD in the area of micro-credit and basic infrastructure development including sanitation facilities. UNICEF assisted SBSUAP mobilized national programmes resources of Health & Nutrition, Water and Environmental Sanitation (WES) for sanitation education & installation of tubewell/latrine etc., Child Development & Education, Protection, Planning & Communication Program towards meeting the basic needs of the children & women in the urban areas with special focus to the urban poor.

Local Partnership for Urban Poverty Alleviation Project (LPUPAP)

The Government of Bangladesh and UNDP implemented the Local Partnership for Urban Poverty Alleviation Project (LPUPAP) to support the efforts, energies and resources of the urban poor to get out of the cycle of poverty. The project began its operation in January 2000 and continued til June 2006. Subsequent delays in preparation of the second phase and concerns to maintain the momentum of field activities resulted in LPUPAP being further extended to June 2007. The funds from UNDP have totaled US$ 18.797 million. The government of Bangladesh contribution was Taka 60.6 million. LPUPAP worked in four city corporations-Chittagong, Khulna, Rajshahi, Barisal-and seven pourashavas-Kustia, Gopalganj, Bogra, Sirajganj, Mymemsingh, Narayanganj and Hobiganj. Under the project, the poor of the cities were organized and empowered to identify their own problems, find out solutions and implement their own programmes to address their needs. LPUPAP built the capacity of community organizations to establish creative partnerships with local government and with other agencies including NGOs and private sector businesses. LPUPAP adopted a community-based, demand driven approach with strong links to the local government through the elected representatives and government officials. Project staffs were in a facilitating role. The urban poor communities were the main project partners and community mobilization and empowerment began with the formation of Primary Groups (of about 20 families) and support to community management of regular savings, leading over time to a community-managed micro-credit fund. Primary Groups then grouped themselves into Community Development Committees (CDCs) representing 200-300 families. Of the Primary Groups, 90% were women's groups and consequently women were strongly represented in the leadership of the CDCs. LPUPAP had three main components. A *Community Development Fund (CDF)* which financed the construction of basic infrastructure in the communities. The CDCs were supported to undertake participatory community action planning and the priority infrastructure was then constructed through community contracts signed between the local government and the concerned CDC. Thus, all construction was managed and overseen by the community leaders, majority women. LPUPAP experience is that such construction has been effective, efficient and of high quality. A *Poverty Alleviation Fund (PAF)* supported apprenticeship training, again planned, prioritized and managed by the community and implemented through contracts between the local government and the CDCs. A third component supported *Community Empowerment and Capacity* building of both communities and local government. In addition to more formal workshop and field level training, community empowerment and capacity building was strengthened by peer-to-peer learning between CDCs within and between towns. Elected representatives and local government officials accompanied community visits to other towns. This helped to develop their awareness of the benefits of the project procedures and their commitment to a community-managed approach. LPUPAP project was implemented in Rupsha and Kulibagan slums. The project had no intervention in Montu Colony slum. Training and capacity building has been a central pillar of LPUPAP. Right from the initial project orientation training for project staff, training has preceded and accompanied all aspects of project implementation. Staff trainings were provided on Community mobilization, Savings and credit, Community action planning, Community contract management: both infrastructure construction and apprenticeship training and Monitoring. Trainings for community were on Participatory urban appraisal, Community action planning, Gender and leadership, Savings and credit management, Accounting, PAF management, Apprenticeship programme, Community contract implementation and management, Construction of physical works and accounting, Operation and maintenance of physical works, Participatory monitoring and Auditing savings and credit (LPUPAP Final Report, 2007).

Advancing Sustainable Environmental Health (ASEH)

Nabolok Parishad implemented Advancing Sustainable Environmental Health (ASEH) project funded by WaterAid Bangladesh in all the wards of KCC during the period April 2005-March 2008. Giving priority of community participation in all spheres of the project is one of the strategies for attaining sustainability of the ASEH project. The project had direct control on technology options of latrines, ability to pay analysis of the community and ensure cost sharing of the community through maintaining bank accounts. ASEH has established Hardware Management Committees in the community of the slums. Inadequate fund for continuation of the sanitation project in future is the major limitation of the ASEH project. Community Situation Analysis (CSA), mass awareness, opinion sharing, knowledge level and practice level monitoring, performing hygiene promotion sessions in the courtyards, tea-stalls, mosques and schools etc. are the activities of ASEH with and for the community people.

Micro-Credit and Child Education Sponsorship Project

World Vision directly does not implement sanitation project in the slum areas. It works in the slums of temporary pattern on unauthorized or government or government khas land. It has been implementing Micro-Credit and Child Education Sponsorship Project in Montu Colony slum from around 1998. It provided the slum households that are the members of its Micro-Credit and Child Education Sponsorship Project with single pit latrines at subsidized rate.

7. SUSTAINABILITY OPTIONS OF SANITATION PROJECTS IN THE SLUMS OF KHULNA CITY

There are many sustainability options of sanitation projects in urban slums. Active and adequate community participation; developing sanitation volunteers and their skill at community level; increased community awareness on benefits of hygiene sanitation; improved socio-economic condition of slum dwellers; appropriate sanitation technologies in the slums; community knowledge on appropriate sanitation technology; adequate fund, logistics and technical staff of concerned organizations; transparency and accountability in project implementation; regular repair and maintenance of latrines and tubewells; availability of adequate water for sanitation are the sustainability options of sanitation projects.

7.1 Active and Adequate Community Participation

Active and adequate community participation can bring sustainability of sanitation projects. Recently completed and ongoing sanitation projects had better mechanisms of community participation than the previous projects. Community cost sharing for establishment and repair-maintenance of latrines and tubewells, community involvement in purchasing quality hardwares, inter and intra community exchange visits and idea sharing etc. were the major components of community participation in recently completed and ongoing sanitation projects. Such components were merely found in previous sanitation projects. This is why most of the sanitation hardwares of SIP and some of the sanitation hardwares of LPUPAP could not sustain. Groups and committees formed under almost all the sanitation projects were found quite inactive after their completion or in the period of nearly completion.

Community involvement was found better in the sanitation projects of now days than the previous projects. Slum dwellers are generally found to memorize ongoing and recently completed project interventions and the implementing organizations. Nabolok Parishad has been currently implementing the WAB funded ASEH project in the study slums and as such the name of Nabolok Parishad was found instant in the memory of slum households.

Table 2: Memorizing Water and Sanitation NGOs and Projects by Households

Name of slums	**Water and sanitation NGOs and projects**			**Total**
	Nabolok Parishad	**LPUPAP/KCC/CDC**	**World Vision**	
Rupsha slum	146	69	0	215
	68.0	32.0	0.0	100.0
Montu Colony slum	26	0	49	75
	35.0	0.0	65.0	100.0
Kulibagan slum	74	20	0	94
	78.4	21.6	0.0	100.0
Total	246	89	49	384
	64.1	23.2	12.7	100.0

Source: Household survey, 2008-2009.

Table 2 reveals that 78.4% and 68% households of Kulibagan and Rupsha slums mentioned Nabolok Parishad’s activities good respectively. Among the three study slums, World Vision works only in Montu Colony slum since around last 10 years. Around 65% households of Montu Colony slum mentioned their involvement with the World Vision. Slum dwellers could not easily memorize the interventions of SIP, UBSDP and LPUPAP. This was mainly due to limited and inefficient community participation interventions of the projects. Most of the slum people are poor. They have little fund or money to invest for sanitation development by their own. They can contribute in a sharing approach. Saving or paying a very insignificant amount at regular interval in a community fund can be an option of raising community fund. If the user community become organized and can participate actively in sanitation and related development projects through sharing of cost for establishment and repair-maintenance of sanitation services, the services must be sustainable.

7.2 Developing Sanitation Volunteers and their Skill at Community Level

Skill is an important local resource that can contribute significantly in sanitation development of the slums. Sanitary latrines and tubewells installed in the slums by different development programs and projects are found in poor condition and about to out of order due to absence of proper repair and maintenance. The GOs and NGOs can not provide adequate services in time for the repair and maintenance of the latrines and tubewells due to shortage of their skilled staff, lack of fund and logistics and even absence of developed mechanisms for rendering repair and maintenance related service to the slums. If the skill of some volunteers or interested persons of the slum areas on repair and maintenance of the latrines and tubewells are developed through providing training, orientation and attachment with the concerned organizations, they can continue regular repair and maintenance of the latrines and tubewells in a sustainable manner.

As the slum households has to face problems to get services for repair and maintenance and cleaning of the latrines from the sanitation organizations, it is important to create experienced person and organization within the community for construction, repair and maintenance of latrines. Unsustainability of CBOs or groups formed under different sanitation project and discontinuation of activities or services of different NGOs are also the driving forces for creating experienced person and organization for self sustenance of the community.

Table 3: Opinion on Creating Experienced Person and Organization within Community for Construction, Repair and Maintenance of Latrines

Name of slums	Need of experienced person		Total
	Yes	No	
Rupsha slum	215	0	215
	100.0	0.0	100.0
Montu Colony slum	75	0	75
	100.0	0.0	100.0
Kulibagan slum	41	53	94
	43.8	56.2	100.0
Total	331	53	384
	69.2	30.8	100.0

Source: Household survey, 2008-2009.

Table 3 reveals that 69.2 % households have given their opinion to create experienced person and organization within the community for construction, repair and maintenance of latrines. User communities can mobilize local resources and can create local institutions in the form of CBOs and Groups and Committees for sanitation development and sustainability.

7.3 Increased Community Awareness on Benefits of Hygiene Sanitation

Hygiene sanitation keeps the slum environment neat and clean, reduces frequency of diseases and illness, keeps the slum people in sound health, reduces medical costs and sufferings of family members, saves school hours of children and working hours of adults, and thereby enhances productivity of slum people to significantly contribute to reduce poverty. Community people of the slums were found not well aware on these benefits of hygiene sanitation. If the community people better know the benefits of hygiene sanitation, they feel interest to improve the sanitation status and its sustainability.

7.4 Improved Socio-economic Condition of Slum Dwellers

Most of the slum people are poor and live from hand to mouth. About 46% households of the study slums had monthly income in the range of Tk.3,000/= to Tk.6,000/=. As the slum dwellers can hardly afford food and clothing, they can not think to invest for improving their sanitation status. This is why, Government Organizations and NGOs have to undertake sanitation status improvement projects repeatedly for the slums. Slum dwellers in now days than the previous period have been contributing more in cost sharing for establishment and renovation of WatSan facilities i.e. latrines and tubewells. If the socio-economic condition of the slum dwellers is improved, they can contribute significantly for establishment and repair-maintenance of WatSan facilities towards becoming the sanitation management system sustainable. Government organizations and NGOs should take more programs on socio-economic development of the slum dwellers.

7.5 Appropriate Sanitation Technologies in the Slums

Slum specific appropriate sanitation technology is an important consideration for sustainability of sanitation management system. If the technologies i.e. latrines do not last long, the system becomes unsustainable. This is why, the slum dwellers need to select appropriate sanitation technologies. Selection of appropriate sanitation technology depends on soil and sub-soil condition, and socio-economic and environmental condition of the slums. Most of the slum dwellers of the study slums use simple pit latrines for their excreta disposal. Pit latrines are the most common and simplest form of excreta disposal in many developing countries. These are almost universally acceptable and widely used in low-income urban communities. Though pit latrines are often not appropriate, they are the cheapest system and most appropriate for individual householders responsible for their own sanitation.

Different types of sanitary latrines are provided under different projects in the slums and so slum dwellers of the study area were found to use different types of latrines. About 55.5%, 20.1%, 15.2% and 9.2% households of the study slums were found to use septic tank system community latrines, twin pit water sealed latrines, simple single pit latrines and hanging latrines respectively. The slums of Khulna City mostly are in low lying urban fringe areas and on derelict lands. The slums are characterized with organic clayey soil and poor drainage. As the single pit kutcha latrines of the study slums were found to last and remain good for a shorter period, single pit kutcha latrines are not suitable and appropriate for the slums. The existing saturated ground of the slums during high ground water level in monsoon season makes the soil wet and the simple single pit kutcha latrines fragile. In this case pits are filled up within short period from their establishment. Pucca twin pit and septic tank system latrines are appropriate for the slums of Khulna city as they were found to last for longer period than the simple single pit kutcha latrines.

7.6 Community Knowledge on Appropriate Sanitation Technology

Technology focuses on the knowledge, the culture, the infrastructure and the tools. Appropriate sanitation technology provides the users good sanitation services that reduce the health and environmental risks of the community. The technology has to match the type of solutions the community wants, is willing and able to manage and sustain and that are in line with the technical, socio-economical, environmental, institutional and political conditions and capacities of the community. If new sanitation technology has to be introduced, testing is needed to allow for the necessary adaptation to the local conditions and to ensure that adequate operation and maintenance can be taken care of before promoting large-scale application. This also includes a review and adaptation of training materials for the different levels of education involved in the use of technology. Slum dwellers mainly require the technical services for cleaning their pits and septic tanks and for repair of latrines and tubewells. But they do not get the services easily and instantly from the Conservancy Department of KCC and sweepers of different sweeper colonies in Khulna City. Community people are not well acquainted with the technical aspects of the latrines. Almost all the community latrines that were provided from SIP and LPUPAP needed renovation. Some of them required replacement with new construction. This implies that the community people could not maintain the latrines properly. And the organizations under sanitation projects could not made the community adaptable with the technologies by providing training and guidelines. Again, the sanitation system in the study slums was found unsustainable as the slum dwellers were not well acquainted with the slum specific appropriate latrines. Nabolok under the ASEH has followed largely the issues of sustaining the technologies used in the slums.

7.7 Adequate Fund, Logistics and Technical Staff of Concerned Organizations

The institutional environment needs to be taken into account in setting up a sanitation system. Management of the system must be adequate in order to make the level of service sustainable over time. Sustainability of the system can easily be achieved if the management capacity at the local level matches the operation and maintenance requirements of the system, requiring only a minimum of support from the government or external institutions.

Almost all the sanitation and community development projects implemented in the slums were designed by LGED and funded by GoB, Unicef, UNDP and USAID. The projects were implemented in collaboration with KCC, DPHE, NGOs, CBOs and profit making private organizations. The projects could not sustain due to limitations and drawbacks of these institutions. The limitations and drawbacks include inadequate fund, limited number of technically sound staff, inefficient linkage and coordination among the institutions, and inadequate research initiatives on sanitation issues of the slums.

KCC, LGED, DPHE and NGOs (Nabolok Parishad, World Vision, WaterAid Bangladesh, PRISM Bangladesh and others) had and have lack of fund to implement and continue sanitation projects for a longer period. If the concerned organizations could continue project activities after their completion and withdrawal of donor support, sanitation status of the slums would always remain good. Due to lack of fund, the organizations could not continue sanitation projects for a longer period and could not cover all the households of the slums with sanitation facilities and services. Sanitation organizations can not provide the slum dwellers with quality sanitation services in time due to having inadequate logistics and technical staff.

Biogas generation from community latrines and sell to nearby poultry farms, shops, households etc. can generate income for development and running of sanitation programs. Sanitation development fund from the ADP of Local Government bodies i.e. KCC can be an can be used continuously for sanitation as the slum owners and households pay directly or indirectly holding tax and contribute to local and national development. Local NGOs can collect fund from national and international donors in the form of grant.

7.8 Good Coordination among Community and Concerned Organizations

During the initial stage of sanitation projects, the implementing organizations had good coordination with the community and other concerned organizations. This was mainly due to budgetary allocation in the projects for meetings, orientation, training, exchange visits etc. in participation of the representatives of community and concerned organizations. After the completion of projects, organizations seldom have the provision of maintaining coordination with the community. Besides, community could not maintain good coordination with the organizations due to inactiveness and abolition of groups and committees.

7.9 Transparency and Accountability in Project Implementation

Corruption, irregularities and nepotism in project implementation is one of the major reasons of unsustainability of sanitation projects. Project staff and community leaders involved in project management i.e. quality monitoring and decision making do not sincerely perform their duties and responsibilities due to corruption, irregularities and nepotism. And thus, quality of hardware and software components deteriorates leading to unsustainability of sanitation projects. Low profile and inexperienced staff recruited in the projects following irregularities and nepotism degrades quality of project interventions. At times, elected representatives i.e. Ward Councilors of City Corporation showing their political influence put pressure to the project management for uneven and unjust distribution of WatSan facilities. This incurs inefficient use of WatSan facilities against attainment of sustainability of sanitation projects.

7.10 Regular Repair and Maintenance of Latrines and Tubewells

Slum dwellers can not regularly repair and maintain the latrines and tubewells due to absence of sustainable mechanism in the slums. Groups and committees formed under recently implemented and ongoing different sanitation projects for repair and maintenance of latrines and tubewells were found inactive. Many of the latrines and tubewells provided by different projects in the slums were found abandoned due to absence of regular repair and maintenance. Sanitation facilities and services in the study slums were found not to be used efficiently. Community people could not use most of the community latrines and tubewells for a longer period due to lack of regular repair and maintenance. The poorly maintained facilities could not cover the slum households at optimum level.

7.11 Availability of Adequate Water for Sanitation

Availability of adequate water is very important for flushing, washing and keeping the latrines neat and clean after their every use. Some of the tubewells adjacent to the latrines in study slums were found out of order for a longer period and the slum dwellers had to use little amount of water carrying from a long distance. This makes the latrines dirty, unhygienic and unused for a long time.

7.12 Managing Space or Land for Latrines and Tubewells in the Slums

Space or land for the establishment of latrine is a very important factor in sanitation development. Slums are densely populated and have lack of space or land to establish latrines for individual household. Besides, the households are not the landowner. They live in the houses established on government khas land or Bangladesh Railway land and other private land. Due to having no permanent place of the slum households, donors and government also seldom take development program in the slum area. As Bangladesh Railway has huge amount of vacant land or annually leased out land and a significant number of city people live on the slums on Railway land, so Bangladesh Railway should think for their development. It can collaborate with the government, local community leaders and local elected representatives.

8. REASONS OF INSUSTAINABILITY AND FAILURE OF SANITATION PROJECTS IN THE SLUMS

The reasons for insustainability and failure of sanitation projects in the slums of Khulna City are as follows:

- **Absence of active and adequate community participation in sanitation projects**

Most of the sanitation hardwares of SIP and some of the sanitation hardwares of LPUPAP could not sustain. Groups and committees formed under almost all the sanitation projects were found quite inactive after their completion or in the period of nearly completion.

- **Absence of adequate and timely services of GOs and NGOs for repair and maintenance of latrines**

The GOs and NGOs can not provide adequate services in time for the repair and maintenance of the latrines and tubewells due to shortage of their skilled staff, lack of fund and logistics and even absence of developed mechanisms for rendering repair and maintenance related service to the slums.

- **Lack of community awareness on benefits of hygiene sanitation**

Community people of the slums were found not well aware on these benefits of hygiene sanitation. If the community people better know the benefits of hygiene sanitation, they feel interest to improve the sanitation status and its sustainability.

- **Poverty and poor socio-economic condition of slum people**

About 46% households of the study slums had monthly income in the range of Tk.3,000/= to Tk.6,000/=. As the slum dwellers can hardly afford food and clothing, they can not think to invest for improving their sanitation status.

▪Inappropriate sanitation technologies in the slums
About 55.5%, 20.1%, 15.2% and 9.2% households of the study slums were found to use septic tank system community latrines, twin pit water sealed latrines, simple single pit latrines and hanging latrines respectively. The existing saturated ground of the slums during high ground water level in monsoon season makes the soil wet and the simple single pit kutcha latrines fragile.

▪Lack of community knowledge on appropriate sanitation technology
Almost all the community latrines that were provided from SIP and LPUPAP needed renovation and replacement. The organizations under sanitation projects could not made the community adaptable with the technologies by providing training and guidelines.

▪Inadequate Fund, Logistics and Technical Staff of Sanitation Organizations
The projects could not sustain due to limitations and drawbacks of inadequate fund, limited number of technically sound staff, inefficient linkage and coordination among the institutions, and inadequate research initiatives on sanitation issues of the slums.

▪Lack of coordination among community and concerned organizations
After the completion of projects, both the organizations and community could not maintain good coordination due to inactiveness and abolition of groups and committees.

▪Corruption, irregularities and nepotism in project implementation
Low profile and inexperienced staff recruited in the projects following irregularities and nepotism degraded quality of project interventions.

▪Irregular repair and maintenance of latrines and tubewells
Many of the latrines and tubewells provided by different projects in the slums were found abandoned due to absence of regular repair and maintenance.

▪Inadequate and polluted water for sanitation
Some of the tubewells adjacent to the latrines in study slums were found out of order for a longer period and the slum dwellers had to use little amount of water carrying from a long distance.

▪Lack of space or land for latrines and tubewells in the slums
Due to having no permanent place or land of the slum households, donors and government seldom take development program of providing latrines and tubewels in the slum area.

8. RECOMMENDATIONS OF MORE OPTIONS FOR SUSTAINABILITY OF SANITATION PROJECTS

The means and ways of attaining sustainability of sanitation project for slums can be-active and adequate community participation in sanitation projects; increased community awareness on benefits of hygiene sanitation; reducing poverty and improving socio-economic condition of slum people to contribute significantly for establishment and repair-maintenance of WatSan facilities; selection of appropriate sanitation technologies for the slums to last for longer period; developing sustainable mechanism for regular repair and maintenance of latrines and tubewells; increasing community knowledge on appropriate sanitation technology; adequate fund, logistics and technical staff of sanitation organizations to provide better sanitation services to the slum people; developing good coordination among community and concerned organizations; ensuring transparency and accountability for efficient and beter implementation of the projects; creating adequate hugienic water options for sanitation; developing sanitation volunteers and their skill at community level; managing space or land for latrines and tubewells in the slums; and mobilizing local resources i.e. biogas generation from community latrines. Following more specific recommendation are done for attaining sustainability of sanitation projects:

- Implementation of phase out or exit strategy of the sanitation projects can be made effective just from the initiation of sanitation and related projects for attaining their sustainability. Establishment of Project Sustainability Units (PSUs) with CBO and GO-NGO representatives can be considered and strictly followed while designing and implementation of the projects.
- Mass awareness campaign, orientation, training, exchange visits etc. can be organized for all the household members of the slums to bring them in practice for using soap and ash for washing hands, use *sandel* (shoes) and use more water for clean-up while using the latrines.
- Separate space and arrangement for keeping soap and ash within the latrine can be considered while designing and installing the latrines so that household members can use soap and ash to wash their hands just after defecation.
- For better functioning and sustainability of groups and committees of sanitation projects, unmarried girls who have the very chance of going to husband's home in separate locations after marriage can not be considered as group members. Likewise, the group members must be the permanent residents of slums. The future projects can have scope of assisting the community to create a fixed place for regular meetings. Every meeting place can have option of a mini office for keeping documents related to slums and their sanitation development projects.

- Duties and responsibilities of group members as per guideline can be made clear to each member prior to formation of groups. The groups can be regulated and controlled in a disciplined way following the guideline. Performance evaluation of the groups can be done on regular basis for their better functioning.

9. CONCLUSION

Regular repair and maintenance of WatSan facilities i.e. latrines and tubewells; community mobilization and capacity development; Participatory Monitoring and Evaluation (PME); control by GOs and LEBs on sanitation and related development projects; and awareness raising of community can bring sustainability of sanitation projects. Establishment of Project Sustainability Units (PSUs) with CBO and GO-NGO representatives can be considered and strictly followed while designing and implementation of the projects. Performance evaluation of the groups formed under sanitation projects can be done on regular basis for their better functioning.

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IMPLEMENTATION OF REVERSE LOGISTIC SYSTEM AS A MEANS OF ENVIRONMENTAL AND ECONOMICAL ISSUE.

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ABSTRACT

Artery and vein is the transportation system of blood in human body as well as all vertebrate. Two directional flows are simultaneously occurred in this circulation system and for this phenomenon it is called balanced circulation with nearly zero environmental impact. An industry is like a human body and logistic deals with the transportation system. But only then the industry will be in a balance, more profitable and more environment friendly when logistic and reverse logistic play effective role consistently. Logistic has been playing a vital role to deliver the product to the customer from the early stages. But the reverse logistic system is such a system that starts its journey after the customer needs are primarily satisfied. More precisely, reverse logistics is the process of moving goods from their typical final destination for the purpose of capturing value, or proper disposal. This research is going with those products which have some nonperishable parts like beverages, battery, automobile parts, electronic devices etc. This paper work has clearly shown that the benefit of reverse logistic and the scope of recycling, remanufacturing, reusing and also the environment friendly damping system for reducing the environment pollution. A framework has done in this research by which it will helpful to design the reverse logistic system. The specific scopes of remanufacturing, recycling, reforming and the importance of damping scientifically are shown clearly in this paper. Battery manufacturing company and Beverage Company was our research field where the reverse logistic system has been proved as profitable concept.

Keywords: *Reverse logistics, Recycling, Remanufacturing, Reforming, and Green Supply Chain.*

1. INTRODUCTION

Increasing world population and standards of living have magnified resource consumption and the disposal rate. Growing concerns about climate changes, local and regional impacts of air, ground and water pollution from industrial activities have significantly expanded the interaction between environmental management and operations. This project intends to define the state of the art in reverse logistics, and to determine trends and best reverse logistics practices. Part of the research charter was to determine the extent of reverse logistics activity in the Khulna city in Bangladesh. Most of the literature examined in preparation for this research emphasized the “green” or environmental aspects of reverse logistics. In this project, green issues are discussed, but the primary focus is on economic and supply chain issues relating to reverse logistics. The objective was to determine current practices, examine those practices, and develop information surrounding trends in reverse logistics practices. This project particularly deals with battery manufacturing company where most of the elements of battery are responsible for environmental pollution intensively.

Researchers from different era and location of the world accomplished their project on this environmental safety with reverse logistics. During the last decade, reverse logistics has received increasing attention from both academic researchers and industrial practitioners. Serious and persistent environmental concerns and government regulations have created a motivation to pursue further research in this field. During the early nineties, the Council of Logistics Management published two studies on reverse logistics. First, Stock (1992) proposed the application of reverse logistics in business and society in general. One year later, Kopicki et al. (1993) elaborated the opportunities on reusing and recycling. In the late nineties, several other studies on reverse logistics were completed. Kostecki (1998) discussed marketing aspects of reuse and issues involving the extension of product life cycle. Stock (1998) investigated how to start and carry out reverse logistics programs. Rogers and Tibben-Lembke (1999) demonstrated a collection of reverse logistics business practices using a comprehensive questionnaire among US industries,

Reverse logistics studies can be divided into several categories. Dowlatshahi (2000) identified five categories as follows: global concepts of reverse logistics, quantitative models, logistics (distribution, warehousing, and transportation), company profiles, and applications. Recently, many researchers have concentrated on the optimization and quantitative models in reverse logistics. Most of the proposed models are similar to traditional facility location models, and are in the shape of a mixed integer linear program for a single period of time (Kroon and Vrijens, 1995; Ammons et al., 1997; Spengler et al., 1997; Barros et al., 1998; Marin and Pelegrin, 1998; Jayaraman et al., 1999; Krikke et al., 1999; Fleischmann et al., 2001). Other researchers studied problems with a single inbound commodity except for Spengler et al. (1997) and Jayaraman et al. (1999). Louwers et al. (1999) proposed the design of a recycling network for carpet waste. The goal of their study was to determine the locations and capacities of the regional recovery centers to minimize investment, processing, and transportation costs. They developed a nonlinear model and solved it optimally with standard software. A comprehensive review on various cases can be found in Brito et al. (2002).

We found little work addressing the environmental costs of material exchange networks. Locklear (2001) elaborated several techniques that can be applied to determine the value of environmental costs. One approach is Contingent Valuation, where external costs are based on how much the public is willing to pay for protection of the environment. Shadow Pricing is another technique, which uses existing regulations to estimate the costs that the society is willing to accept for the reduction of pollution. In 1990, Tellus Institute conducted an analysis to estimate the external costs for seven different components of air emissions including CO2 and NOx (Locklear, 2001). Their6 estimations are based on the Contingent Valuation method and have been frequently cited in the literature. According to their results, in US dollars per pound, values for CO2 and NOx are 0.012 and 3.4 respectively

2. METHODOLOGY

Reverse logistics stands for all operations related to the reuse of products and materials. It is the process of planning, implementing, and controlling the efficient, cost effective flow of raw materials, in-process inventory, finished goods and related information from the point of consumption to the point of origin for the purpose of recapturing value or proper disposal and as a result near about zero environmental effect. More precisely, reverse logistics is the process of moving goods from their typical final destination for the purpose of capturing value, or proper disposal. Remanufacturing and refurbishing activities also may be included in the definition of reverse logistics.. For instance, goods move from the customer to the distributor or to the manufacturer and recycling, remanufacturing, reforming damping are the next steps. So two types of process are aggregated here:

1. **Collection of product from customer.**
2. **Functional process.**

2.1 Collection of product from customer:

Typical reverse logistics activities would be the processes a company uses to collect used, damaged, unwanted (stock balancing returns), or outdated products, as well as packaging and shipping materials from the end-user or the reseller. Once a product has been returned to a company, the firm has many disposal options from which to choose. When a product is getting damage then customers never put their attention on it rather throws into drain. So collection is the base of the reverse logistic and analyzing this transportation network design of supply chain is initiated. Among all the types of transportation model TAILORED NETWORK is widely used here which represents shortly ‘Situation demand networking system’. It is the combination of different network system where there is a well combination of supplier, customers, Distribution Centre, Cross-docking and milk run approach. The main advantage of this network is reducing the cost, time and improving the responsiveness. And also this is the perfect network for collecting product reversely.

2.1.1 *Reverse Logistics and Environmental Logistics*

While promoting economic development, logistics activities also increase the quantity and frequency of related activities, increase energy consumption, aggravate air pollution and waste pollution, and bring many side-effects on economic sustainable development of the whole society. The concept of environmental logistics can be described as follows: to keep a friendly environment and make good use of logistics resources, restrict the side effects on environment of logistics during operating process is very important for us to develop environmental logistics. By improving logistics stages such as transport, storage, package, load and unload as well as distributing process, the objective of reducing environment pollution and energy consumption can be attained. Reverse logistics, as an organic part of environmental logistics, plays an important role to some extent. It can make good use of existing resources, cut down the demand on raw materials and save operating costs. However, some activities related to reverse logistics may put many side-effects on environment. Even they can destroy the positive effects brought about by recycling, remanufacturing and the reuse of new products and materials, etc. Therefore, in addition to concentrating on the saving of costs, enterprises should carefully deal with the relation between reverse logistics and environmental protection, meanwhile, strengthen research on the techniques related to reverse logistics, only by those can the side-effects on environment resulted from additional logistics activities be reduced.

2.2 Functional process:

If the product can be returned to the supplier for a full refund, the firm may choose this option first. If the product has not been used, it may be resold to a different customer, or it may be sold through an outlet store. If it is not of sufficient quality to be sold through either of these options, it may be sold to a salvage company that will export the product to a foreign market. If the product cannot be sold “as is,” or if the firm can significantly increase the selling price by reconditioning, refurbishing or remanufacturing the product, the firm may perform these activities before selling the product. If the firm does not perform these activities in-house, a third party firm may be contracted, or the product can be sold outright to a reconditioning/remanufacturing/refurbishing firm. After performing these activities, the product may be sold as a reconditioned or remanufactured product, but not as new. If the product cannot be reconditioned in any way, because of its poor condition, legal implications, or environmental restrictions, the firm will try to dispose of the product for the least cost. Any valuable materials that can be reclaimed will be reclaimed, and any other recyclable materials will be removed before the remainder is finally sent to a landfill. Generally, packaging materials returned to a firm will be reused. Clearly, reusable totes and pallets will be used many times before disposal. Often, damaged totes and pallets can be refurbished and returned to use. This work may be done in-house, or using companies whose sole mission is to fix broken pallets and refurbish packaging. Once repairs can no longer be made, the reusable transport packaging must be disposed of. However, before it is sent to a landfill, all salvageable materials will be reclaimed. So this system ensures us a controlled damping system where pollution is in environmental manner. And functional process start for the used product by breaking down as only the primary objective then other respected operation. After this related parallel operations like recycling, remanufacturing, reforming and damping are functioned.

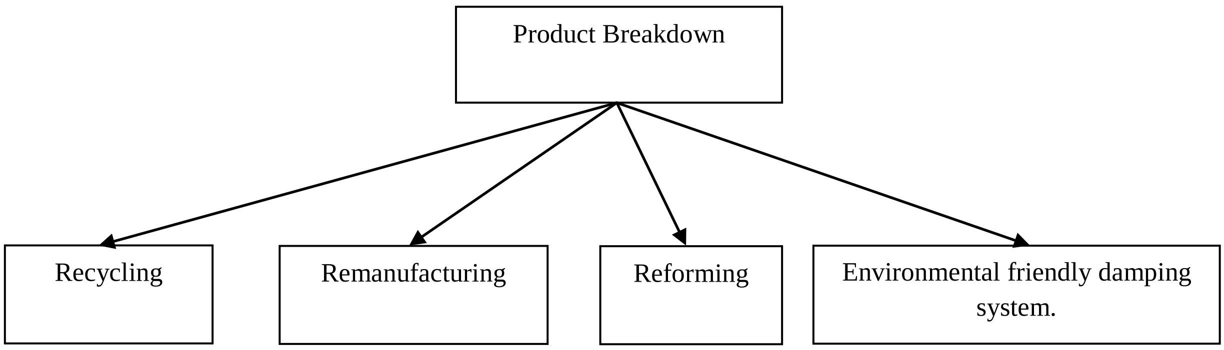

Fig 1: Functional process overview.

3. ILLUSTRATIONS

3.1 Findings of the study

Battery manufacturing company was our research field for this calculative project where maximum raw materials are collected from vendor. And most of the time they collect a reasonable percentage of raw materials from the local party who are the middleman between customer and manufacturer. A very negligible portion of some raw material is come from the company recycling plant which is really insufficient. For the purpose of this research the total logistic system of the company is observed well from the top to bottom. A flow diagram is depicted where the directional flow of product is viewed. The function of Logistic in the battery manufacturing company;

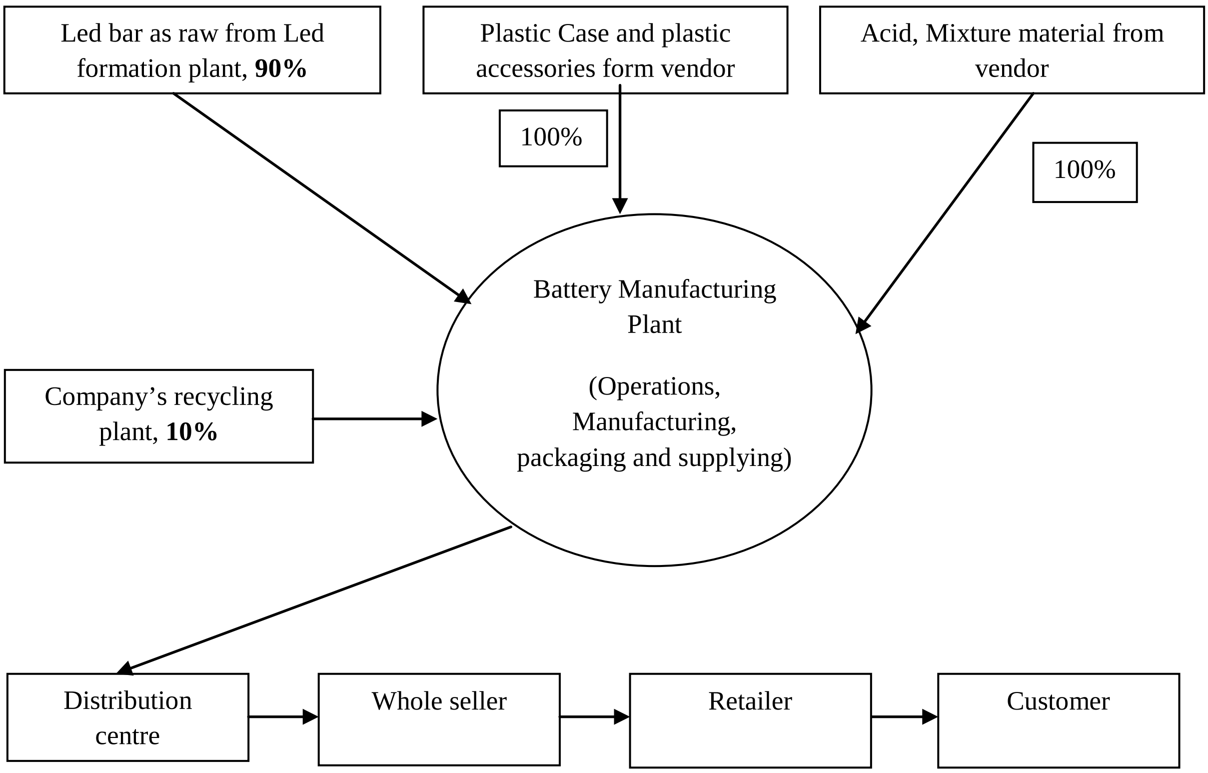

Fig 2 : Flow direction of logistic

The company offers one year warranty for the product and the approximate life cycle is 5 years. After the damage of the battery naturally is t is sold to near local party who are illegally involve in this business. They never follow the scientific approach of the recycling, damping activities. But they sell the led to the led processing party or rarely to the battery manufacturing company. So after life cycle the flow of the battery is:

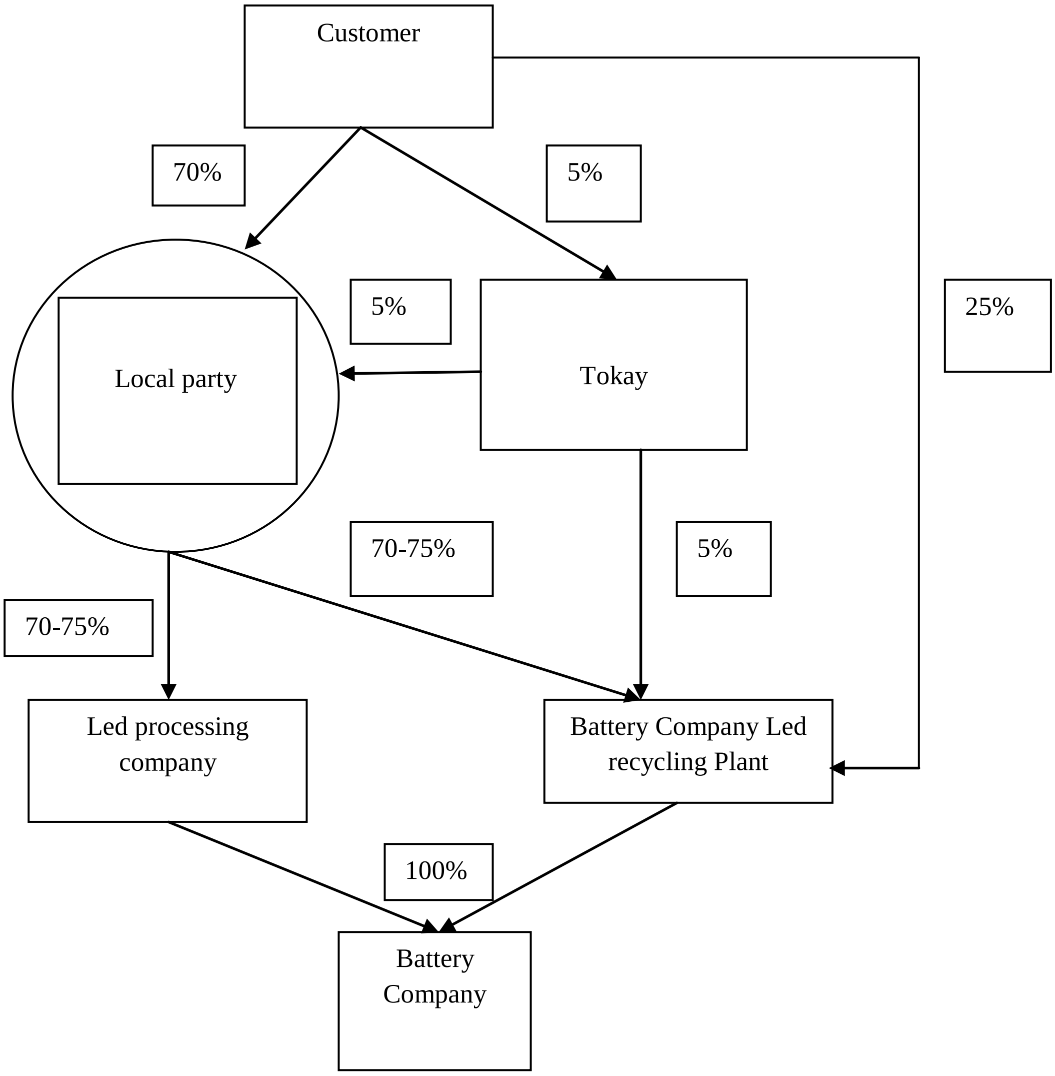

Fig 3: Flow chart of a used battery after life cycle.

3.2 Improvements

Reverse logistic is one and only the way of controlling the scenario of random breaking of battery which is the main cause of environment pollution. A newly design product flow with reverse logistic can solve this problem entirely. Managing reverse logistic flow is the main concern for this purpose. A few initiatives can make the system easy and efficient like:

3.2 .1 *Standardization of Processes*

One of the most common difficulties is observed with current reverse logistics systems is the lack of standardization of processes throughout an organization. If processes are not standardized, it is very difficult for people in an organization to communicate to each other how to handle reverse logistics problems.

3.2 .2 Centralized Return Centers

Centralized Return Center (CRC) is must be in agreement that using a separate CRC offered many benefits. This approach is must help the user to attract to perform their activities in association with reverse logistic system.

3.2 .3 Secondary Markets

Management has to move secondary market for benefit of the company. The material which is not useable for quality product is must move to secondary market.

3.2 .4 Web-Based Secondary Markets

A trend in the disposition of goods is the utilization of the worldwide web. It appears that in the future it will be an important mechanism for dispositioning from the reverse logistics flow. The web provides a direct link to consumers.

3.2 .5 Enrich environmental safety consensuses:

Adequate initiative for built national consensus on environmental safety must ensure. This activity definitely motivate user to move their useless product to CRC.

3.2 .6 Design TAILORED NETWORK:

With respect to the cost a transportation network must to design. In this research work TAILORED network is preferred for the low cost service. The transportation mode using in time of logistic is used for reverse logistic and only for this no extra transportation cost is needed. However, after considering aforementioned condition a ne flow diagram is proposed whish can make the system effective and also there is a lower transportation cost. So this is stood for an advanced collection process of product from customer.

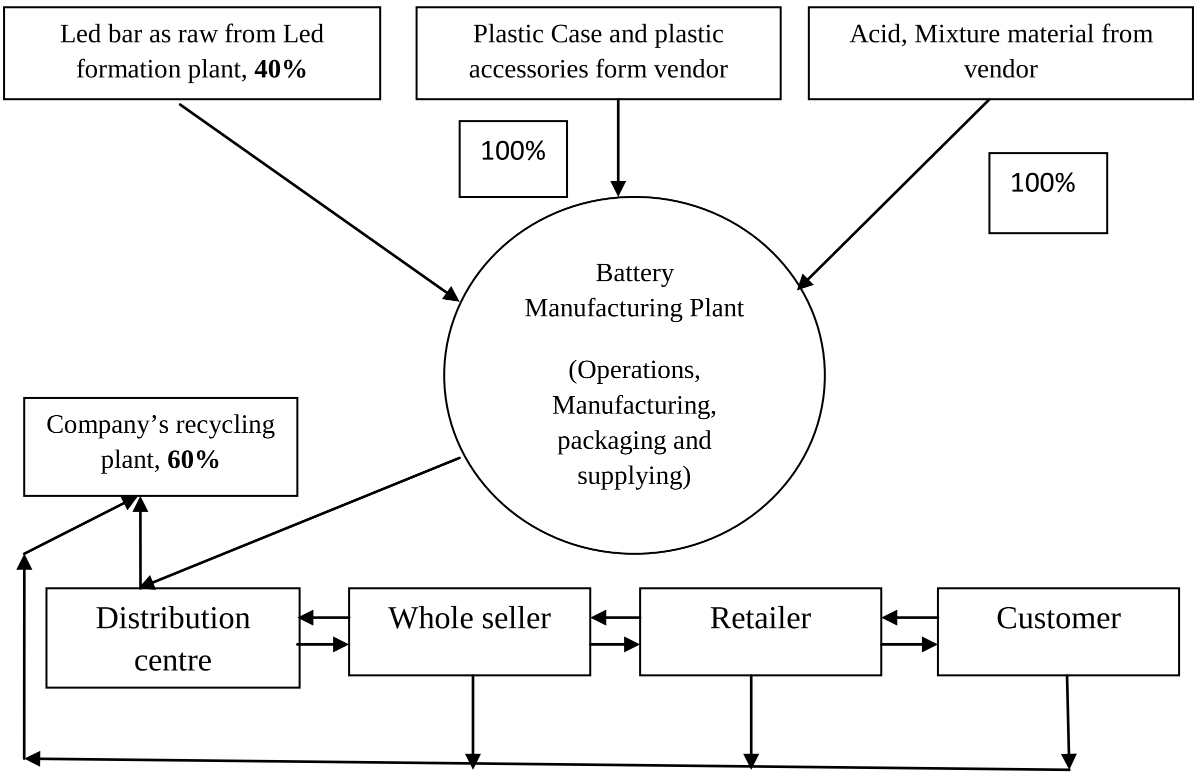

Fig 4: Product with reverse logistic

At the end of collection process battery is separating is the core element and then decision has to take which element are going to recycle, remanufacturing, reusing and damping. A percentage calculation of a battery is given below:

Table 1: Battery Content (after break down)

Name of element	**Percentage (%)**	**Required operation**	**Use**
Grid lead	26.70%	Recycling/Remanufacturing	75%-85%
Active material	36.0%	Reuse	100%
Top Lead	10.60%	Recycle	90%
Plastic/other	10.60%	Reuse	100%
Electrolyte	27.50%	Reuse	90%

3.2 .7 Recycling:

Recycling plant is needed mandatorily in any battery manufacturing company for purpose of both environmental safety and cost. In a regional calculation of this battery company it is clear that about 70% raw materials can be collected from a well designed recycling plant. At the same time a fixed recycling plant must reduce environmental safety. Led grid and plastic Casing are the main element of recycling.

3.2 .8 Remanufacturing:

Remanufacturing is the approach of reducing waste and making profit. There are some parts like some special type of plastic casing, cap and some lead grid which are useable for two or more times of a total battery. These elements can be used in repetitive manner which save money and must be for environmental friendly.

3.2 .9 Reusing:

Reusing concept is totally new and effective for reducing waste. Those items which are totally useless can be used in making show piece. Some plastic item those are not useable may be sold in secondary market or in any parallel business. Raw degradable plastic can be used in making plastic goods. This approach helps to open a new dimension of business.

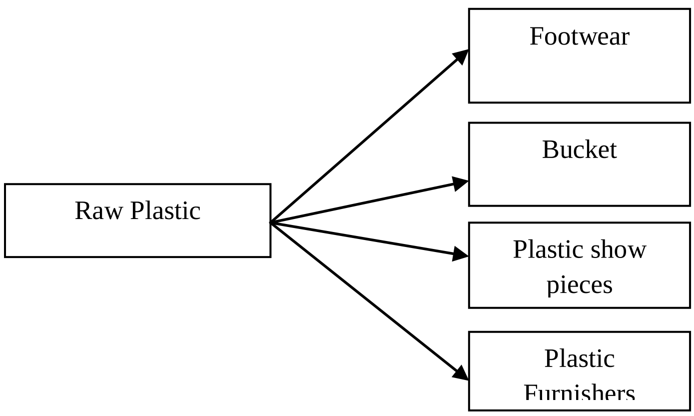

Fig 5: Reusing of raw plastic.

***3.2 .10 Damping*:**

Environment has deep relation in damping system by which it may be safe or polluted. So a scientific damping system must be designed as the part of environment treatment plant. Waste acid substance must be neutralized before draining. Buffer solution can be used in this purpose.

4. CONCLUSION

In Battery manufacturing company Reverse Logistic system is found as profitable policy. The Battery Company has benefitted because of reverse logistic. The company has now producing IPS, remake new batteries as a new product by reusing the led plate and accessories of collecting from used battery and has been able to attain 10% more benefitted. The toxic acid has neutralized scientifically and the lead is recycled so that environmental effect has reduced. This system has ensured the nearly optimum waste which is directly saved our land as well as environment. Also the rate of water pollution due to toxic acid is limited. The developed shortlist will help to find out the scope of reverse logistic system. The era of recycling, remanufacturing and reforming has opened only for reverse logistic system. This system also helps to attain environment friendly surroundings. Reverse logistic system is such a gateway which is functioning in recycling, remanufacturing, reforming. This paper has clearly described about to design a reverse logistic system in supply chain to make it a more profitable sector. This framework has helped to find out the necessity of reverse logistic in the sector of green supply chain for the environmental safety. Although our resources are limited, but reverse logistic can make this limited resource unlimited by helping in recycling, remanufacturing, reforming sectors. In every battery company one or two year guarantee is provided as after sale service but the fact is that there is no policy after two years or more when the battery may get dam. Reverse logistic is such a change maker of this damage device. By collecting those batteries remanufacturing, recycling and reusing concept can be applied. Moreover to damp any part of the battery an environment friendly scientific damping system is followed. Thus the company gets benefitted and environment is getting free from mass pollution. In that way Beverage Companies, Toiletries Companies, Electronic Company etc can recollect their non perishable bottle, tube, packet etc and can use those as in recycling raw material for the sake of environmental safety including profit. The proposed design of reverse logistic system in this paper has shown profitability of the company.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SUSTAINABLE DEVELOPMENT IN CONSTRUCTION SECTORS

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ABSTRACT

Recent global attention to the issues and challenges of sustainable development is forcing construction sectors to conduct self-assessments to identify where they stand within the framework for sustainability, and more importantly, to identify drivers, opportunities, strategies and technologies that support achieving this goal. The principal objectives of this paper are to present a brief overview of an overall framework for sustainability and then to discuss the implications for the construction industries. The call and desire for sustainable construction is in realization of the construction industry's capacity to make a significant contribution to environmental sustainability. The paper is mainly concentrated on sustainable construction keeping in view of economical, social and environmental aspects. This study also highlights the development and implementation tools of construction sustainability. Related to that, the paper includes in depth analysis of sustainability of concrete as well as other sustainable materials. The paper has also consented on the implementation of sustainable construction in context of Bangladesh.

Keywords: *Sustainable development; construction sector; communication infrastructures; sustainable construction*

1. INTRODUCTION

The development path that we have been taking, in the past few centuries, has been ultimately detrimental to the health of our surrounding ecological context. We are consuming an increasing share of the natural resources available to us on this planet, and we are creating sufficiently large amounts of waste and pollution such that the earth can no longer assimilate our wastes and recover from the negative impacts. This is a result of a growing population as well as new technologies which make it easier for us to access natural. Strategies, technologies, and opportunities are presented to improve the sustainability of the built environment.

The construction sector, relative to its level of development, accounts for more than 60 percent of gross capital formation in most countries, and defines the physical infrastructure upon which effective growth and development is achieved. Construction activities extend beyond the erection of houses, hospitals, schools, offices and factories to civil engineering works such as roads, bridges and communication infrastructure. In fulfilling these roles, the construction industry exerts enormous demand pressures on global natural resources. The environmental significance of such pressures comes into play when some of these resources are non-renewable, bringing the construction industry in direct conflict with the physical environment. The call and desire for sustainable construction is in realization of the construction industry's capacity to make a significant contribution to environmental sustainability because of the enormous demands it exerts on global resources. To facilitate sustainability in construction, available techniques need to be moulded and adapted as per time, need and locality. Thus, the concept of sustainable construction is a direct response to the continuing rise in global resource consumption and the attendant deterioration in the global physical and biotic environment. However, just as the concept of environmental sustainability is still unfolding as knowledge about the environment improves, so is the understanding of sustainable construction as a concept, which now extends beyond the fabric of the built environment. Thus, the concept of sustainable construction now transcends environmental sustainability, embracing economic, technical and social sustainability, which emphasizes possible value, added to the quality of life of individuals and communities. This we can call 'Engineering Sustainability'.

Generally, responses to global environmental deterioration vary greatly, and so is the case with sustainable construction. A marked distinction exists between the developed countries and their developing counterparts in their ability to deal with environmental problems. This ability is defined by their capacity to initiate and implement effective environmental policies. However, there is a huge gap between these blocks of countries with regards to technical and financial capacities, which are crucial to the successful rearrangement of economic and social relations as applied to production and consumption.

2. OBJECTIVES AND SCOPES

The main objectives of this paper are:

- To study the basic concept of sustainability in the realm of construction sector and to clarify the conceptual differences between gross environmental sustainability and construction sustainability from engineering point of view.
- To explore key elements of sustainability in construction i.e. economical, social and environmental as well as technical.
- To evaluate the principles to be implemented so far as sustainability in construction is concerned and also to find out some strategies for such implementation.
- To propose some relevant structural considerations helpful in rural constructions in Bangladesh, especially for the areas prone to natural disasters.

The paper is mainly concentrated on sustainable construction keeping in view of economical, social and environmental aspects. This study also highlights the development and implementation tools of construction sustainability.

3. WHAT IS SUSTAINABLE CONSTRUCTION

Sustainable construction can be defined as a construction process which incorporates the basic themes of sustainable development. Such construction processes would thus bring environmental responsibility, social awareness, and economic profitability objectives to the fore in the built environment and facilities for the wider community. There are three components of sustainability: environment, economy and society. To meet its goal, sustainable development must provide that these three components remain healthy and balanced. Furthermore, it must do so simultaneously and throughout the entire planet, both now and in the future. At the moment, the environment is probably the most important component, and an engineer or architect uses sustainability to mean having no net negative impact on the environment.

4. KEY FEATURES OF SUSTAINABLE CONSTRUCTION

Sustainable development offers a new way of thinking which reconciles the human drive to improve our quality of life with the limitations imposed on us by our global context. It requires unique solutions for improving our welfare that do not come at the cost of degrading the environment or impinging on the well-being of other people. Although there is no general agreement regarding the precise meaning of sustainability, beyond respect for the quality of life for future generations, most interpretations and definitions of the term "sustainable" refer to the viability of natural resources and ecosystems over time, and to the maintenance of human living standards and economic development.

4.1 Social Sustainability

Sustainability is inherently anthropocentric, since it is the welfare of humans with which we are concerned. More than a concern for mere survival, sustainability is a desire to thrive, to have the best life possible. There are many socio-cultural issues which influence sustainability. The most prominent issue is inter-generational equity, in which we must ensure that we leave our progeny with the tools and resources they need to survive and enjoy life. In addition, we should not forsake the quality of life that people today are experiencing. Instead, we must strive to raise the standard of living of those people who today lack the most basic requirements such as clean water and adequate food. Other issues in this realm are environmental justice, population growth, human health, cultural needs, and personal preferences. These elements have a great deal to do with our quality of life and should not be ignored at any cost.

4.2 Environmental Sustainability

Environmental concerns are also very important for sustainability. The natural environment is the physical context within which we live. Sustainability requires that we recognize the limits of our environment. There are limits to the quantities of natural resources that exist on the planet. Some of these resources, such as trees and wildlife, are renewable so long as we leave enough intact to regenerate. Other resources, such as minerals, are renewed at such slow rates that any use whatsoever depletes the total stock. We need to minimize our consumption of all resources, renewable and delectable. Another key environmental issue is to minimize our impact on global ecosystems: the earth is like an organism and we must maintain it in a healthy state. Natural ecosystems can survive some impacts, but these must be small enough so that the earth can recover. In some cases there are particular resources or elements of an ecosystem which are essential to its health. For example, enough timber must be provided for future generations. Protecting ecosystem health may involve the protection of an endangered species, the preservation of a wetland or protection of biodiversity in general. The environmental component has our attention now because deterioration of our environment is driving the current worldwide focus on sustainable development. Probably the most troubling for the long-tern health of the planet and for the goal of sustainability are the climate changes resulting from the thinning of the ozone layer and the progressive decline in biodiversity resulting from loss of habitat. Both of these changes are a direct result of human development.

4.3 Economic Sustainability

Economics, as it pertains to sustainability, does not simply refer to Gross National Product, exchange rates, inflation, profit etc. Economics is important to sustainability because of its broader meaning as a social science that explains the production, distribution and consumption of goods and services. The exchange of goods and services has a significant impact on the environment, since the environment serves as the ultimate source of raw material inputs and the repository for discarded goods. Economic gain has been the driver for much of the unsustainable development that has occurred in the past. A shift to sustainability will only occur if it is shown not to be excessively costly and disadvantageous. Part of sustainability is changing the way things are valued to take into consideration the economic losses due to lost or degraded natural resources. Once this is done sustainable development will be revealed to be a more economically beneficial option than current development patterns. The economic component is given less attention in the developed countries of the world, but is equally essential to the goal of sustainable development. There is poverty throughout the planet, and the global inequities in consumption of resources are staggering. Economic sustainability and environmental sustainability are closely linked. Much environmental degradation occurs when people are struggling to obtain the resources essential for life (food, water, shelter, etc), and it is inevitable that the basic economic struggle may take precedence over environmental sustainability. Conversely, environmental deterioration exacerbates economic inequity, for example diseases associated with lack of clean water are a significant cause of poverty.

5. ABOUT SUSTAINABLE DEVELOPMENT

There are many definitions of sustainable development. Sustainable development refers to the fulfillment of human needs through simultaneous socio-economic and technological progress and conservation of the earth's natural systems (Sage, 19999). Sustainable world progress is dependent upon continued economic, social, cultural, and technological progress. To achieve this, careful attention must also be paid to the preservation of the earth’s natural resources. Sustainable development is a term generally associated with the achievement of increased techno-economic growth coupled with the preservation of the natural capital that is comprised of environmental and natural resources.

Sustainable development is about ensuring a better quality of life for everyone, now and for generations to come, through: **(DETR, 2000)**

- Social progress which recognizes the needs of everyone;
- Effective protection of the environment;
- Prudent use of natural resources;
- Maintenance of high and stable levels of economic growth and employment.

There is a more commonly used definition for sustainable development, which was formulated by the World Commission on Environment and Development (WCED), led by the Norwegian Prime Minister Gro Harlem Brundtland, in 1983 that is sustainable development is development which meets the needs of the present without compromising the ability of future generations to meet their own needs (Chaharbaghi & Willis, 1999). Sustainable development includes three broad components; social, environmental, and economics as shown in figure 1. However, without appropriate technological adaptation, sustainable development is not sure to take place inside construction field.

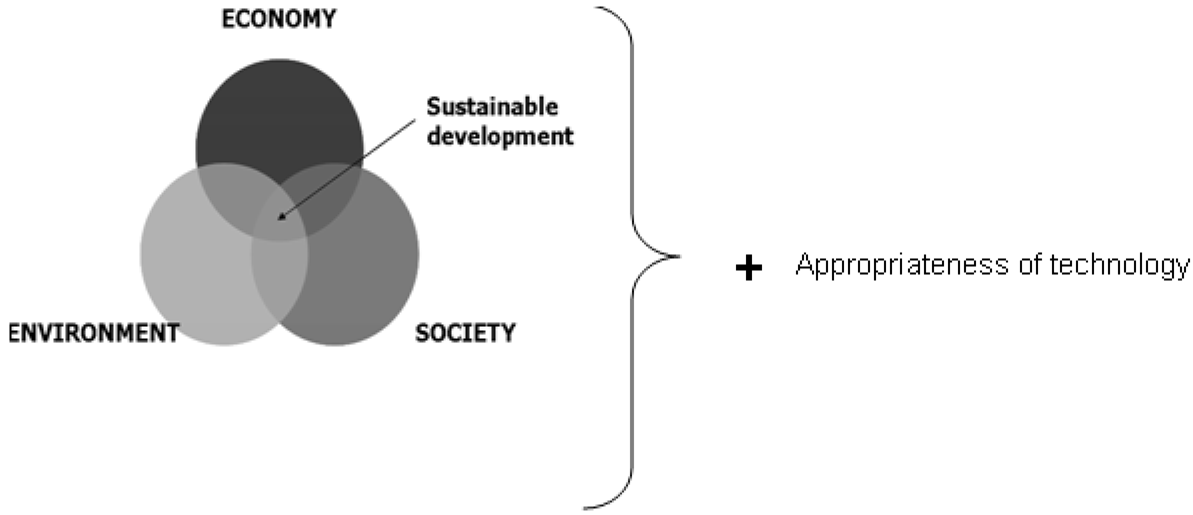


Figure 1: Themes of sustainable development in construction

6. SUSTAINABILITY OF CONSTRUCTION MATERIALS

6.1 Concrete

Concrete is manufactured from aggregates, cement and water. It usually contains a small amount of some chemical admixture also. In order to assess the environmental impact of concrete manufacture, it is necessary to consider the impact of each separate constituent.

6.1.1 Energy consumption related to concrete ingredients

Aggregates may be crushed and washed. They are usually separated into various size fractions and reconstituted so as to satisfy the grading requirements. A modest amount of energy is involved in all these processes.
Portland cement is usually manufactured by heating a mixture of limestone and shale in a kiln to a high temperature (approximately 1500°C), then intergrading the resulting clinker with gypsum to form a fine powder. The reaction between limestone and shale to produce clinker produces CO_2. Furthermore, the fuel used in the kiln and the electricity in the grinding mills themselves produces some amount of gaseous waste, principally CO_2 and CO. These gases are non toxic and are released to the atmosphere, where they contribute to global warming.
The water in concrete is normally ordinary tap water with no further processing. Thus it has very little embodied energy and no waste. It is only an environmental issue in locations where the water is already not sufficient for basic needs. Concrete is usually manufactured by combining and mixing these constituents in large batches in a ready-mixed concrete plant and hauling the mixture to the construction site in a truck. These processes (moving materials, mixing them, and hauling the concrete) require modest amounts of energy and produce small amounts of waste.

6.1.2 Impact on sustainability relayed to concrete works

The environmental impact of using concrete at a construction site is basically similar to the impact of manufacturing concrete in ready-mixed concrete plant. The concrete is moved to its desired location, consolidated into the formwork and finished. After the concrete has set and gained some strength, the formwork is typically removed. These are all low-energy operations. Waste includes unused concrete, contaminated wash water and used formwork. Formwork may be wood, which must be disposed in a landfill, but sometimes it is steel and can be reused. The impact of concrete on sustainability during the lifetime of the structure is primarily a function of its role in energy transmittance (i.e., its insulating properties) and its role in energy storage. Concrete is not an especially good heat conductor, not as good as steel, for example. It is also not an especially good insulator, not as good as wood, for example. A very high porosity is necessary to provide good insulating properties and concrete has less porosity than wood. On the other hand, concrete provides a large thermal mass so it can store energy and release it later. At the end of its service life, a concrete structure must be demolished and disposed. The demolition process is done by brute force. Concrete is sometimes recycled, most commonly used as rock in a pavement sub base.

7. PRINCIPLES OF IMPLEMENTATION FOR CONSTRUCTION SUSTAINABILITY

Defects and inefficient processes are expensive forms of wasting environmental resources and pose a danger to both construction workers and the end-users of the product. Badly performing construction products also reduce the quality of life of those using these products. One of the most important steps towards sustainable construction is to improve the quality of construction products and the efficiency and safety of the construction process. It is a priority for the construction sector to reduce its use of resources. Areas of improving the quality of construction processes and products include (Plessis, 2002):

7.1 Reduction of building material wastage

Reducing material wastage helps to reduce global material consumption, the amount of construction waste and, in the long term, the amount of demolition waste. It also reduces construction costs, making houses more affordable. Management, design and cultural practices have a great influence on the wastage rates. These can be changed through education, site planning, management and design practices, as well as the use of new technologies.

7.2 Increasing the use of recycled waste as building materials

Recycling waste as building materials is a convenient way to reduce the environmental impact of the construction industry. Recycling helps to reduce the consumption of natural resources, deposition of landfill and energy consumption of material production and all its associated pollution.

7.3 Energy efficiency in buildings

This can be achieved through cutting down both consumption and embodied energy. Reducing energy consumption can be accomplished through education, avoiding unnecessary improvement of systems (air-conditioning, heating, water heating), improvement of insulation, use of alternative energy sources and passive solar design improvements. Consumption can also be reduced through the redesign of appliances such as water heaters and lighting sources.

7.4 Water conservation

A combination of user education and design and technical changes is required. Improved water metering systems, rainwater harvesting systems, re-using water, waterless technologies and low-flow, aerated and self-closing faucets are suggested. The use of water on construction sites and in the production of materials also needs to be reassessed.

7.5 Durability and maintenance

Increasing knowledge on the service life of the built environment and the capability of generating and managing life data are certainly a major challenge for achieving a more sustainable construction industry. This should include physical and functional durability of the constructed asset including the optimization of the service life in all phases of the building process. It also requires that building flexibility and capacity be upgraded. Maintenance also needs to be considered during design and life-cycle costs can be used to select more competitive technologies.

7.6 Innovation in building materials and methods

An entirely new design paradigm is required and new materials should be low cost, environmental friendly but without compromising the strength.

7.7 Avoiding negative environmental impacts

Built facilities impact the natural environment in many ways over their entire life cycles. Four categories of impacts which built facilities have on the earth's ecological systems and resource are (Yeang,1995) :

- Spatial displacement of natural ecosystems and modification of surrounding ecosystems as a result
- Impacts resulting by human use of the built environment and the tendency for that use to spur further human development of the surrounding ecosystems
- Depletion of matter and energy resources from natural ecosystems during the construction and use of the facility
- Generation of large amounts of waste output over the whole life cycle of the facility, which is deposited in and must be absorbed by natural ecosystems.

Given their large scale and long life cycles, built facilities have particularly large and long-lasting effects on the environment as a whole.

8. STRATEGIES FOR IMPLEMENTING SUSTAINABLE CONSTRUCTION

Three general objectives can shape the implementation of sustainable design and construction, while keeping in mind the three categories of sustainability issues mentioned earlier (social, environmental and economics). These objectives are:

- Minimizing consumption of matter and energy over the whole life cycle of consumption, while
- Satisfying human needs and aspirations with sensitivity to cultural context, and
- Avoiding negative environmental impact.

In the following subsections, specific strategies for approaching each of the three objectives are given in detail:

8.1 Shared responsibility

Sustainability is not the sole responsibility of governments and the construction industry. Citizens need to get involved and be aware of the impacts of their behaviours and their use and misuse of resources. Individual participation of people is the key to achieving decisions needed to secure changes in the consumption patterns of the majority of the population. It is important to develop campaigns that on the one hand inform the public regarding the benefits and opportunities of the use of environmentally friendly building materials and products and, on the other, encourage a change in consumer habits towards a more sustainable use of resources. A general interest is required in the issue of sustainability by the construction sector, its clients and other stakeholders. This should be propagated differently through the various stakeholders (Plessis, 2002):

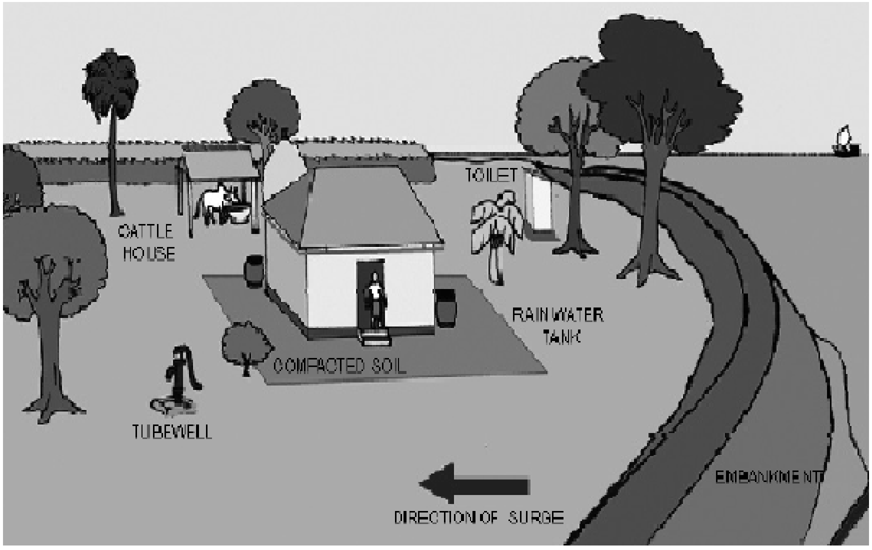
- Politicians: Sustainability issues should actively appear on the agenda of political leadership.
- Manufacturers: Environmental responsiveness should constitute a criterion in materials or product specifications.
- Local authorities: Sustainability should constitute a criterion or drive requirements for plan approvals, land use or land sub-division.
- Built environment professionals: Sustainability considerations should make it to the brief, design criteria or specifications. Related training institutions have a key role to play for training in these issues.

8.2 Improving the capacity of construction sector

One of the most critical barriers to sustainable construction is the lack of capacity of the construction sector to actually implement sustainable practices. This lack of capacity is a factor both of the number of human resources and the skill levels of these resources. There simply are not enough professionals, tradesmen and laborers who have been trained to support sustainable construction. In fact, the capacity of the construction sector in many developing countries can barely deal with the demands of routine construction. The vast majority of construction firms are small enterprises that rely on outsourcing personnel as required. This has severely affected skills training and the retention of expertise in the industry as construction workers become highly mobile, walking in and out of the industry, depending on performance in other sectors of the economy.

9. BUILDING SUSTAINABLE CONSTRUCTION IN COASTAL AREA OF BANGLADESH

- The havocs caused by cyclone Sidr indicate that existing housing practice is vulnerable to cyclone and it needs improvement. But due to socio-economic parameters it is not possible to make any radical change in present housing practice. Any new approach conflicting with ethnic cultural values will not be welcomed by local people. So, the strategic changes have to be within the society and also have to comply with cultural environment. In this article indigenous technologies are supported with appropriate modification. The existing house construction techniques need improvement in almost each section.
- Before constructing the house, the location of the house should be checked. Many of the landless people have a tendency to build houses on the embankment or on the outside slope of the embankment. This is very dangerous because tidal surge will inundate them easily. House should be constructed inside of an embankment.



Some considerations:

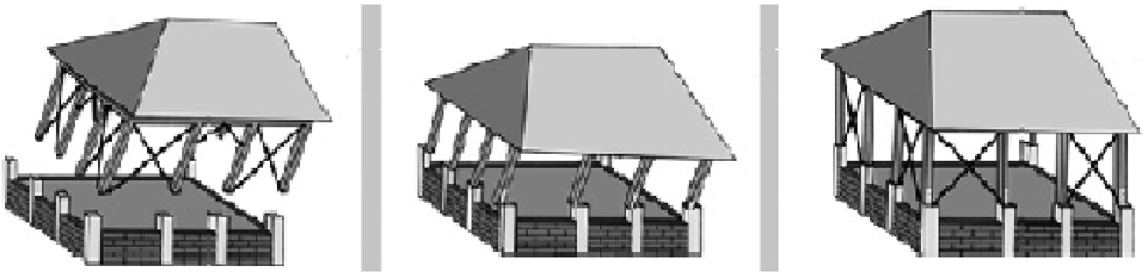
Shorter side of house should be parallel to the coast of the sea or river-bank

Tubewell should be atleast 30′ from latrine or cattle-house

Rainwater collection is necessary as it is helpful during emergency

Figure 2: Overall Housing plan for Coastal Area

- The house has to be erected over a well prepared plinth. 5 inch by 10 inch brick wall along the periphery is necessary. The soil of the plinth needs to be compacted too. The height of the plinth should be be 2 ft to 3 ft.
- For structural framing, timber or concrete posts are recommended. The available research materials for rural housing primarily deal with bamboo posts. But now, bamboo posts are being replaced by timber posts. On the other hand, the concrete posts have to be pre-cast; otherwise concrete quality control will be difficult in saline environment. But cast-in-situ concrete columns can also be used depending on availability of non-saline water. Concrete quality has to be controlled strictly. In saline environment, to reduce corrosion concrete has to be dense for reducing corrosion so that water cannot penetrate inside.
- For building wind resistant house, bracing is of utmost importance. Introduction of vertical cross braces around the basic frames improves both lateral and torsional stiffness. It was also observed that lateral resistance does not improve at all by horizontal bracings at the top. A house with proper cross bracings is most likely to overcome a strong storm.



(a) (b) (c)

Figure 3: (a) improper Anchorage (b) No bracing (c) Proper bracing and anchorage

- It as been observed that, after every cyclone, flood etc., there is a sudden shortage of safe drinking water. It has been going on for ages. Therefore, improved sustainable solutions should be looked out for providing safe drinking water on the aftermath of a disaster. use simple practices for rainwater harvesting. One of the simplest of techniques involve inclined water channels provided around the perimeter of the roof along with a storage tank right beneath the lowest point of the channel.

- For community buildings it has to keep in mind that any building constructed in coastal areas will eventually function as a cyclone shelter. So, these structures will be designed and constructed considering essential structures. Load bearing wall building or the masonry building should be avoided. The design of the frame structure has to conform with BNBC, 1993. Construction material has to be chosen very carefully considering the marine environment.

10. CONCLUSIONS

In striving to achieve sustainability in the built environment, three themes emerge. Firstly, awareness of the impacts that built facilities have on both human and natural systems is essential, and should be considered as early as possible in the planning and design of any built facility. Secondly, the ecological, social, and economic contexts of the facility must be taken into account for all project decision making. Finally, sustainable designers and constructors must be aware of the connectivity of human systems to the natural environment. No human action can take place without affecting the ecological context in which it occurs. All human activity must be undertaken with an awareness of the potential consequences to other humans and nature, especially the construction of built facilities. In this paper it is tried to provide a brief overview of the wide range of technological issues at an industry level, while emphasizing the need for an integrated approach and understanding of the different components of a sustainable system. In order to achieve sustainability for society as a whole and for construction in particular, intelligent decision making is required which includes full consideration and knowledge of the many trade-offs and impacts associated with each alternative available to be chosen. Sustainability is a desirable state towards which to strive.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ERRORS INVOLVED IN THE ESTIMATION OF LEACHATE POLLUTION INDEX OF SOLID WASTE LANDFILL IN BNGLADESH

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ABSTRACT

An important part of maintaining a solid waste landfill is managing the leachate through proper treatment methods designed to prevent pollution into surrounding ground and surface waters. Any assessment of the potential impact of a landfill on groundwater quality requires consideration of the components of the leachate most likely to causze an envionental impact as well as the source of concentration of those components. Leachate pollution index (LPI)is an environmental index used to quantify and compare the leachate contamination potential of solid waste landfill. This index is based on the concentration of 18 pollutants of the leachate and their corresponding significance. That means, for calculating the LPI of a landfill, concentration of these 18 parameters are to be known. However, sometimes the data for all the 18 pollutants included in the LPI may not be available to calculate the LPI. In this studyr, the possible errors involved in calculating the LPI due to the nonavailability of data are reported by the author. The leachate characteristic data for solid waste landfill at Chittagong in Bangladesh have been used to estimate these errors. Based on this study, it can be concluded that the errors may be high if the data for the pollutants having significantly high or low concentration are not available. However, LPI can be reported with a marginal error if the concentrations of the nonavailable pollutants are not completely biased.

Keywords: *Landfill, leachate, sub-index value, pollutant weight, error analysis, lechate pollution index*

1. INTRODUCTION

Landfill leachate is liquid that moves through or drains from a solid waste landfill. Leachate is the main medium for the contaminants transportation from the landfill to groundwater and surface water (Rowe 1995). Landfill leachate is formed from the infiltration and passage of water through solid waste which results in a combination of physical, chemical and microbial processes that transfers pollutants from waste materials to the water (Jasper et al. 1985; Kjeldsen et al. 2002).

The most common source of landfill leachate is rainwater filtering down through the landfill and aiding bacteria in the process of decomposition (Kelley 1976). Modern landfills are often designed to prevent liquid from leaching out and entering the environment; however, if not properly managed, the leachate is at risk for mixing with groundwater near the site, which can have terrible effects (Chian and DeWalle 1976).

Leachate can consist of many different organic and inorganic compounds that are typically either dissolved or suspended in the wastewater (Christensen and Kjeldsen 1995) .The leachate may be virtually harmless or dangerously toxic, depending upon what is in the landfill (Leckie et al. 1979; Kouzeli-Katsiri et al. 1999). Typically, landfill leachate has high concentrations chemical oxygen demand (COD) associated, BOD, nitrogen, phenols, pesticides, chloride, solvents and heavy metals are common in these systems (Lo 1996). As leachate liquid emerges from a landfill site, it is often black or yellow, with a strong acidic smell.

The potential long term environmental impact of a landfill on groundwater quality will depend on the leachate charecteristics, the mass of contaminant in the facility, the percolation of fluid through the waste, the nature of the leachate containment system and the site hydrology (Henry and Heinke 1996; Farquhar and Rovers 1973). Leachate characterstics may be expected to evolve over time, increasing from initial values to peak and then subsequently decreasing as the potential contaminants are either flushed out of the system, biodegraded or precipitaded (Jasper et al. 1985; Blight et al. 1999).

A large number of environmental indices have been eveloped in last four daades. Various indices are developed to quantify the pollution or quality of water and air. Usually, the indices are formulated based on studies conducted by the indices developers or on the Delphi technique (Kumar and Alappat 2009). In an effort to evelop a method for comparing the leachate pollution potential of various landfill sites in a given geographical area, an index known as Leachate Pollution Index (LPI) was formulated using Rand Corporation Delphi Technique (Kumar and Alappat 2003).

The LPI can be used to report leachate pollution changes in a particular landfill over time. The trend analysis so developed for the landfill can be used to assess the post closure monitoring periods. The leachate trend at a given landfill site can facilitate design of leachate treatment facilities for other landfills in the same region (Rafizul et al. 2011). The LPI can also be used to compare leachate contamination potential of different landfills in a given geographical area or around the world. To quantify and compare the leachate contamination potential of municipal landfills 18 characterstics to be known.The other potential applications of LPI include ranking of landfill sites based on leachate contamination potential, resource allocations for landfill remediation, enforcement of leachate standards, scientific research and public information (Kumar and Alappat 2003).

The intention of this study was to calculate the error invloved in estimation of LPI due to nonoavailability of data. In this study it is analyzed the possible error associated with estimation of LPI. The study area was Garbage Treatment Plant Landfill at Chittagong where daily waste disposal were 200 MT on average.

2. METHODOLOGY ADOPTED

2.1 Leachate Pollution Index (LPI)

The formulation process and complete description on the development of the LPI has been discussed elsewhere (Kumar and Alappat 2003). The LPI represents the level of leachate contamination potential of a given landfill. It is a single number ranging from 5 to 100 (like a grade) that expresses the overall leachate contamination potential of a landfill based on several leachate pollution parameters at a given time.

2.2 LPI Variables and Their Weight

The 18 parameters chosen and their corresponding weights are as follows: chromium (Cr): 0.064; lead (Pb): 0.063; chemical oxygen demand (COD): 0.062; mercury (Hg): 0.062; biochemical oxygen demand (BOD_5): 0.061; arsenic (As): 0.061; cyanides (Cn): 0.058; phenolic compounds: 0.057; zinc (Zn): 0.056; pH: 0.055; total kjeldhal nitrogen (TKN):0.053; nickel (Ni): 0.052; total coliform bacteria (TCB): 0.052; ammonia nitrogen (NH_4-N): 0.051; total dissolved solids (TDS): 0.050; copper (Cu): 0.050; chlorides (Cl-): 0.048; and total iron (Fe): 0.044. The weight factor indicates the importance of each pollutant variable to the overall leachate pollution. The sum of the weights of all 18 parameters is one.

2.3 Variable Curves

The averaged sub-index curves for all the pollutant variables have been reported by Kumar and Alappat (2003).

2.4 Variable Aggregation

The weighted sum linear aggregation function was found to be the most suitable one for the calculation of LPI (Kumar and Alappat 2004) and is as follows:

$$LPI = \sum_{i=1}^{n} w_i p_i \qquad (1)$$

Where, where LPI is weighted additive leachate pollution index; w_i = the weight for the i^{th} pollutant variable; p_i = the sub-index value of the i^{th} leachate pollutant variable, number of leachate pollutant parameters; n =18 and $\sum w_i = 1$.

However, when the data for all the leachate pollutant variables included in LPI are not available, the LPI can be calculated using the data set of the available leachate pollutants by the equation:

$$LPI = \frac{\sum_{i=1}^{n} w_i p_i}{\sum w_i} \qquad (2)$$

Where m=number of leachate pollutant for which data are available, but in that case, m<18 and $\sum w_i < 1$.

2.5 Errors Involved in Calculating LPI Due to Nonavailability of Data

To assess the errors involved in calculating LPI due to nonavail ability of data, a case study is taken up. Leachate samples from Chittagong Garbage Treatment Plant Landfill were collected and analyzed in the laboratory is provided in Table 1 to evaluate the error invlolved in calculation LPI due to nonavailability of data.

Table 1: Leachate characterstics of Chittagong Garbage Treatment Plant Landfill

SL	Leachate pollutant	Concentration*	Sub-index value (p_i)
1	Total chromium (Cr)	1.3	8
2	Lead (Pb)	1	12
3	Chemical oxygen demand (COD)	9700	78
4	Mercury (Hg)	0.007	5.5
5	Biochemical oxygen demand (BOD_5)	4800	52
6	Arsenic (As)	0.01	5
7	Cyanide (Cn)	0.7	11
8	Phenol compounds	3.2	10
9	Zinc (Zn)	2.9	7
10	pH	8	6
11	Total kjeldhal nitrogen (TKN)	718	20
12	Nickel (Ni)	0.06	7.5
13	Total colifom bacteria (TCB)	6700	84
14	Ammonia nitrogen (NH_4-N)	487	50
15	Total dissolved solid (TDS)	11350	23
16	Copper (Cu)	3.5	28
17	Chlorides (Cl-)	3250	24
18	Total Iron (Fe)	79	9

*All values in mg/L except pH and total coliform unit (cfu/100ml). * Average of 4 samples taken between 1st August and 30th September 2011.*

3. CASE STUDY

To estimate the possible errors involved in calculating LPI, due to the nonavailability of leachate data, two approaches have been made as

- Ignoring pollutant data based on weight factor and
- Ignoring pollutant data based on sub-index value.

The sub-index values of all the pollutant parameters in lechate based on their concentrations are reported in Table 1. The subindex values have been derived from the subindex curves for all the parameters reported by Kumar and Alappat (2003). The LPI value based on these sub-index values has been calculated using Equation (1) and provided in the fifth column, Table 2. The LPI calculated based on these 18 parameters is considered to be the true LPI value of the landfill.

3.1 Errors Introduced by Ignoring Pollutant Data Based on Weight Factor

In this approach, two options are discussed. In the first option, the data of the pollutants having low weight factors is ignored and in the second option, the data of the pollutants with high weight factors are assumed to be not available.

3.1.1 Removing Pollutants with Low Weight Factors

1. In the first step, the concentration of the total iron, the parameter having lowest weight, is presumed to be unknown. Hence, by deleting the subindex value of total iron, the LPI value is derived by using Eq. (2). The derived LPI value is reported in the sixth column, Table 2.
2. In the next step, the concentration of chlorides, the parameter having second lowest weight, is also presumed to be unknown in addition to the concentration of total iron. Again using Eq. (2), the LPI of the data set with 16 parameters is calculated and reported in the seventh column, Table 2.
3. In a similar fashion, it is presumed that the concentrations of copper, total dissolved solids, ammonia nitrogen, total coliform bacteria, nickel, total kjeldhal nitrogen, pH, and zinc are also not known one by one in addition to the earlier unknown concentrations of the parameters. The derived LPI values considering

concentration of 15, 14, 13, 12, 11, 10, 9,8,7,6,5 and 4 parameters are calculated and reported in columns 8, 9,10, 11, 12, 13, 14,15,16,17,18 and 19 of Table 2, respectively.

4. The percentage error introduced calculating LPI values, with respect to the LPI value when data are available for all 18 leachate pollutants, is also reported in the last row of respective columns of Table 2.
5. The variation in LPI values with respect to the number of parameters considered in calculating LPI is provided in Figure 1. It also gives the percentage error introduced in calculating LPI values with respect to the number of parameters considered.

3.1.2 Removing Pollutants with High Weight Factors

A similar procedure was followed here starting with the parameterhaving the highest weight factor.

1. In the first step, the concentration of the chromium, which has the highest weight factor, is presumed to be unknown. The LPI value, ignoring the subindex value of chromium, is calculated and reported in column 6, Table 3.
2. Then, step by step it is presumed that the concentrations of the pollutants lead, COD, mercury, BOD_5, arsenic, cyanide, phenol, zinc, and pH are not known in addition to the earlier presumed unknown parameters. The LPI values so calculated are reported in Table 3.
3. The percentage error in calculating LPI values, with respect to the LPI value when data for all 18 parameter are considered, is also reported in the last row of respective columns of Table 3.
4. Figure 2 shows the variation in LPI values with respect to the number of pollutants considered in calculating LPI. It also gives the percentage error introduced in calculating LPI values with respect to the number of parameters considered.

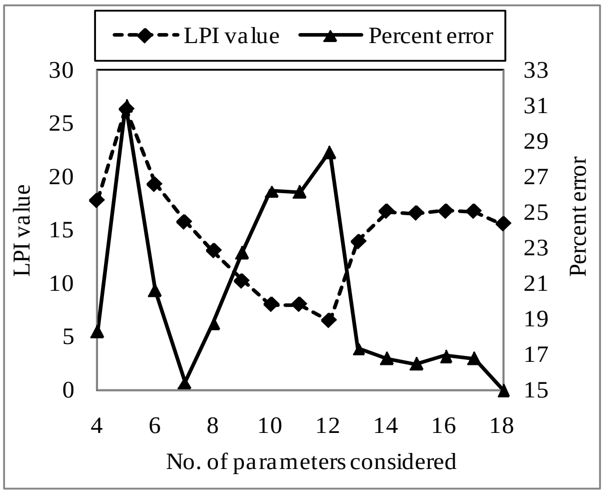

Figure 1: Variation of LPI and percent error when low weight parameters are ignored

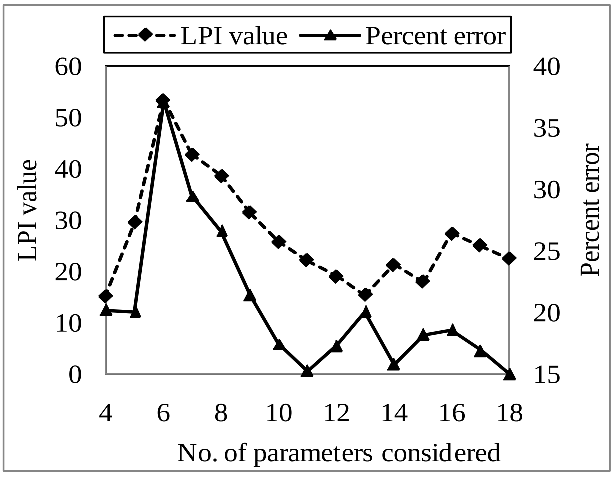

Figure 2: Variation of LPI and percent error when high weight parameters are ignored

Table 2. Estimating errors involved in calculating LPI values due to nonavailability of data (Parameters with low weight factors ignored)

Pollutant	**Pollutant weight, w_i**	**Pollutant concentration, C_i**	**Subindex value, p_i**	**LPI with 18 parameter, w_ip_i**	**LPI with 17 parameter, w_ip_i**	**LPI with 16 parameter, w_ip_i**	**LPI with 15 parameter, w_ip_i**	**LPI with 14 parameter, w_ip_i**	**LPI with 13 parameter, w_ip_i**	**LPI with 12 parameter, w_ip_i**	**LPI with 11 parameter, w_ip_i**	**LPI with 10 parameter, w_ip_i**	**LPI with 9 parameter, w_ip_i**	**LPI with 8 parameter, w_ip_i**	**LPI with 7 parameter, w_ip_i**	**LPI with 6 parameter, w_ip_i**	**LPI with 5 parameter, w_ip_i**	**LPI with 4 parameter, w_ip_i**
Cr	0.064	1.3	8	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512
Pb	0.063	1	12	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756
COD	0.062	9700	78	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836
Hg	0.062	0.007	5.5	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341
BOD_5	0.061	4800	52	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	-
As	0.061	0.01	5	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	-	-
Cn	0.058	0.7	11	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	-	-	-
Phenol	0.057	3.2	10	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	-	-	-	-
Zn	0.056	2.9	7	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	-	-	-	-	-
pH	0.055	8	6	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33		-	-	-	-	-
TKN	0.053	718	20	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	-		-	-	-	-	-
Ni	0.052	0.06	7.5	0.39	0.39	0.39	0.39	0.39	0.39	0.39	-	-		-	-	-	-	-
TCB	0.052	6700	84	4.368	4.368	4.368	4.368	4.368	4.368	-	-	-		-	-	-	-	-
NH_4-N	0.051	487	50	2.55	2.55	2.55	2.55	2.55	-	-	-	-		-	-	-	-	-
TDS	0.05	11350	23	1.15	1.15	1.15	1.15	-	-	-	-	-		-	-	-	-	-
Cu	0.05	3.5	28	1.4	1.4	1.4	-	-	-	-	-	-		-	-	-	-	-
Chloride	0.048	3250	24	1.152	1.152	-	-	-	-	-	-	-		-	-	-	-	-
Iron	0.045	79	9	0.405	-	-	-	-	-	-	-	-		-	-	-	-	-
Summation	1.000			24.327	23.922	22.770	21.370	20.220	17.670	13.302	12.912	11.852	11.522	11.130	10.560	9.922	9.617	6.445
Total weight				1.000	0.955	0.907	0.857	0.807	0.756	0.704	0.652	0.599	0.544	0.488	0.431	0.373	0.312	0.251
Derived LPI				24.327	25.049	25.105	24.936	25.056	23.373	18.895	19.804	19.786	21.180	22.807	24.501	26.601	30.824	25.677
Percent error				0.000	2.969	3.197	2.503	2.996	3.922	22.330	18.594	18.665	12.936	6.247	0.716	9.346	26.706	5.551

Note: All concentrations are in mg/L except for pH and total coliform bacteria (cfu/100mL)

Table 3. Estimating errors involved in calculating LPI values due to nonavailability of data (Parameters with high weight factors ignored)

Pollutant	**Pollutant weight, w_i**	**Pollutant concentration, C_i**	**Subindex value, p_i**	**LPI with 18 parameter, w_ip_i**	**LPI with 17 parameter, w_ip_i**	**LPI with 16 parameter, w_ip_i**	**LPI with 15 parameter, w_ip_i**	**LPI with 14 parameter, w_ip_i**	**LPI with 13 parameter, w_ip_i**	**LPI with 12 parameter, w_ip_i**	**LPI with 11 parameter, w_ip_i**	**LPI with 10 parameter, w_ip_i**	**LPI with 9 parameter, w_ip_i**	**LPI with 8 parameter, w_ip_i**	**LPI with 7 parameter, w_ip_i**	**LPI with 6 parameter, w_ip_i**	**LPI with 5 parameter, w_ip_i**	**LPI with 4 parameter, w_ip_i**
Cr	0.064	1.3	8	0.512	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Pb	0.063	1	12	0.756	0.756	-	-	-	-	-	-	-	-	-	-	-	-	-
COD	0.062	9700	78	4.836	4.836	4.836	-	-	-	-	-	-	-	-	-	-	-	-
Hg	0.062	0.007	5.5	0.341	0.341	0.341	0.341	-	-	-	-	-	-	-	-	-	-	-
BOD_5	0.061	4800	52	3.172	3.172	3.172	3.172	3.172	-	-	-	-	-	-	-	-	-	-
As	0.061	0.01	5	0.305	0.305	0.305	0.305	0.305	0.305	-	-	-	-	-	-	-	-	-
Cn	0.058	0.7	11	0.638	0.638	0.638	0.638	0.638	0.638	0.638	-	-	-	-	-	-	-	-
Phenol	0.057	3.2	10	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	-	-	-	-	-	-	-
Zn	0.056	2.9	7	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	-	-	-	-	-	-
pH	0.055	8	6	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	-	-	-	-	-
TKN	0.053	718	20	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	-	-	-	-
Ni	0.052	0.06	7.5	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	-	-	-
TCB	0.052	6700	84	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	-	-
NH_4-N	0.051	487	50	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	-
TDS	0.05	11350	23	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15
Cu	0.05	3.5	28	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4
Chloride	0.048	3250	24	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152
Iron	0.045	79	9	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405
Summation	1.000			24.327	23.815	23.059	18.223	17.882	14.710	14.405	13.767	13.197	12.805	12.475	11.415	11.025	6.657	4.107
Total weight				1.000	0.936	0.873	0.811	0.749	0.688	0.627	0.569	0.512	0.456	0.401	0.348	0.296	0.244	0.193
Derived LPI				24.327	25.443	26.414	22.470	23.874	21.381	22.974	24.195	25.775	28.081	31.110	32.802	37.247	27.283	21.280
Percent error				0.000	4.589	8.577	7.634	1.860	12.111	5.560	0.542	5.954	15.432	27.881	34.837	53.108	12.150	12.526

Note: All concentrations are in mg/L except for pH and total coliform bacteria (cfu/100mL)

3.2 Errors Introduced by Ignoring Pollutant Data Based on Sub-indexValue

In this approach, three scenarios were considered as reveals:

1. Firstly, it was presumed that data for one parameter having the highest sub-index value are not available and then calculate LPI. Next it is presumed that the data for two sub-indices having the highest sub-index values are not available and so on for three, four, fifth to fourteen parameters. From Table 2, column 4, it is observed that the pollutants having the highest sub-index values are TCB, COD, BOD_5 and NH_4–N with subindex values of 84, 78, 52, and 50, respectively. To start with, it is presumed that data for TCB are not available. Based on this assumption, the LPI value is calculated using Eq. (2), and reported in column 6, Table 4. In the next step it is presumed that data for COD are also not available in addition to TCB. The LPI value based on this assumption is calculated using Eq. (2) and reported in column 7, Table 4. Similarly, it is presumed that data for three and then four pollutants are not available, and the corresponding LPI values are calculated and reported in columns 8 and 9 of Table 4. Moreover, in the similar fashion, it is presumed that data for fifth to fourteen pollutants are not available, and the corresponding LPI values are calculated and reported in columns 9 to 19 in Table 4. The percentage error introduced in calculating these fourteen LPI values is also calculated and reported in the last row of respective columns in Table 4 and the results are shown in Figure 3.
2. Then it is presumed that the data for one parameter having the least sub-index value are unknown and then LPI calculated. Subsequently it is presumed that, data for two, three, four and fourteen parameters having the lowest sub-index values are not available. From column 4 of Table 2, it is observed that the parameters having the lowest sub-index values are As, Hg, pH and Zn with sub-index values of 5, 5.5, 6 and 7, respectively. The above procedure is repeated for calculating LPI values (reported in Table 5). The percentage error introduced in calculating LPI for each presumption is also calculated and reported in respective columns of Table 5 and the results are shown in Figure 4.
3. After that it is presume that the data for two parameters, having the highest sub-index value and lowest sub-index value are unknown simultaneously. The parameter having the highest sub-index value is TCB and the one having the lowest sub-index value is As. The LPI for this is calculated and reported in column 6, Table 6. Subsequently it is presume that data for four parameters: two parameters with highest sub index values (TCB and COD) and two parameters with lowest sub-index values (As and Hg) are not available. The LPI for this presumption is calculated and reported in column 7, Table 6. Moreover, it is presume that data for six parameters: three parameters with highest sub index values (TCB, COD and BOD_5) and three parameters with lowest sub-index values (As, Hg and pH) are not available. The LPI for this presumption is calculated and reported in column 8, Table 6. Subsequently, it is presume that data for eight parameters: four parameters with highest and four with lowest sub index values are not available. The LPI for this presumption is calculated and reported in column 9, Table 6. The percent error introduced in calculating LPI values is reported in the last row of respective columns in Table 6 and the results are shown in Figure 5.

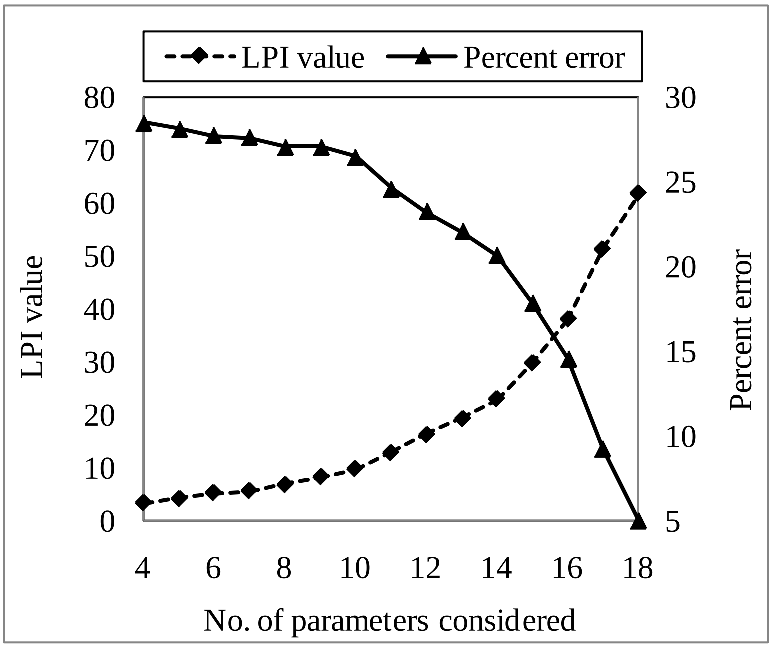

Figure 3: Variation of LPI and percent error when pparameters with highest sub-index values ignored.

Table 4. Estimating errors involved in calculating LPI values due to nonavailability of data (Parameters with highest sub-index values ignored)

Pollutant	**Pollutant weight, w_i**	**Pollutant concentration, C_i**	**Subindex value, p_i**	**LPI with 18 parameter, w_ip_i**	**LPI with 17 parameter, w_ip_i**	**LPI with 16 parameter, w_ip_i**	**LPI with 15 parameter, w_ip_i**	**LPI with 14 parameter, w_ip_i**	**LPI with 13 parameter, w_ip_i**	**LPI with 12 parameter, w_ip_i**	**LPI with 11 parameter, w_ip_i**	**LPI with 10 parameter, w_ip_i**	**LPI with 9 parameter, w_ip_i**	**LPI with 8 parameter, w_ip_i**	**LPI with 7 parameter, w_ip_i**	**LPI with 6 parameter, w_ip_i**	**LPI with 5 parameter, w_ip_i**	**LPI with 4 parameter, w_ip_i**
Cr	0.064	1.3	8	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	0.512	N.C	N.C
Pb	0.063	1	12	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	N.C	N.C	N.C	N.C	N.C	N.C
COD	0.062	9700	78	4.836	4.836	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Hg	0.062	0.007	5.5	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341	0.341
BOD_5	0.061	4800	52	3.172	3.172	3.172	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
As	0.061	0.01	5	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305	0.305
Cn	0.058	0.7	11	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	N.C	N.C	N.C	N.C	N.C
Phenol	0.057	3.2	10	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	N.C	N.C	N.C	N.C
Zn	0.056	2.9	7	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392	0.392
pH	0.055	8	6	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
TKN	0.053	718	20	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Ni	0.052	0.06	7.5	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	N.C
TCB	0.052	6700	84	4.368	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
NH_4-N	0.051	487	50	2.55	2.55	2.55	2.55	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
TDS	0.05	11350	23	1.15	1.15	1.15	1.15	1.15	1.15	1.15	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Cu	0.05	3.5	28	1.4	1.4	1.4	1.4	1.4	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Chlorides	0.048	3250	24	1.152	1.152	1.152	1.152	1.152	1.152	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Total Iron	0.045	79	9	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	0.405	N.C	N.C	N.C
Summation	1.000			24.327	19.959	15.123	11.951	9.401	8.001	6.849	5.699	4.639	3.883	3.245	2.675	2.27	1.758	1.368
Total weight				1.000	0.948	0.897	0.835	0.774	0.724	0.676	0.628	0.575	0.512	0.454	0.397	0.342	0.278	0.226
Derived LPI				24.327	21.054	16.860	14.313	12.146	11.051	10.132	9.075	8.068	7.584	7.148	6.738	6.637	6.324	6.053
Percent error				0.000	13.455	30.696	41.166	50.072	54.573	58.352	62.696	68.825	70.619	70.619	72.302	72.716	74.005	75.118

Note: All concentrations are in mg/L except for pH and total coliform bacteria (cfu/100mL)
N.C= parameter not considered

Table 5. Estimating errors involved in calculating LPI values due to nonavailability of data (Parameters with lowest sub-index values ignored)

Pollutant	**Pollutant weight, w_i**	**Pollutant concentration, C_i**	**Subindex value, p_i**	**LPI with 18 parameter, w_ip_i**	**LPI with 17 parameter, w_ip_i**	**LPI with 16 parameter, w_ip_i**	**LPI with 15 parameter, w_ip_i**	**LPI with 14 parameter, w_ip_i**	**LPI with 13 parameter, w_ip_i**	**LPI with 12 parameter, w_ip_i**	**LPI with 11 parameter, w_ip_i**	**LPI with 10 parameter, w_ip_i**	**LPI with 9 parameter, w_ip_i**	**LPI with 8 parameter, w_ip_i**	**LPI with 7 parameter, w_ip_i**	**LPI with 6 parameter, w_ip_i**	**LPI with 5 parameter, w_ip_i**	**LPI with 4 parameter, w_ip_i**
Cr	0.064	1.3	8	0.512	0.512	0.512	0.512	0.512	0.512	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Pb	0.063	1	12	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	0.756	N.C	N.C	N.C	N.C	N.C
COD	0.062	9700	78	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836	4.836
Hg	0.062	0.007	5.5	0.341	0.341	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
BOD_5	0.061	4800	52	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172	3.172
As	0.061	0.01	5	0.305	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Cn	0.058	0.7	11	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	0.638	N.C	N.C	N.C	N.C	N.C	N.C
Phenol	0.057	3.2	10	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Zn	0.056	2.9	7	0.392	0.392	0.392	0.392	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
pH	0.055	8	6	0.33	0.33	0.33	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
TKN	0.053	718	20	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	N.C	N.C	N.C	N.C
Ni	0.052	0.06	7.5	0.39	0.39	0.39	0.39	0.39	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
TCB	0.052	6700	84	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368	4.368
NH_4-N	0.051	487	50	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55	2.55
TDS	0.05	11350	23	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	N.C	N.C	N.C
Cu	0.05	3.5	28	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	N.C
Chlorides	0.048	3250	24	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	N.C	N.C
Total Iron	0.045	79	9	0.405	0.405	0.405	0.405	0.405	0.405	0.405	N.C	N.C	N.C	N.C	N.C	N.C	N.C	N.C
Summation	1.000			24.327	24.022	23.681	23.351	22.959	22.569	22.057	21.652	21.082	20.444	19.69	18.63	17.48	16.33	14.93
Total weight				1.000	0.939	0.877	0.822	0.766	0.714	0.662	0.617	0.617	0.559	0.496	0.443	0.393	0.343	0.282
Derived LPI				24.33	25.58	27.00	28.41	29.97	31.61	33.32	35.09	34.17	36.57	39.69	42.05	44.47	47.60	52.93
Percent error				0.00	5.16	11.00	16.77	23.21	29.93	36.96	44.25	40.46	50.34	63.17	72.85	82.81	95.66	117.57

Note: All concentrations are in mg/L except for pH and total coliform bacteria (cfu/100mL)
N.C= parameter not considered

Table 6: Estimating errors involved in calculating LPI values due to nonavailability of data (Parameters with highest and lowest sub-index values ignored simultaneously)

Pollutant	Pollutant weight, w_i	Pollutant concentration, C_i	Subindex value, p_i	LPI with 18 parameter, w_ip_i	LPI with 16 parameter, w_ip_i	LPI with 14 parameter, w_ip_i	LPI with 12 parameter, w_ip_i	LPI with 10 parameter, w_ip_i
Cr	0.064	1.3	8	0.512	0.512	0.512	0.512	N.C
Pb	0.063	1	12	0.756	0.756	0.756	0.756	0.756
COD	0.062	9700	78	4.836	4.836	N.C	N.C	N.C
Hg	0.062	0.007	5.5	0.341	0.341	N.C	N.C	N.C
BOD_5	0.061	4800	52	3.172	3.172	3.172	N.C	N.C
As	0.061	0.01	5	0.305	N.C	N.C	N.C	N.C
Cn	0.058	0.7	11	0.638	0.638	0.638	0.638	0.638
Phenol	0.057	3.2	10	0.57	0.57	0.57	0.57	0.57
Zn	0.056	2.9	7	0.392	0.392	0.392	0.392	N.C
pH	0.055	8	6	0.33	0.33	0.33	N.C	N.C
TKN	0.053	718	20	1.06	1.06	1.06	1.06	1.06
Ni	0.052	0.06	7.5	0.39	0.39	0.39	0.39	0.39
TCB	0.052	6700	84	4.368	N.C	N.C	N.C	N.C
NH_4-N	0.051	487	50	2.55	**2.55**	2.55	2.55	N.C
TDS	0.05	11350	23	1.15	1.15	1.15	1.15	1.15
Cu	0.05	3.5	28	1.4	1.4	1.4	1.4	1.4
Chlorides	0.048	3250	24	1.152	1.152	1.152	1.152	1.152
Total Iron	0.045	79	9	0.405	0.405	0.405	0.405	0.405
Summation	1.000			24.327	19.654	14.477	10.975	7.521
Total weight				1.000	0.887	0.774	0.657	0.539
Derived LPI				24.33	22.16	18.70	16.70	13.95
Percent error				0.00	8.92	23.11	31.33	42.64

Note: All concentrations are in mg/L except for pH and total coliform bacteria (cfu/100mL)
N.C= parameter not considered

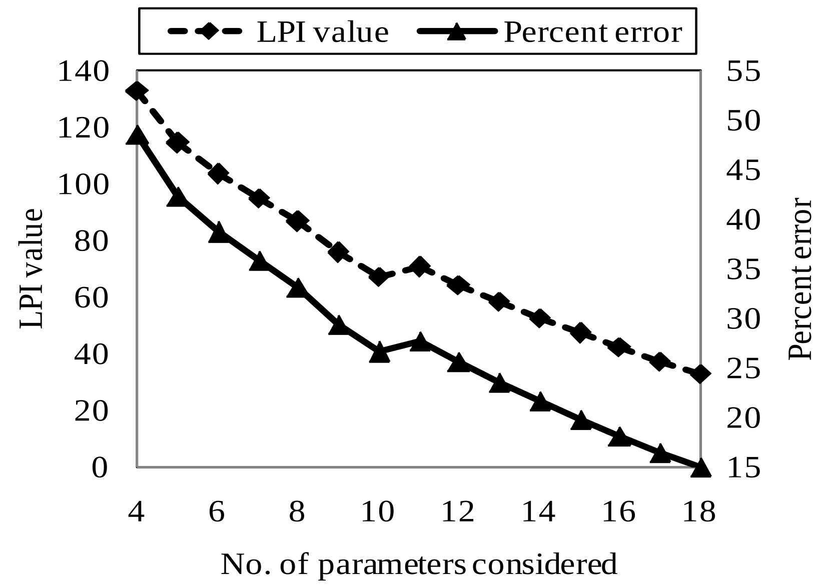

Figure 4: Variation of LPI and percent error when pparameters with lowest sub-index values ignored

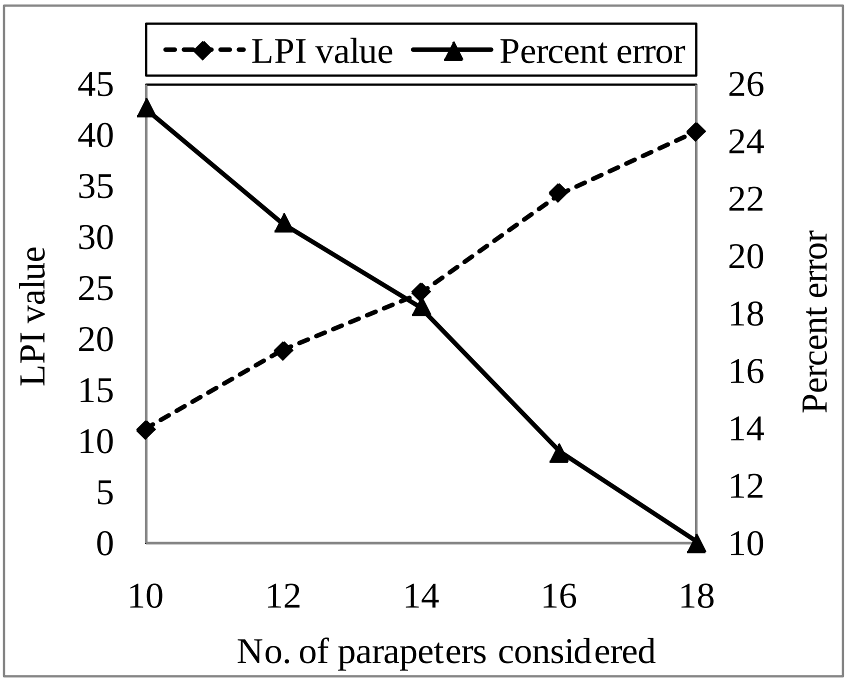

Figure 5: Variation in LPI due to nonavailability of data parameters with highest and lowest sub-index values ignored simultaneously

4. RESULTS AND DISCUSSIONS

4.1. Errors Introduced by Ignoring Pollutant Data Based on Weight Factor

Based on Table 2 and Figure 1, it can be depicted that the error introduced in calculating LPI is 2.97%, when concentration of one parameter, i.e., total iron (pollutant with lowest weight factor) is not considered. It depicts that error increases to 3.19% when concentration of two parameters, total iron and chlorides, is unknown. Then the error decreases to 2.50% when data for three parameters, total iron, chlorides, and copper, is not known. The error introduced in LPI is the highest, i.e., 26.71%, when data of thirteen parameters is not considered. After this, the error decreases with the increase in the number of missing parameters. The error is just 0.72% when only seven parameters are considered (that is when data for eleven parameters are not considered).

Similarly from Table 3 and Figure 2, it is observed that the error is 4.59%, when data for one parameter that is chromium (pollutant with highest weight factor) is not considered in calculating LPI. The error increases to 8.58% when data for two pollutants, chromium and lead, are ignored in calculating LPI. The error is highest, i.e., 53.11%, when data for the twelve pollutants are ignored. But the percent error dips to 12.15% when thirteen parameters are not considered in calculating the LPI value. This leads to the conclusion that the error involved in calculating LPI does not vary with the number of parameters considered and the variation is erratic. The erratic behavior in the error introduced in the LPI is due to the fact that the parameters ignored while calculating LPI had significantly different subindex values with respect to the overall LPI.

4.2. Errors Introduced by Ignoring Pollutant Data Based on Subindex Value

The error introduced is highest, i.e., 75.12%, when data for the fourteen parameters having the highest subindex values are not considered, followed by 74.0% when data for the thirteen parameters having the highest subindex values are not considered. However, it may reduce to 30.69 and 13.45% when data for the two and one parameters having the highest subindex values, respectively are not considered. Moreover, the errors introduced due to nonconsideration of data of one parameters having the lowest subindex values are 5.16% and it rises gradually to 117.57% when data for the fourteen parameters are not considered.

Here, it is important to note that derived LPI are lower than the true LPI value in the case when pollutants with high sub-index values are ignored. On the contrary, the derived LPI are higher than the true LPI when data for the pollutants with low sub-index values are ignored. Hence, the results obtained by ignoring data for the pollutants with high subindex values produce falsified results, leading to a false sense of security, indicating a relatively more polluted environment as less polluted.

But in the case when data for the pollutants with low subindex values are ignored, distended results are obtained and the results will raise an unnecessary alarm by indicating a comparatively less polluted environmental situation to be more contaminated. Based on this discussion, it is possible to conclude that the errors involved in LPI values are high and dangerous when the data for the pollutants having high subindex values are not available as compared to the scenario when data for the parameters having low subindex values are not available. The error involved in LPI values is low when data for the pollutants having highest and lowest subindex values are not considered simultaneously.

5. CONCLUSION

Result reveals that maximum error (22.33%) is introduced in calculating LPI when the data for the six low weight parameters are not considered, but the error is as low as 1.00% when data for eleven parameters are not considered. Similarly the error involved in calculating LPI is maximum (53.10%) when data for twelve high weight parameters are ignored, but the error involved is low (1.0%) when data for eleven high weight factors are ignored. Here it can be concluded that the errors introduced in calculating LPI values are not at all related to the number of parameters whose concentrations are not known. From this it can be concluded that LPI is more reliable and accurate as a larger number of parameters are available in its formulation. In contrary, the error introduced in calculating LPI is more sensitive when data for the parameters having high sub-index values are not considered as the derived LPI values are lower than the true LPI value and produce vague results. Finally, it can be concluded that errors introduced in calculating LPI are marginal when the data of the parameters having highest and lowest sub-index values are not considered simultaneously.

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MATHEMATICAL MODEL FOR DHAKA CITY SOLID WASTE MANAGEMENT

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ABSTRACT

Dhaka, the capital of Bangladesh, is generating more than 5000 tons of solid wastes per day. The Dhaka City Corporation, the responsible authority, is only damping these wastes or simply burning it down without treating them further as a very useful source of energy. The main research objective is to bring an engineering solution of the present problem. A Massive Integrated Bio-plant is designed, based on that a mathematical model showing benefit-cost ratio of it. The method has been formulated using an integer linear problem. It has been presented as a decision making tool in the planning and management of integrated program of solid waste collection, transportation, incineration, recycling, composting, disposal etc. The proposed model includes: trucks for transportation of waste, replacement trucks and their depots, incineration with energy recovery, damping site, plastic plant, bio-gas plant, fertilizer plant etc. The obtained result is the benefit-cost ratio per trip derived from the cost owing to investment and management cost, operational costs from the use of replacement trucks, benefit from energy generation from various plants. The mathematical model can be used as an important devise for planners in solid waste management and may be useful for the Dhaka city solid waste management.

Keywords: *Solid waste management, Mathematical Model, Benifit-Cost ratio, Massive Integrated Bio-plant.*

1. INTRODUCTION

The protection of the environment and natural resources is increasingly becoming very important through environmentally sustainable waste management program. Therefore following the part of waste managers, a sustainable approach to the waste management and to integrate strategies that will produce the best practicable option becomes necessary. This is a very challenging task since it involves taking into account economic, technical, regulatory, and environmental issues (see Costi et al [1]). Waste management can become more complex if social and political considerations are also into account.

It is not very ago, Dhaka, the capital of Bangladesh possessed an image as a city of green and water bodies. Within last few decades this serene and tranquil city has been transformed into one the most polluted and crowded cities in the world. Dhaka City Corporation (DCC) is responsible for managing the city. The juristic of DCC area is 360 km^2 and accommodates over 15 million people. It is projected that the population would be more than 20 million by the year 2015. Right now it is the 9th largest cities in the world and 28th among the mostly densely populated cities in the world (23029/km^2).

People around Dhaka city gathered here for better jobs and livelihood because Dhaka is called the city of opportunity for the people of Bangladesh. This huge amount of population is generating massive amount of wastes which DCC is dealing with. Unfortunately most of these wastes are damped in the landfill areas and then burnt. If utilized in the engineering way with proper technology under the supervision of experts, these wastes can be recycled, reused and can be used to generate tlectricity, biogases and various different products.

This is the high time to plan a massive project that can stand parallel and uphold the gas and electricity scarcity. The developed countries like ANNEX countries which include UK, Germany, and USA etc. have already developed the technology of building massive integrated bio-gas plant which is used for multiple ways of generating various products and useful energy. Unfortunately the people of Bangladesh are being deprived of enjoying such facilitations even though they are producing massive wastes and uselessly throwing them away. People of Dhaka city largely depends on gas and electricity for cooking and other activities. As the population increases the problem will also rise and it can be easily predicted that one day there will be nothing to spare for these poor citizens of Dhaka!

2. OBJECTIVES

The establishment of the mathematical model is on a massive integrated bio-plant. The work focuses on the construction and design of a schematic of one such plant. The study will also illustrate a detailed description of the mathematical model that can be used as tools for decision makers in the day to day planning and management of integrated program of solid waste collection, incineration, recycling, treatment and disposal.

3. METHODOLOGY

3.1. Schematic Diagram of the Massive Integrated Bio-plant

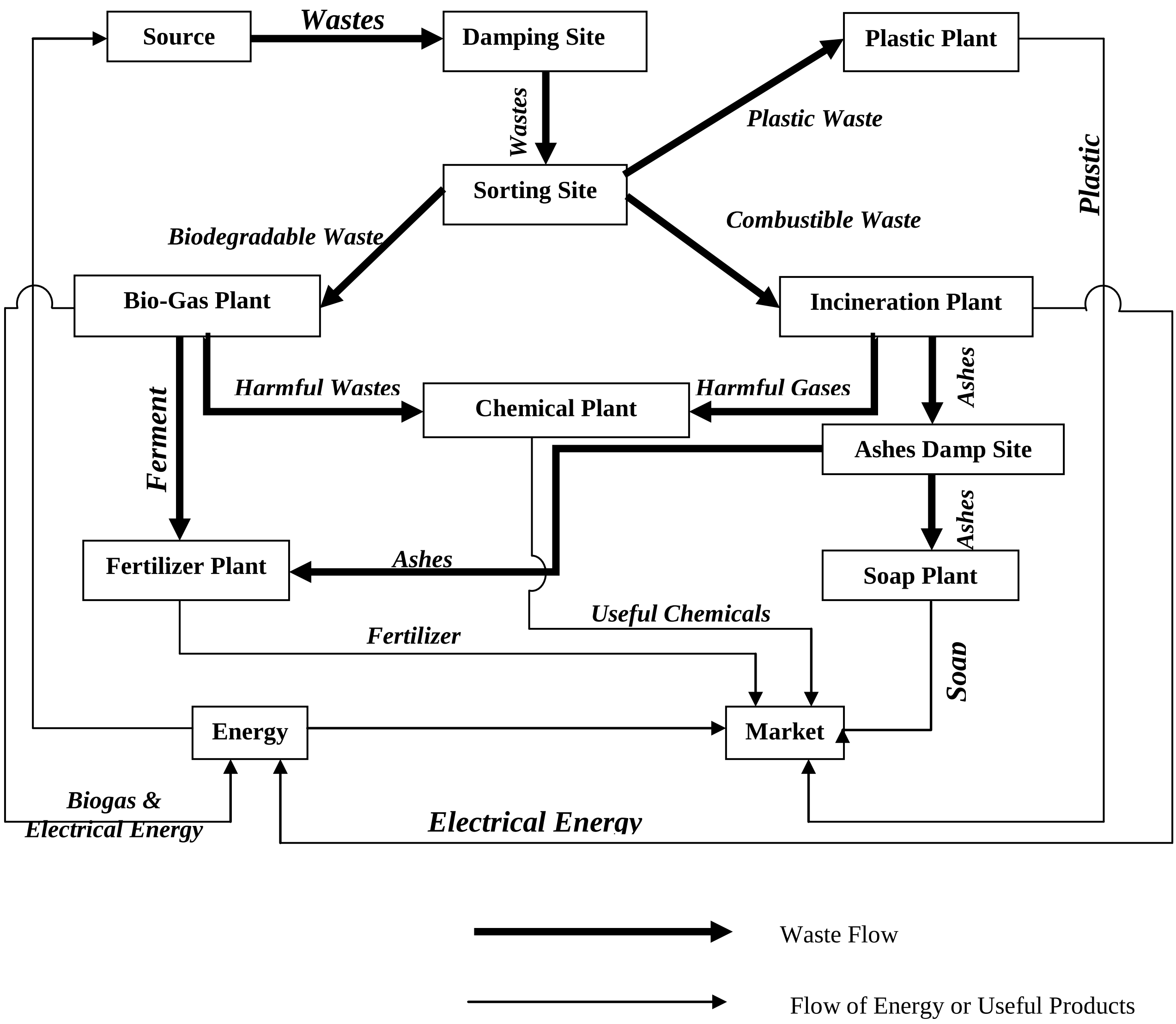

Figure 01: Schematic Design of a Massive Integrated Bio Plant

3.2. Working Principle of the Massive Integrated Bio-plant

Wastes generated by the sources are damped in the damping sites. These wastes after some treatment are sent to the sorting sites where three categories of wastes are separted; namely the biodegradable wastes, combustible wastes and plastic wastes.

Biodegradable wastes are those wastes which has got the putrefying ability like garbage. Therefore these wastes are transferred to Bio Plant. Combustible wastes are the wastes which have high heating value, low humidity, moderate activation energy and which are easily burnt; like paper, leaves, bits of wood, straw etc. are taken to the incineration plant for combustion or burning. Plastic Wastes like ployvinyl chloride (PVC), low-density polyethene (LDPE), polypropylene (PP), polystyrene (PS) are taken to the plastic plants, mainly for recycling after series of treatments.

Biogas plant produces bio methane and electrical energy as the useful products, which are directly or indirectly passed back to the source or market. This plant also produces harmful wastes especially gases which are treated in the chemical plant before it is released to the environment or send back to the market.The ferments produced in the biogas plants are transported to the fertilizer plant where they are converted to low priced fertilizer which is back to the market.

The purpose of incineration plant is to treat combustible waste. The electrical energy produced is back to the source directly or indirectly. The harmful gases produced are treated in the nearby chemical plant before it is released to the environment and useful chemical to the market. The ashes produced are damped to the ash damp sites where the types of ashes are classified. One part ashes are transported to the fertilizer plant and the other part is transported to the soap plant.

Plastic plants take care of the plastics wastes most of which are recycled back to the market. The soap plants are plants where different soaps are manufactured. There may be different brands of soaps that can be manufactured by certain specific type of ashes. The chemical plants are those where harmful wastes are converted to useful product and human friendly chemicals so that there would be no adverse effect of these chemicals on the environment.

3.3. Mathematical Model Formation

The model is formulated as an integer linear program (see Wolsey [13]). It has been presented as a decision making tool for solid waste planners in decisions concerning the overall management of solid waste. Several treatment plants and facilities along with the waste collection component has been considered within the proposed model: trucks for the transportation of waste; replacement trucks and their depots; incinerators with energy recovery; damping site; several plants for recycling useful products and energy etc. The user may prefer to measure the transportation costs in terms of costs per trip made from the waste source; in that case the model is more appropriate.

The objective function of the model describes tipping fees, total investment and maintenance costs, costs for buying or hiring trucks, transportation costs as well as operational costs from the use of replacement trucks. The benefits from energy generation, compost, and recycling are also incorporated in the objective function. Apart from the transportation costs, installation and operational costs for plants and landfills, and benefits from different plants, the objective function also includes truck purchase costs as well as costs due to the presence of replacement trucks depots. Special attention has been given to deciding the number and the type of trucks that are used to transport a given type of waste from the waste collection points to the plants or landfills. Replacement trucks are also considered with the observation of possible breakdowns of the operational trucks. Moreover, instead of measuring the amount of waste using the number of trucks used multiplied by their capacities, continuous variables can be introduced to measure directly the amount of waste that goes to the plants and landfills.

3.3.1. Assumptions

The mathematical model is based on the following assumptions:

1. The waste handling operations in the mathematical model are to be executed daily.

2. Each plant or site is considered to have the benefit-cost ratio more than 1 and is justified to invest for the proposed model.

3. It is assumed that each plant or site is giving more rate of return than minimum acceptable rate of return (MARR).

4. Waste source and the damping site are located at the centers of the waste generating areas.

5. The damping site and the sorting site is considered to be at the same location.

6. The incinerator and the ash damp site is also considered at the same location.

3.3.2. Schematic Model for Mathematical Formation

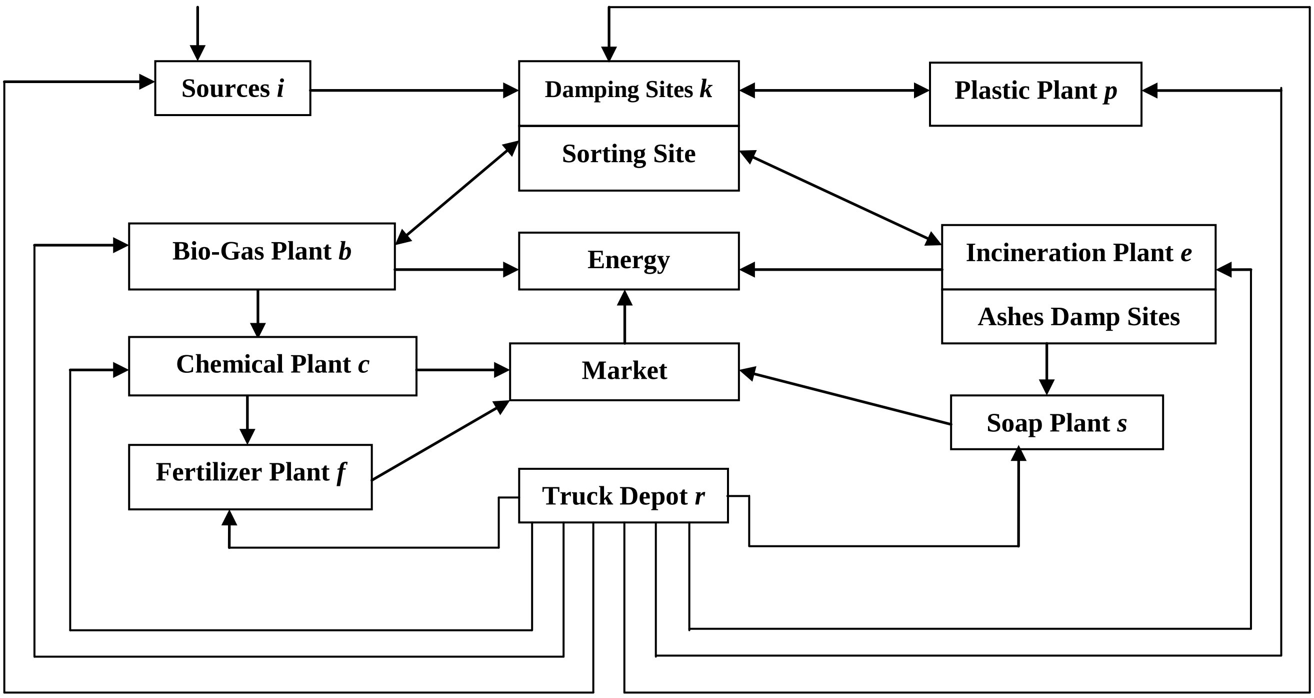

Figure 02: A Schematic Diagram of the Massive Integrated Bio plant according to the assumptions

3.3.3. Indices

i = 1, 2…	I: location of waste source
b = 1, 2…	B: location of biogas plant
c = 1, 2…	C: location of chemical plant
e = 1, 2…	E: location of incineration plant
f = 1, 2…	F: location of fertilization plant
k = 1, 2…	K: location of damping site
p = 1, 2…	P: location of plastic plant
r = 1, 2…	R: location of truck depot
s = 1, 2…	S: location of soap plant
l = 1, 2…	L: truck type
g = 1, 2…	G: waste Type

3.3.4. Variables

X_{kbg}, X_{kpg}, X_{keg} : respectively the total number of trips made by trucks of type *l* used every day to carry waste of type *g* from damping site *k* to a biogas plant at *b*, to a Plastic Plant *p* and to an incinerator at *e*.

x_{kbg}, x_{kpg}, x_{keg}: respectively the number of trucks of type *l* used every day to carry waste of type *g* damping site *k* to a biogas plant at *b*, to a Plastic Plant *p* and to an incinerator at *e*.

Y_{ikg}, Y_{bkg}, Y_{ekg}, Y_{pkg} : the total number of trips made by trucks of type *l* used every day to carry waste of type *g* from a source *i*, biogas plant at *b*, an incinerator at *e*, to a plastic plant *p* to a damping site *k*.

y_{ikg}, y_{bkg}, y_{ekg}, y_{pkg}: the number of trucks of type *l* used every day to carry waste of type *g* from a source *i*, biogas plant at *b*, an incinerator at *e*, to a plastic plant *p* to a damping site *k*.

n_{rb}, n_{rc}, n_{re}, n_{rf}, n_{rk}, n_{rp}, n_{rs} : respectively number of trucks of type *l* used every day from a replacement trucks depot at *r* to a biogas plant at *b*, to a chemical plant at *c*, to an incinerator at *e*, to a fertilizer plant at *f*, to a damping site at *k*, to a plastic plant *p* and to a soap plant at *s*.

z_b, z_c, z_e, z_f, z_k, z_p, z_{rs}, z_r : 0-1 variables indicating respectively, the presence of biogas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f*, a damping site at *k*, to a plastic plant *p*, a soap plant at *s* and a truck depot at *r*.

w_b, w_c, w_e, w_f, w_k, w_p, w_s : amount of waste transported everyday respectively, to a biogas plant at *b*, to a chemical plant at *c*, to an incinerator at *e*, to a fertilizer plant at *f*, to a damping site at *k*, to a plastic plant *p* and to a soap plant at *s*.

T_l: The number of trucks of type *l* used every day.

$(RT)_l$: The number of replacement trucks of type *l* required every day.

3.3.5. Input Data/Parameter

α_l : capacity (in tonnes) of a truck of type *l*.

m_{rb}, m_{rc}, m_{re}, m_{rf}, m_{rk}, m_{rp}, m_{rs}, m_{ri} : respectively the cost of moving a truck of type *l* from a replacement trucks depot at *r* to a biogas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f*, a damping site at *k*, a plastic plant *p*, a soap plant at *s* and a waste source at *i*.

u_{kb}, u_{kp}, u_{ke} : respectively transportation cost per unit of waste carried by a truck of type *l* from a damping site at *k* to biogas plant at *b*, to a plastic plant *p* and to an incinerator at *e*.

d_{ik},d_{bk}, d_{ek}, d_{pk} : respectively transportation cost per unit of waste carried by a truck of type *l* from a waste source at *i*, a biogas plant at *b*, an incenerator at *e* and a plastic plant at *p* to a damping site at *k*.

β_b, β_c, β_e, β_f, β_p, β_s : revenue respectively, per unit of waste at bio-gas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f* , a plastic plant at *p* and a soap plant at *s*.

f_l: the cost of buying a new truck of type *l*, $l = 1 ..., L$.

d_i: amount of waste at Source *i*.

ρ_b, ρ_c, ρ_e, ρ_f, ρ_p, ρ_s : fraction (%) of unrecovered waste respectively, at biogas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f*, to a plastic plant *p* and a soap plant at *s*.

δ_b, δ_c, δ_e, δ_f, δ_k, δ_p, δ_s, δ_r : respectively installation cost incurred in opening a biogas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f*, a damping site at *k*, to a plastic plant *p*, a soap plant at *s* and a replacement trucks depot at *r*.

γ_b, γ_c, γ_e, γ_f, γ_k, γ_p, γ_s : respectively variable cost incurred in handling a unit of waste at a biogas plant at *b*, a chemical plant at *c*, an incinerator at *e*, a fertilizer plant at *f*, a damping site at *k*, a plastic plant at *p* and a soap plant at *s*.

4. RESULTS/FINDINGS

The objective function represents the overall daily waste management costs. The first component, $V_1 (z, w, X, Y)$, gives the investment and waste handling expenses as well as transportation costs. The second component, $V_2(n,z)$, gives expenses owing to the use of replacement trucks. The third component, $V_3(x,y,n)$, is the total cost incurred for buying all the trucks of various kinds and capacities. In this function we have the installation cost parameters δ, and the variable cost parameters γ. The variables X and Y have been defined before. The component $Bn (w)$ is the income and revenues earned from per unit waste from different plants. The benefit-cost ratio per unit trip can then be derived dividing the benefit by the total cost, V, which is the summation of all the cost components V_1, V_2 and V_3. The model works on daily basis.

The component V_1 will show the overall cost for installation, transportations and other variable cost (e.g. maintenance).

$$\begin{aligned} V_1 (z, w, X, Y) = \; & [\Sigma_b (\delta_b z_b + \gamma_b w_b) + \Sigma_c (\delta_c z_c + \gamma_c w_c) + \Sigma_e (\delta_e z_e + \gamma_e w_e) + \Sigma_f (\delta_f z_f + \gamma_f w_f) \\ & + \Sigma_k (\delta_k z_k + \gamma_k w_k) + \Sigma_s (\delta_s z_s + \gamma_s w_s) + \Sigma_p (\delta_p z_p + \gamma_p w_p)] + \alpha_l [\Sigma_{glkb} u_{kb} X_{kbg} \\ & + \Sigma_{glkp} u_{kp} X_{kpg} + \Sigma_{glke} u_{ke} X_{keg} + \Sigma_{glik} d_{ik} Y_{ikg} + \Sigma_{glbk} d_{bk} Y_{bkg} \\ & + \Sigma_{glek} d_{ek} Y_{ekg} + \Sigma_{glpk} d_{pk} Y_{pkg}] \end{aligned} \tag{1}$$

The component V_2 concerns the total costs owing to the presence of replacement trucks (or standby trucks).

$$\begin{aligned} V_2 (n,z) = \; & \Sigma_{ril} (m_{ri} n_{ri}) + \Sigma_{rbl} (m_{rb} n_{rb}) + \Sigma_{rcl} (m_{rc} n_{rc}) + \Sigma_{rel} (m_{re} n_{re}) + \Sigma_{rfl} (m_{rf} n_{rf}) \\ & + \Sigma_{rkl} (m_{rk} n_{rk}) + \Sigma_{rsl} (m_{rs} n_{rs}) + \Sigma_{rpl} (m_{rp} n_{rp}) + \Sigma_r (\delta_r z_r) \end{aligned} \tag{2}$$

The component V_3 gives the total cost for buying all trucks required in the daily management of waste.

$$V_3 (x,y,n) = \Sigma_l f_l [T_l + (RT)_l] \tag{3}$$

The component Bn gives the benefits at the plants owing to the production of electric energy, compost, recycled materials and other revenues of the useful part per unit waste.

$$\begin{aligned} Bn (w) = \; & \Sigma_b \beta_b (1 - \rho_b) w_b + \Sigma_c \beta_c (1 - \rho_c) w_c + \Sigma_e \beta_e (1 - \rho_e) w_e \\ & + \Sigma_f \beta_f (1 - \rho_f) w_f + \Sigma_p \beta_p (1 - \rho_p) w_p + \Sigma_s \beta_s (1 - \rho_s) w_s \end{aligned} \tag{4}$$

Total Cost:

$$V = V_1 + V_2 + V_3 \tag{5}$$

The benefit-cost ratio per unit trip:

$$Benefit: Total\ Cost = Bn/ V \tag{6}$$

5. DISCUSSION

5.1. Limitations of the Mathematical Model

Some of the limitations of the model is as follows:

1. Although regulatory, technical, and environmental constraints are not comprehensively considered in these models, it is assumed that they can be handled in details without affecting the results of the model.

2. In our model we neglect the traffic jam in Dhaka city. If this would be considered the model would be more realistic.

3. In reality the united sites like the damping site, sorting site, incinerator, and the ash damping site may not exists at the same place. In that case the mathematical models need to be modified.

4. As this is a massive project design and requires a huge amount of money to start with. As it needs time to construct, time to time modification of the design might result a better benefit. This means it is not the final model and has a lot of scopes to be modified.

5. Selection of suitable locations for such a massive project might give a bit of trouble for the investors. They may not risk their money on the basis of these models only.

6. Some of the plant in the design might give lesser benefit-cost ratio and should be discarded. But in our model we assume the individual plant has got the benefit cost ratio more than one, so that the mathematical models could be established.

7. During transportation there is some loss of the waste due to littering or scattering.

5.2. Application of the Mathematical Model

1. The mathematical model can be simulated in AMPL modeling system with a compatible solver CPLEX (see ILOG AMPL CPLEX System [10]). This powerful algebraic modeling language solves problems in linear, non-linear, and integer programming etc.

2. The model can be used as important tools for planners in solid waste management in an urban environment.

3. The model may as well be adapted for use in other areas of application like industrial warehouse location and product distributions among industry agents.

6. CONCLUSIONS

The study illustrates a schematic design of a massive integrated bio plant. The plant concerns with following plants: biogas plants, plastic plants, incinerators, fertilizer plants, soap plants and chemical plants. These plants work in parallel with damping sites, truck depots and the waste source. The model works on daily basis and the revenues from these plants are considered as benefit; the total cost includes the installation cost, variable cost, maintenance cost, tipping cost, transportation cost and the cost for buying trucks. Based on the devise a mathematical model is developed showing the benefit-cost ratio per unit trip. The mathematical formula is derived applying the linear integer program.

The formula only works under certain assumptions and limitations. The performance can be further studied using a powerful algebraic programming AMPL and CPLEX. In that case the overall management of solid waste may be more easily accessible to solid waste planner in decision making. Although implementation of this mathematical model is expensive, risky and time consuming; there are various ways this model can be modified and made suitable in real life.

ACKNOWLEDGEMENTS

My deepest gratitude and respect to my supervisor Dr. A.B.M. Zohrul Kabir, Professor, Mechanical and Chemical Engineering Department (MCE), Islamic University of Technology (IUT) for his encouragement, enlightening advices and extreme patience throughout the work for establishing such a creative project. It is his idea to set up a mathematical model unless otherwise it would remain undone. My special thanks to some of the officers working in Dhaka City Corporation (DCC) and Local Government Engineering Departmant (LGED). Finally I thank my family whose love has kept me going through the project; myfriends in IUT and colleagues in my office deserves thanks too.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ANALYSIS OF LEACHATE CHARACTERISTICS IN PILOT SCALE LANDFILL LYSIMETER THROUGH STATISTICAL APPROACH

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ABSTRACT

The main focus of this study was to analyze the characteristics of leachate in landfill lysimeter in terms of inorganic and organic compounds as well as metal and heavy metal concentrations against their operational conditions based on statistical tools through statistical package for social science (SPSS) software. To these attempts, three pilot scale landfill lysimeter test facilities were set-up at KUET campus, Bangladesh. Three different situations of landfill were considered here, however, both the sanitary landfill and open dump conditions having a base liner and two different types of cap liner were simulated. The simulated landfill lysimeters were maneuvered at different operational conditions of leachate detection (A_1) and collection (A_2) system of open dump lysimeter-A as well as the leachate collection system of sanitary landfill lysimeter-B and C. Based on the evaluated results it was observed that the physical, chemical and biological characteristics of the leachate were varied significantly for the different condition of construction materials used as cap and base liner in lysimeter. The relevant results of leachate characteristics from general statistics and extensive statistical tools such as Analysis of Variance (ANOVA) and correlations has also been evaluated and discussed. For analyzing the leachate data, the confidence level 95% for ANOVA was considered.

1. INTRODUCTION

The municipal solid waste (MSW) refers to the materials discarded in the urban areas for which municipalities are held responsible for collection, transport and final disposal (Alamgir et al., 2005). The management tier of MSW mainly prominence on waste generation, source storage and segregation, collection, transport and finally the ultimate disposal site. There are two options for MSW dumping all over the world; one is open dumping another is sanitary landfill (WasteSafe 2005). The term 'landfill' is used herein to describe a unit operation for final disposal of 'municipal solid waste (MSW)'on land, designed and constructed with the objective of minimum impact to the environment. This term encompasses the other terms such as 'secured landfill' and 'engineered landfills' which are also sometimes applied to MSW disposal units (Tubtimthai 2003). The term 'landfill' can be treated as synonymous to 'sanitary landfill' of MSW, only if the latter is designed on the principle of waste containment and is characterized by the presence of a liner and leachate collection system to prevent ground water contamination. Sanitary landfill is one of the secure and safe facilities for the disposal of MSW; however, it needs high standard of environment protection in the operation of landfill (Davis and Cornwell 1998). Moreover, it is a well-suited method for managing of MSW all over the world and to investigate the performance of sanitary landfill the behavioral patterns namely; leachate generation, landfill gas (LFG) emissions etc. are required (Visvanathan et al. 2002). The word Lysimeter is a combination of the two Greek words "Lusis" means "Solution" & "Metron" means "Measure" and the original aim was to measure water balance (Rafizul et al., 2009). Based on this concept and to investigate the performance of both the base and cap liners made of Compacted Clay Liner (CCL), three lysimeter (designated as A, B and C) as a control mechanism test set-ups were designed and hence constructed at KUET campus, Khulna, Bangladesh.

The produced leachate in municipal solid waste (MSW) disposal sites is considered as one of the highly contaminated resources from physical, chemical and biological point of view. A number of incidences have been reported in the past, where leachate had contaminated the surrounding soil and polluted the underlying ground water aquifer or nearby surface water bodies (Rafizul et al. 2009). However, the best possible knowledge of leachate characteristics at a specific site is an essential management tool. This is not only important for new containment landfills, where leachate will be extracted and treatment/disposal facilities must be designed in advance, but also important for the old landfill sites, where the environmental safeguards rely on the natural attenuating properties of the geological strata, in both saturated and unsaturated zones beneath them, to reduce levels of contaminants to the levels where they pose minimal threats to environment (Robinson et al. 1992). Regulations usually require the MSW to be placed above the water table and not directly overlying the aquifer. In this environment, moisture may infiltrate through the top, generally resulting in an unsaturated flow through the refuse; leachate is then generated through the complex physical and biochemical processes and is transported to the underlying strata. In that manner leachate may pose a severe pollution threat to ground water supplies (Kelly et al. 1976).

To analyze the lysimeter leachate characteristic through statistical measurement has been done by a one-way ANOVA test. This one-way ANOVA test was completed using the SPSS 16.0. The different sampled landfill leachate concentrations were placed into the program to determine, which mean concentrations in the leachate parameters were significantly different from one another. The ANOVA test was used because it has no restriction on the number of means.

Table 1: Operational conditions of lysimeter to simulate different landfill conditions

Lysimeter	*Operating condition*	*Refuse (kg)*	*Liner specification*	*Simulation*
A	Open dump lysimeter with leachate detection (A_1) system	2860	400mm thick CCL as a barrier between leachate detection and collection system of lysimeter-A	present practice of open dumping
	Open dump lysimeter with leachate collection (A_2) system			
B	Sanitary landfill lysimeter with gas collection and leachate recirculation system	2985	Cap liner-I (300mm thick CCL)	applicability of designed top cover
C		2800	Cap liner-II (900mm thick natural top soil)	

2. OVERVIEW OF LANDFILL LYSIMETERS AT KUET CAMPUS

Three landfill lysimeters designated as A, B and C were designed and hence constructed at KUET campus, Bangladesh based on a reference cell shown in Figure 1, showing all the design components in details. The operational condition, liner specifications, simulation behavior and the total weight of MSW deposited in each lysimeter are presented in Table 1. Three cylindrical lysimeter having outer diameter of 1.98 m and inner diameter of 1.48 m, with a height of 3.35m, and a leachate collection tank (3.68mx1.56mx1.64m) accommodating four separate leachate discharge pipes in the temporary collection and storage containers, were constructed using 250mm thick brick wall resting on a 250mm thick of reinforced cement concrete mat foundation at a depth of 760mm below the existing ground surface. The lysimeter were plastered inside and outside with two coatings of waterproofing agent to avoid leakage and corrosion due to acidic environment. The MSW deposited in lysimeter mainly consists of 93 (w/w) organic (food and vegetables), 3 (w/w) of plastic/polythene and 2 (w/w) of leather/rubber, 1 (w/w) of animal bone and rubber/leather as well as 1 (w/w) of rope/straw and egg pill. However, the organic content and moisture content of MSW was found 52 and 65%, respectively, and the total volume was 2.80m^3 (height 1.6m) with a manual compaction to achieve the unit weight of 1,064kg/m^3. At the bottom of each landfill lysimeter, a concrete layer of 125mm thickness was provided then the lysimeters were filled with stone chips (diameter 5-20mm) and coarse sand (diameter 0.05- 0.4mm) to the height of 15cm of each to ensure uniform and uninterrupted drainage. At the base of each landfill lysimeter after placing the perforated leachate collection pipe, a geo-textile sheet having 0.60m wide and 1.65m length was placed on the top to avoid a rapid clogging of this perforated pipe by the sediments from the lysimeter.

Moreover, the type and volume of MSW deposited in open dump lysimeter-A was exactly the same as deposited in the reference cell. In open dump lysimeter-A, a compacted clay liner (CCL) of 400mm thickness was placed as the base liner and a layer of compost of 150mm thick was used as the top cover to simulate the behavior of present practice of open dumping in Bangladesh (Table 1). In this lysimeter the MSW was not covered by a top cover system to pervert the movement of air, water and generated LFG. Moreover, the thickness of the deposited MSW in lysimeter-A is such that it is expected the atmospheric air can move in the entire MSW deposited in this cell with negligible inference. Due to the mentioned practical situations, lysimeter-A, represents an open dump landfill condition comparing the counterparts i.e sanitary landfill lysimeter-B and C

In contrary, in sanitary landfill lysimeter-B (Figure 2), the characteristics and volume of the MSW is similar to that of the lysimeter-A and lysimeter-C. However, it differs with open dump lysimeter-A, by a top cover similar to that of Pilot Scale Sanitary Landfill (PSSL) constructed in Khulna without having a base liner, because this cell aims to examine the applicability of the designed top cover. The top cover consists of stone chips (diameter 5-20mm) and coarse sand (0.05-0.40 mm diameter) layer each of 100 mm thickness, then a 300 mm compacted clay liner (CCL). On the CCL, there were 150mm of coarse sand (0.4-.05mm diameter) and 150mm of stone chips (diameter 5-20mm), which was followed by 600mm thick top soil (Table 1). However, in sanitary landfill lysimeter-C, there was also no base liner and the top cover was different than that of the lysimeter-B. In this case no CCL was used; however, 900 mm topsoil was used instead of 300 mm CCL and 600 mm top soil (Table 1). But the drainage and gas collection layers remain same as the lysimeter-B. In both the lysimeters, 38mm diameter of gas collection and 25mm diameter of leachate recirculation pipe were installed. During the installation of these pipes and penetration through the top cover, special arrangements i.e. disc shaped rubber gasket were used to prevent the leakage of gas Designated compaction of the CCL in the lysimeter means the degree of compaction which was provided in the pilot scale sanitary landfill (PSSL) at Rajbandh, Khulna. To achieve the designated compaction at the CCL of lysimeter, locally manufactured hammer similar to that used in the PSSL was employed.

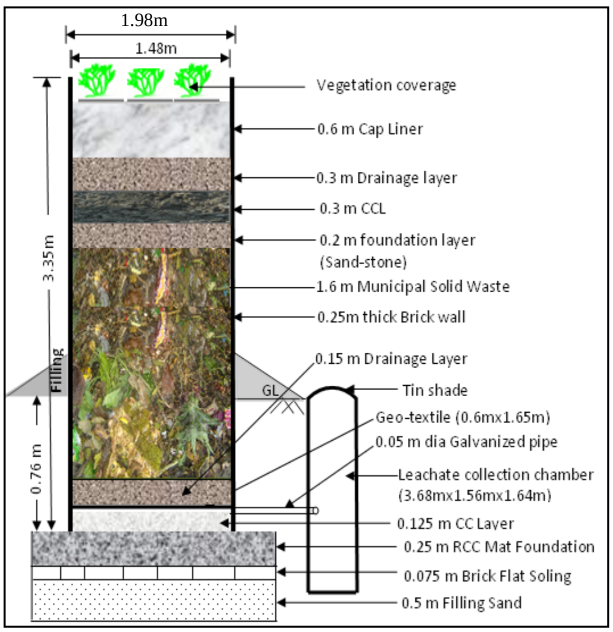

Figure 1: Schematic diagram of reference cell for landfill lysimeter design

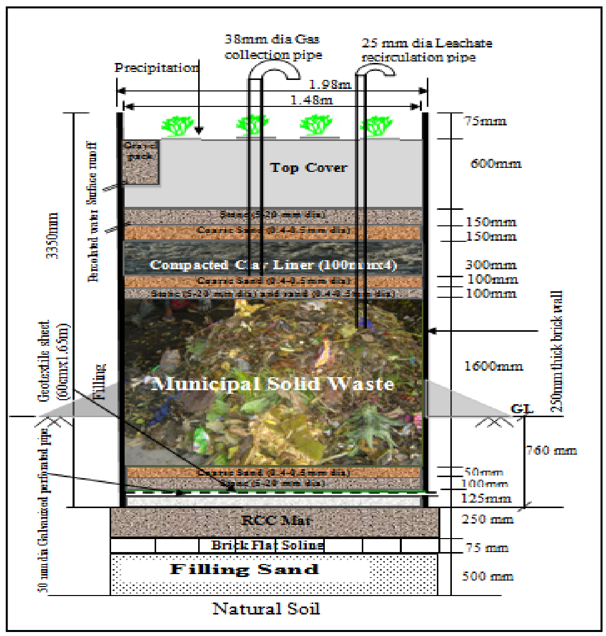

Figure 2: Schematic diagram of sanitary landfill lysimeter-B

3. CHARACTERIZATION OF MATERIALS USED IN THIS STUDY

The total weight of deposited MSW in lysimeter A, B and C were 2860, 2984 and 2798Kg, respectively, and total volume was 2.80m^3 (height 1.6m). The unit weight was obtained in the range of 1000-1064 kg/m^3. It can be depicted that the MSW deposited in lysimeter are characterized in the laboratory and the food and vegetable wastes was the predominant component and about 93(w/w). The moisture content was found of 65%. Result obtained from sieve analysis test is provided in Table 2.

Table 2: Sieve analysis of MSW deposited in lysimeter

Finer fraction (%)				
300-200 mm	100 mm	76.2 mm	38.2 mm	19.1 mm
100	76.25	63.72	45.22	24.34

Based on the tests conducted on the clay which was used as liner, the moisture content, plastic limit, liquid limit, plasticity index, and shrinkage limit were found as 22, 22, 43, 21 and 16%, respectively. It is important to know the physical and mechanical properties of the soils as thoroughly as possible before assessing their physico-chemical or hydro-mechanical behavior (Rafizul et al. 2009). Moreover, Daniel (1995) a researcher stated that using clay as the CLL in solid waste landfill the following fundamental criteria to be satisfied such as coefficient of permeability ($k \leq 1x10^{-7}$ cm/s), plasticity index (PI<7), water content (w%)>plastic limit (PL%) as well as at least 30% fines and 15% clay. In addition, the percentages of soil constituents were found as sand, silt and clay of 10, 56.6 and 33.4%, respectively, while, the value of optimum moisture content and maximum dry density of 18% and 16 kN/m^3, respectively and the coefficient of hydraulic conductivity of $1.90x10^{-7}$ cm/sec. The mineralogical compositions of clay used as CCL as shown in Table 3 as measured in the laboratory of the Department of Applied Geology, Karlsruhe University, Germany (Roehl, 2007) .

Table 3: Mineralogical compositions of clay used as CCL in lysimeter (Roehl, 2007)

Minerals (amount in weight-%)		Sample (0-7m)	Sample (13-23m)
Non-clay minerals	Quartz	19%	17%
	Feldspars	<1%	<1%
	Carbonates	<1%	<1%
Non-swelling clay minerals	Illite	~50%	~50%
	Kaolinite	~10%	~10%
	Chlorite	<1%	1-2%
Swelling clay minerals: Smectite		20%	19%

4. METHODOLOGY ADOPTED

In lysimeter the leachate generation rate is mainly influenced by climate such as rainfall and evaporation in which the lysimeter were situated and it depends on the initial moisture content of MSW amongst others (Rafizul et al. 2012). Leachate samples were analyzed for physico-chemical parameters of pH by pH meter (HACH, Model No. Sens ion 156), alkalinity by titration method, hardness by EDTA titrimetric method as well as COD by closed reflexive method as per the Standard Methods (APHA, 1998). However, the Ca, Na, K and Mg ions were determined using flame atomic absorption spectrophotometer (VARIAN; AA/2400) with proper standard calibrations. In addition, Heavy metals viz., Cu, Cr, Cd, Ni, Pb, Mn, Fe and Zn were analyzed using spectrophotometer (HACH; DR/2400) as per the Standard Methods (APHA, 1998).

ANOVA has been conducted on the relevant parameters in leachate using the following assumption-

- Each landfill was independent of one another and
- The data has the same variance, they were positively skewed.

After completing the one-way ANOVA test, a Turkey's test was also conducted because it is the most commonly used Post Hoc analysis test.

5. RESULTS AND DISCUSSIONS

On the basis of experimental results obtained by conducting several tests on the collected leachate sample in the laboratory, results were obtained and hence discussed in followings.

5.1 Variation of Organic and Inorganic Compounds

The variation of inorganic compound in terms of pH, alkalinity and hardness as well as the organic carbon of COD at varying elapsed period were analyzed and hence discussed in the following articles.

5.1.1 pH

The variation of pH with respect to the elapsed period and operational conditions for both the open dump lysimeter-A and sanitary landfill lysimeter-B and C is provided in Figure 3. From Figure 3, it can be signified that detection system of open dump lysimeter-A had the highest pH with 8.47 than that of collection system at the waste deposition period of 6 weeks then gradually decreased. In case of present study, pH in leachate varied from 5.88-8.75 and the maximum concentration of pH was found for collection system of lysimeter-A of 8.75.

Table 4: One-way ANOVA analysis for pH

Multiple Comparisons

Dependent Variable:Value of Ph

	(I) Lysimeter group	(J) Lysimeter group	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval Lower Bound	95% Confidence Interval Upper Bound
Tukey HSD	A1	A2	-.48325*	.12555	.001	-.8093	-.1572
		B	-.65025*	.12555	.000	-.9763	-.3242
		C	-.71150*	.12555	.000	-1.0375	-.3855
	A2	A1	.48325*	.12555	.001	.1572	.8093
		B	-.16700	.12555	.545	-.4930	.1590
		C	-.22825	.12555	.269	-.5543	.0978
	B	A1	.65025*	.12555	.000	.3242	.9763
		A2	.16700	.12555	.545	-.1590	.4930
		C	-.06125	.12555	.962	-.3873	.2648
	C	A1	.71150*	.12555	.000	.3855	1.0375
		A2	.22825	.12555	.269	-.0978	.5543
		B	.06125	.12555	.962	-.2648	.3873
Games-Howell	A1	A2	-.48325*	.11508	.000	-.7854	-.1811
		B	-.65025*	.12568	.000	-.9803	-.3202
		C	-.71150*	.12904	.000	-1.0504	-.3726
	A2	A1	.48325*	.11508	.000	.1811	.7854
		B	-.16700	.12196	.522	-.4874	.1534
		C	-.22825	.12542	.272	-.5578	.1013
	B	A1	.65025*	.12568	.000	.3202	.9803
		A2	.16700	.12196	.522	-.1534	.4874
		C	-.06125	.13521	.969	-.4162	.2937
	C	A1	.71150*	.12904	.000	.3726	1.0504
		A2	.22825	.12542	.272	-.1013	.5578
		B	.06125	.13521	.969	-.2937	.4162

*. The mean difference is significant at the 0.05 level.

Table 4 shows the output of the ANOVA analysis using the Post Hoc (Turkey HSD) test with the statistically significant difference between the lysimeter operational conditions. Here, Table 3 reveals that in case of A_1 and A_2 system of open dump lysimeter-A, the significance level is 0.001, which is below 0.05 and, therefore, there is a statistically significant difference. Here, it can be established that the variation of pH in case of A_1 and A_2 system of open dump lysimeter-A may be occurred due to the providing of 400mm thick CCL as a barrier between the A_1 and A_2 system of lysimeter-A. As the A_2 provided just below the MSW deposited in lysimeter-A and the followed A_1 was separated with the 400mm thick CCL and this operational mode may be considered for the variation of pH between these two systems. Similarly in case of A_2 system with respect to lysimeter-B and C, the significance level is 0.545 and 0.269, respectively, which is above 0.05, therefore, it shows homogeneity of variances. In contrary, In case of sanitary lysimeter-B, the statistically significant level is 0.962, which is above 0.05 with respect to sanitary lysimeter-C, therefore, it shows homogeneity of variances.

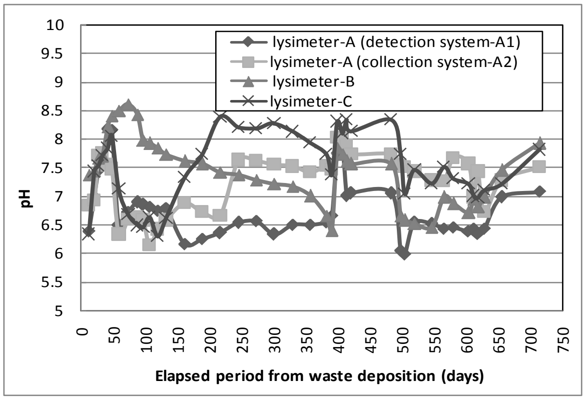

Figure 3: Variation of pH with elapsed period

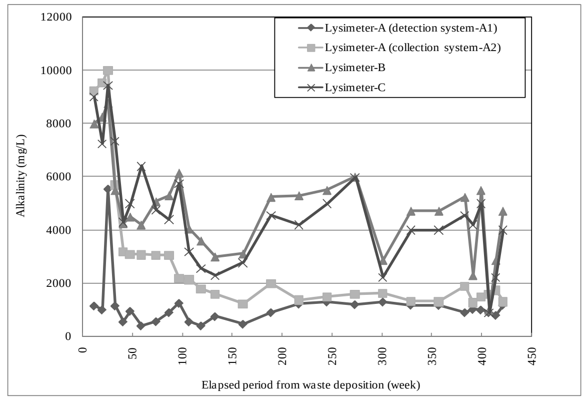

Figure 4: Variation of Alkalinity with elapsed period

5.1.3 Alkalinity

The variation of alkalinity in leachate is provided in Figure 4. Figure 4 reveals that alkalinity was significantly decreased in the range of 8000-1111.2 mg/L and 9000 -2278 mg/L for lysimeter-B and C, respectively, up to the deposition period close to 23 weeks. The Alkalinity varied from 1111.2 to 9000 mg/L. The variation of Alkalinity is due to the difference of two lysimeters in terms of thickness of cap liner and its compaction condition. In contrast, alkanility for the collection system of sanitary landfill lysimeter-B and C may be occurred due to the difference of landfill lysimeter-B and C in terms of thickness and compaction conditions of cap liner.

From one way ANOVA test with obtained data it was found that, there was a significant difference with alkalinity between the landfills. Using the Post Hoc test, it showed that lysimeter-B was significantly different from lysimeter-C. No significant difference was found between others. In contrary, In case of sanitary lysimeter-B, the statistically significant level is 0.962, which is above 0.05 with respect to sanitary lysimeter-C, therefore, it shows homogeneity of variances.

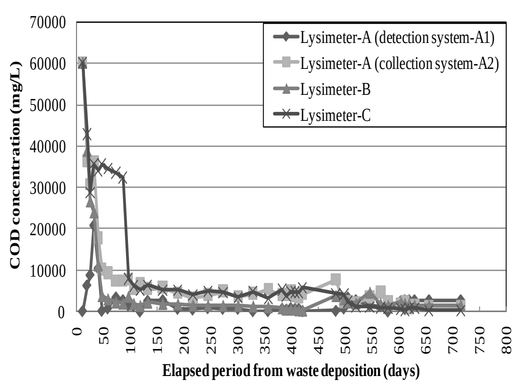

Figure 5: Variation of COD concentration with elapsed period

5.1.4 COD concentration

The concentration of COD in leachate was significantly decreased in the range of 60,000-800mg/L and 60,000-1280 mg/L for the collection system of sanitary landfill lysimeter-B and C, respectively, up to the elapsed period of

23weeks shown in Figure 5. A decrease of COD occurs over the elapsed period of landfill and it can be attributed to a combination of reduction in organic contaminants and the increased biodegradation of organic compounds (Krug and Ham 1995). A constant decrease in COD is also expected as degradation of organic matter continues (Ehrig 1989). Diaz (1996) introduced that the concentration of COD varied from 0-89,520mg/L of leachate generated from MSW in landfill. Diaz (1996) introduced that the concentration of COD varied from 0-89,520mg/L of leachate generated from MSW in landfill.

From one way ANOVA test with obtained data it was found that, there was a significant difference with COD concentration between the landfills. Using the Post Hoc test, it showed that, lysimeter-A2, B and C showed significant difference between them. No significant difference was found for lysimeter-A1 with others.

5.*1.5 Hardness*

Figure 6 represents the hardness in leachate and it significantly decreased in the range of 10,000-1852mg/L, 14,000-1344 mg/L and 10,000 to 1324 mg/L for the collection systems of lysimeter-A, B and C, respectively, up to the deposition period of 17 weeks and then it was increased up to the waste deposition period of 23 weeks for both the cases. Diaz (1996) introduced that the values of Hardness varied from 0 to 22,800mg/L of leachate generated from MSW in landfill.

From one way ANOVA test with obtained data it was found that, there was a significant difference with hardness between the landfills. Using the Post Hoc test, it showed that, lysimeter-C showed significant difference with the rest. Insignificant difference between others.

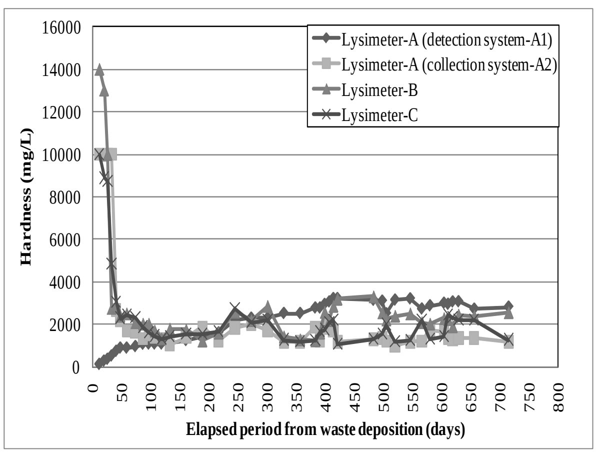

Figure 6: Variation of hardness with elapsed period

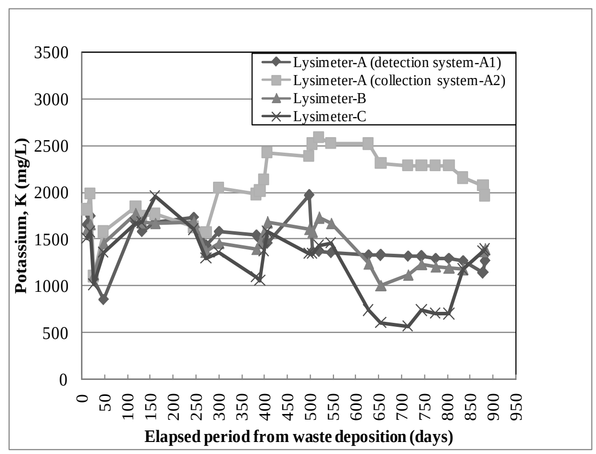

Figure 7: Variation of potassium concentration with elapsed period

5.2 Variation of Metal Concentrations

5.2.1 Potassium

Potassium concentration was ranging of 855.9 to 1967.0 mg/L, 1102.1 to 2589.0 mg/L, 1008.6 to 1764.3 mg/L and 567.2 to 1956.7 mg/L, for leachate detection (A_1) and collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 7. At the beginning of lysimeter operation, **p**otassium concentration was determined as 1650.5 and 1815.8 mg/L and after 50 day, it was dropped markedly to 855.9 and 1576.8 mg/L, and at 500 day, it was reached as maximum values of 1370 and 2590 mg/L, for leachate detection (A_1) and collection system (A_2) of lysimeter-A, respectively.

From one way ANOVA test with obtained data it was found that, there was a significant difference with Potassium between the landfills. Using the Post Hoc test, it showed that, lysimeter-B showed significant difference with lysimeter-A1. Insignificant difference between others.

5.2.2 Calcium

Calcium was ranging from 253.5 to 789.9 mg/L, 217.9 to 664.2 mg/L, 153.0 to 578.0 mg/L and 140.0 to 664.4 mg/L, for leachate detection (A_1) and collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 8. The highest concentration of Ca was measured in leachate detection (A_1) system of lysimeter-A with 789.9 mg/L, whereas, the lowest of 140 mg/L for lysimeter-B. On the other side, Ca concentration was decreased rapidly below 400 mg/L after day 300 in both the cases of leachate detection (A_1) and collection system (A_2) of lysimeter-A. However, causes a slow decrease in Ca concentration for both the cases of lysimeter-B and C, and markedly decreased around 200 mg/L after day 650.

From one way ANOVA test with obtained data it was found that, there was a significant difference with calcium between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_2 showed significant difference with the rest of the lysimeters. No significant difference between Lysimeter-A_1 and B.

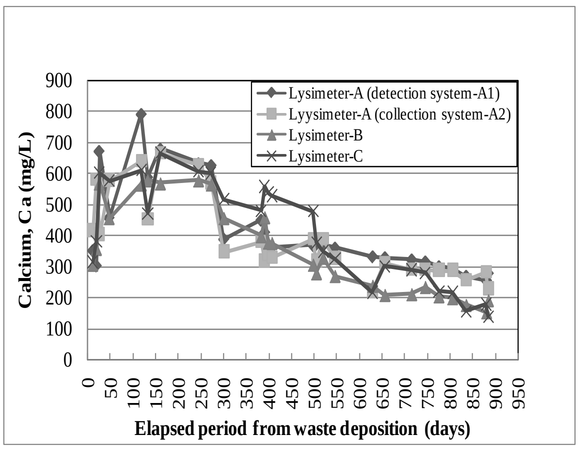

Figure 8: Variation of Calcium concentration with elapsed period

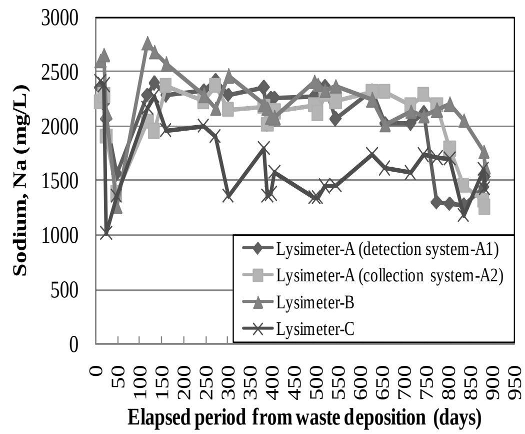

Figure 9: Variation of Sodium concentration with elapsed period

5.2.3 Sodium

Sodium concentration was ranging of 1267.9 to 2424 mg/L, 1251.6 to 2369.6 mg/L, 1256.3 to 2764.3 mg/L and 1011.9 to 2409.1 mg/L, for leachate detection (A_1) and collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 9. The highest concentration of Na was measured in case of lysimeter-B with 2764.3 mg/L, whereas, the lowest of 1011.9 mg/L in lysimeter-C. In contrast, Na for the collection system of sanitary landfill lysimeter-B and C may be occurred due to the difference of landfill lysimeter-B and C in terms of thickness and compaction conditions of cap liner.

From one way ANOVA test with obtained data it was found that, there was a significant difference with sodium between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_2 showed significant difference with lysimeter-A_1. No significant difference between others.

5.2.4 Magnesium

Figure 10 represents the concentration of magnesium and it was ranging from 147.25 to 589.9 mg/L, 127.15 to 564.2 mg/L, 228.4 to 605.9 mg/L and 156 to 594.2mg/L, for leachate detection (A_1) and collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively. At the beginning of the aerobic landfill lysimeter operation, Mg concentration was determined as 455.7 and 405.5 mg/L and at 120 day operation, it reached as maximum values of 589.9 and 540.6 mg/L for leachate detection (A_1) and collection system (A_2) of lysimeter-A, respectively.

From one way ANOVA test with obtained data it was found that, there was a significant difference with magnesium between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_1 showed significant difference with the rest of the lysimeters. No significant difference between lysimeter-A and B.

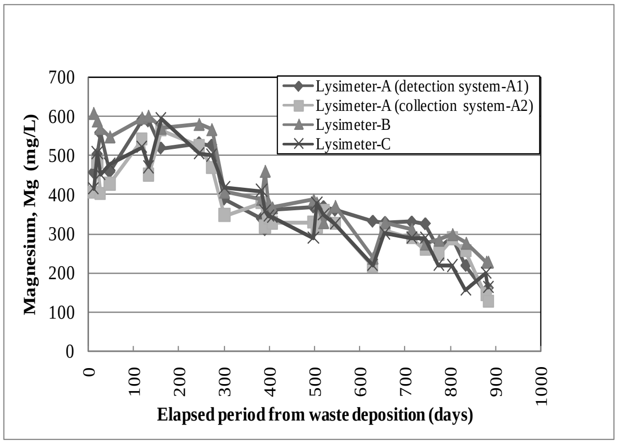

Figure 10: Variation of Magnesium concentration with elapsed period

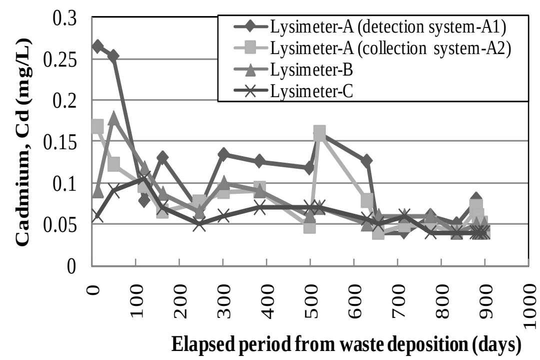

Figure 11: Variation of Cadmium concentration with elapsed period

5.3 Variation of Heavy Metal Concentrations

5.3.1 Cadmium

Cadmium concentration was measured in the range of 0.04 to 0.265, 0.04 to 0.168, 0.04 to 0.178 and 0.04 to 0.105 with in relation to the changing of elapsed period from waste deposition, for leachate detection system (A_1), leachate collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 11. The Cd concentration was found 0.265 and 0.168 mg/L, and after 120 day, it was decreased markedly 0.078 and 0.097 mg/L, for leachate detection (A_1), leachate collection system (A_2), respectively. At the day of 660 to until the end of this trial, there was no significance change for cadmium.

From one way ANOVA test with obtained data it was found that, there was a significant difference with cadmium between the landfills. Using the Post Hoc test, it showed that, lysimeter-B showed significant difference with the lysimeter-A2 and C. No significant difference between the other operating conditions of lysimeter.

5.3.2 Copper

Copper concentration was measured in the range of 0.04 to 0.98, 0.04 to 0.97, 0.04 to 0.76 and 0.04 to 0.60 with in relation to the changing of elapsed period from waste deposition, for leachate detection (A_1), leachate collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 12. At the beginning Cu concentration was found 0.04 and 0.04 mg/L, and after 100 day operation, it was increased rapidly up the day of operation 500, 0.98 and 0.97 mg/L, for the leachate detection (A_1), leachate collection system (A_2), respectively. At the day of 630 and 775 to until the end of this trial, it was decreased rapidly for both the cases of lysimeter-A system.

From one way ANOVA test with obtained data it was found that, there was a significant difference with copper between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_1 showed significant difference with the lysimeter-A_2. Same condition for lysimeter-B and C.

5.3.3 Chromium

At the beginning of lysimeter-A operation, chromium concentration was found 0.065 and 0.06 mg/L, for leachate detection (A_1) and leachate collection system (A_2), respectively. After 120 days, Cr concentration was decreased as 0.02 and 0.02 mg/L for both the cases, and after 120 day to end of this trail, the leachate detection system showing the higher than the leachate collection system.

From one way ANOVA test with obtained data it was found that, there was a significant difference with chromium between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_1, A_2 and C showed significant difference with themselves. Rest of the combinations were insignificant.

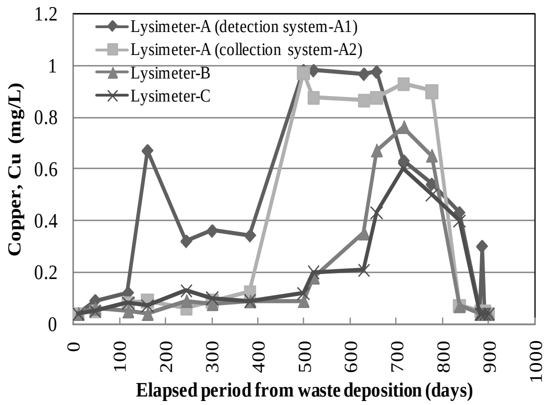

Figure 12: Variation of copper concentration with elapsed period

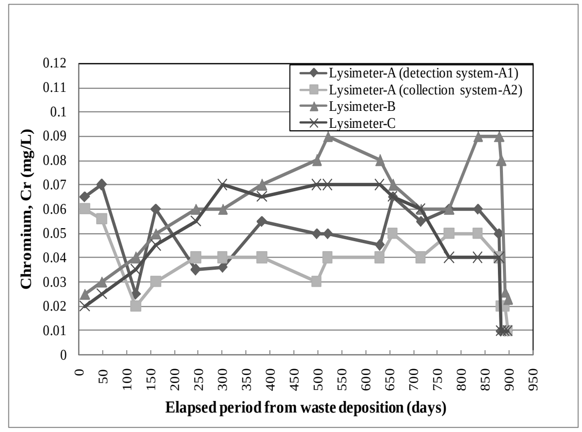

Figure 13: Variation of chromium concentration with elapsed period

5.3.4 Manganese

Manganese concentration was ranging of 0.80 to 6.0, 2.5 to 21.2, 0.50 to 19.3 and 0.80 to 8.0 mg/L, with in relation to the changing of elapsed period, for the leachate detection (A_1), leachate collection system (A_2), lysimeter-B and C, respectively shown in Figure 14. However, the highest concentration of Mn was found for the collection system of lysimeter-A as 21.2 mg/L and both the lysimeter operation showing almost the same concentration of Mn. From one way ANOVA test with obtained data it was found that, there was a significant difference with manganese between the landfills. Using the Post Hoc test, it showed that, except lysimeter-B and C, the rest of the combinations had significant difference.

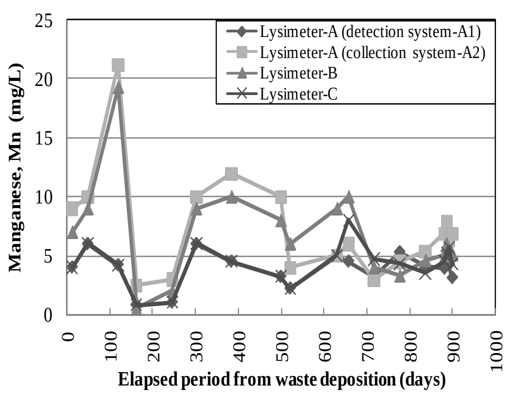

Figure 14: Variation of Manganese concentration with elapsed period

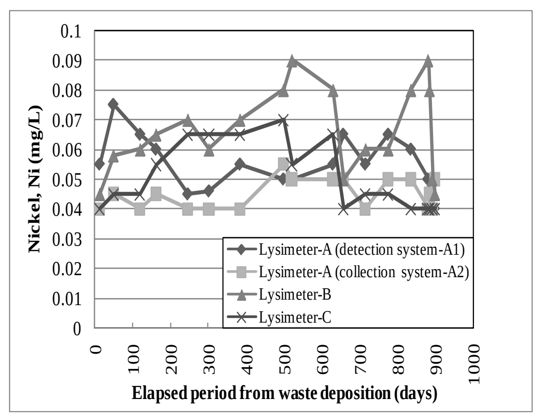

Figure 15: Variation of Nickel concentration with elapsed period

5.3.5 Nickel

Nickel concentration was ranging of 0.04 to 0.075, 0.04 to 0.055, 0.04 to 0.09 and 0.04 to 0.07 with in relation to the changing of elapsed period, for leachate detection (A_1), leachate collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 15. From the beginning of lysimeter-A operation up to the day of 520, nickel concentration increased remarkably and reached to the values of 0.09 and 0.07 mg/L, and after 520 day operation, it was dropped, for lysimeter-B and C, respectively.

From one way ANOVA test with obtained data it was found that, there was a significant difference with nickel between the landfills. Using the Post Hoc test, it showed that, only lysimeter-A1 and C showed significant difference.

5.3.6 Zinc

Zinc concentration was ranging of 0.25 to 1.267, 0.15 to 0.97, 0.10 to 0.55 and 0.10 to 0.576, with in relation to the changing of elapsed period, for leachate detection (A_1), leachate collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 16. From the beginning of lysimeter-A operation up to the day of 380, the Zn concentration was increased rapidly with 0.55 and 0.576 mg/L, for lysimeter-B and C, respectively.

From one way ANOVA test with obtained data it was found that, there was a significant difference with zinc between the landfills. Using the Post Hoc test, it showed that, lysimeter-B and C showed significant difference. Rest of the combinations were insignificant.

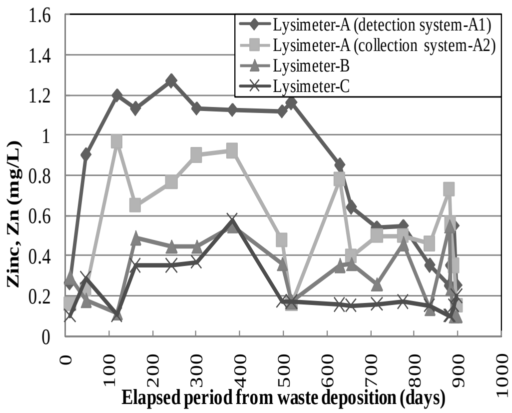

Figure 16: Change of zinc concentration with elapsed period

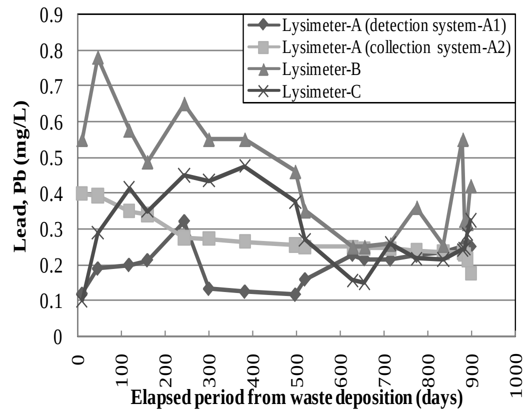

Figure 17: Change of lead concentration with elapsed period

5.3.7 Lead

The lead concentration was ranging of 0.12 to 0.32, 0.12 to 0.397, 0.25 to 0.78 and 0.10 to 0.476 mg/L, with in relation to the changing of elapsed period, for the leachate detection (A_1), leachate collection system (A_2) of lysimeter-A, lysimeter-B and C, respectively shown in Figure 17. The highest concentration of Pb was measured in lysimeter-B with 0.78 mg/L, whereas, the lowest of 0.12 mg/L in both the cases of leachate detection system of lysimeter-A and the lysimeter-C and the concentration of Pb was differed regarding to entire lysimeter operation system.

From one way ANOVA test with obtained data it was found that, there was a significant difference with lead between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_2 showed significant difference with that of lysimeter- C. No significant difference between the rest.

5.3.8 Iron

The values of Iron of leachate in case of lysimeter-C was found as 72.0mg/L which was higher than that of lysimeter-B of 11.2mg/L at the deposition period of 2weeks, then sharply decreased up to the period of 9 weeks for both the cases and after that it was increased up to the elapsed period of 10 weeks then also decreases shown in Figure 18. In case of lysimeter-C, the Iron has changed significantly in the range of 72.0 to 3.33mg/L and for lysimeter-B of 11.2 to 2.22mg/L for the increase of waste deposition period from 2 to 4weeks. The variation of Iron is due to the difference of lysimeter-B and C in terms of thickness and compaction condition of cap liner.

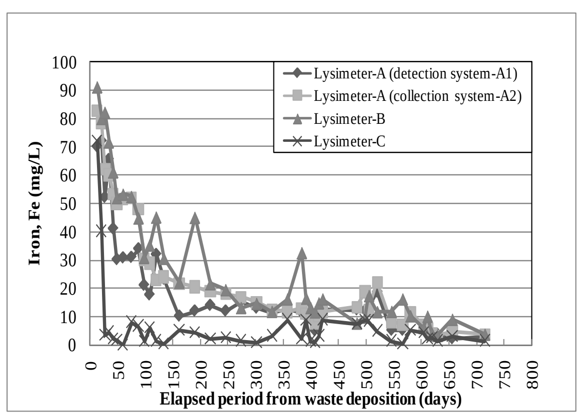

Figure 18: Variation of Iron concentration with elapsed period

From one way ANOVA test with obtained data it was found that, there was a significant difference with lead between the landfills. Using the Post Hoc test, it showed that, lysimeter-A_2 showed significant difference with that of lysimeter- C. No significant difference between the rest.

6. CONCLUSIONS

Based on experimental results, it was observed that leachate quality were varied in relation to the increasing of elapsed period from waste deposition in lysimeter as well as the specific variations of local civil engineering materials used for the construction of landfill lysimeter. Moreover, the result from ANOVA test reveals that leachate qualities were varied significantly with the variation of entire lysimeter operational condition. Finally, it can be concluded that the knowledge of leachate quality will be useful in planning and providing remediameterl measures of proper liner system in landfill design and leachate treatment.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

WASTE PAPER AND PLASTIC RECYCLING – AN INCOME GENERATING ACTIVITY TOWARDS SUSTAINABLE SOLID WASTE MANAGEMENT. A CASE STUDY- KHULNA CITY, BANGLADESH

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ABSTRACT

Management of solid waste has continued to be a major problem and a big challenge in most urban centers particularly in the rapidly growing cities in developing countries. Although the recycling option of waste management has been ignored by the government, the informal sector has been practicing recycling as a source of income since long ago in Bangladesh. This paper presents the findings of a study that was conducted in Khulna city to assess and analyze the existing recycling practice of waste paper and plastic. In spite of health and social problems related with informal recycling, it provides considerable economic profit. The study reveals that the profit margin for paper recycling varies between 40 to 80%, whereas for plastic recycling it varies between 23 to 100%. The study finds that about 26% of the totally generated waste paper and about 69% of the totally generated waste plastic are recycled in Khulna city. Due to this recycling of waste paper and plastic of together 23.85 ton/day, the informal sector is saving the expenditure of solid waste management of 8,586,859 BDT/year. Flow diagrams and mass balances for the existing recycling pattern of waste paper and plastic have been developed in this study.

Keywords: *Recycling, informal sector, income generation, flow diagram, mass balance*

1. INTRODUCTION

The generation of municipal solid waste (MSW) is increasing day by day in Bangladesh as well as in Khulna city. The total MSW generated in Khulna city is about 520 ton/day and its physical composition is food & vegetables 78.9%, paper 9.5%, plastic 3.1%, textile 1.3%, metal 1.1%, glass 0.5%, rubber 0.5%, dust 3.7% and others 1.3% (WasteSafe, 2005). From this composition it is clear that paper (9.5% = 49.4 ton /day) and plastic (3.1% = 16.1 ton/day) are the second and third highest fraction respectively in Khulna city's waste. The quantity of the recyclable solid waste (RSW) is 70 tons/day in Khulna which includes paper, plastic, metal, bone and glass (Moniruzzaman, 2007). From this RSW of Khulna city mainly paper and plastic which are light weight and easily accessible, are recovered by the waste collectors of the informal sector. The informal sector is characterized by small-scale, labor-intensive, largely unregulated and unregistered, low-technology, manufacturing or provisions of services (Wilson et al., 2001). Informal sector entrepreneurs or enterprises do not pay taxes, have no trading license and are not included in social welfare or government insurance schemes (Hann et al., 1998). In the context of solid waste recycling, the informal sector refers to the waste collectors, dealers and recycling industries (RIs).

Waste paper covers a wide variety of different types of used papers such as newspapers, magazines, junk mail, white envelopes without windows, office paper, holiday brochures, catalogues and white directories (e.g. phone books) which can be recycled. However, lack of source segregation results in waste paper getting contaminated and becoming unusable. Recycling of MSW is now recognized as the most environmentally sound strategy by following only a preventive strategy of source reduction and reuse (EPA, 2004).There are mainly three reasons for waste paper recycling. Firstly, waste paper dumped into landfill sites does certainly biodegrade but this produces methane gas that is responsible for global warming. Secondly, recycled paper uses 55% less water than making virgin paper and helps preserve our forests (Gumber, 2004). Thirdly, waste paper recycling actually uses less energy than making virgin paper (According to Gumber 60-70% energy savings over virgin paper production).

Plastic is one of the most commonly used materials in the world. However, its popularity is also its curse. Our landfills are filling with plastic waste that does not biodegrade, and we are using nonrenewable resources to produce more plastic. When plastics are buried in a landfill, they occupy about 25 percent of the space (Museum of Solid Waste, 2006). According to Diamadopoulos et al. (1995), if solid wastes are not recycled, the space in landfills will be exhausted very quickly and necessitate the construction of new ones. Plastics are not the waste and energy culprits that some people think they are. Therefore, recycling and lengthening the lifespan of plastic products is important. Fortunately, plastic recycling has many advantages, and it is one of the easiest ways to be environmentally friendly. The main advantages of plastic recycling are: (1) energy conservation, (2) reduced CO_2 emission, (3) saving landfill space and (4) saving marine life.

Bangladesh is generally faced with the rapid deterioration of environmental and sanitation conditions due to the conventional system of collection, transportation and the crude dumping of MSW. Therefore, urban solid waste management has become a major concern for cities and towns in the country. Many efforts in Bangladesh have only been focused on waste collection and disposal to attain proper solid waste management. However, the recycling activities have been going on traditionally by the informal sector for their livelihood in Bangladesh from times immemorial. This paper looks in brief at the current waste paper and plastic recycling pattern in terms of collection, transportation and transformation in Khulna city. It is also worthwhile to evaluate the income generation from this informal sector recycling existing in the country. It is expected from this research that the scenarios in Khulna city can serve as an example in explaining the traditional recycling pattern not only as a way of sustainable solid waste management but also as an income generating activity of Bangladesh.

2. METHODOLOGY

In order to a get a clear picture of the waste paper and plastic recycling activities a field survey was done all over Khulna city. It was observed that many individual waste collectors, recyclable dealers and recycling industries (RIs) are practicing recycling activity in Khulna. A different set of questionnaires were designed for waste collectors, dealers and the employees of RIs to obtain information about the ongoing waste recycling system. Sixty waste collectors of different ages were selected from all over the study area to take the interview. The amount of waste paper and plastic collected in one day by each of them were weighed. Information about their leaving place, mode of transportation and working conditions was also obtained from the questionnaire survey.

All the dealers were surveyed in this study and most of them participated in the interview. The quantity of waste collection, the buying and selling prices were collected from the record book of each of the dealers. The information about the source of their collection and selling place were obtained from the questionnaire survey. The study used surveys about the employees working in the recycling industry in order to get information concerning problems, prospects, patterns and the quantity of recycling in real life situations. The number of RIs that are producing recycled materials from the processed waste was determined by the field survey. Each of these RIs was surveyed in this study. The quantities of recycled product from waste paper or plastic were obtained from the record book of industries. Information about the prices of RSW was also obtained from the questionnaire survey.

3. INFORMAL SECTOR RECYCLING

The various groups of the informal sector such as waste collectors, dealers and some RIs who do not receive any funding from the government are practicing recycling activities in an unorganized and unplanned way as a source of their income. The informal sector recycling is described as follows.

(a) Sources: Households, institutes, market places are the primary source of waste paper and plastic in Khulna. People are using plastic and paper products for different purpose of their daily life. Waste paper and plastic have economic value. Some portion of this waste is separated by some people at primary source for the purpose of sale.

(b) Waste collectors: The waste collectors are the first link in a long chain of recycling. They are visible in every community of the city and came from nearby slums. This group comprises of men, women and children. Waste collectors are categorized into two groups: House to house waste collectors (usually men and women; locally known as feriwala) and waste bin collectors (usually children; both sexes, age below 15 years; locally referred to as tokai). Generally tokais collect old paper and plastic products from the mixed waste dumped in the waste bin, disposal site and road side as shown in Figure 1. They carry the separated waste in a plastic bag and sell it to the dealers at different prices depending on the type and quality of the waste. They never use gloves and work in unhygienic conditions. This job is identified as the main source of their livelihood. During rainy season their collection quantity is very little. Feriwalas, on the other hand, buy the separated waste paper and plastic items stored for selling in homes and institutions. They purchase recyclable waste which is not contaminated by mixing with biodegradable waste in exchange for money or gifts and sell those materials to the dealers at a small profit.

Figure 1: Recyclable solid waste collection from road side bin by a collector

(c) Dealers: Recycle dealers are the second link in the chain of recycling. They are categorized into three broad groups on the basis of the quantity of waste collection according to Moniruzzaman et al., 2011 as follows. (1) Small Scale Recycling Dealers (SSRD): their collection of RSW is less than 250 kg/day on average, (2) Medium Scale Recycling Dealers (MSRD): RSW collection by each MSRD is within a range between 250 kg/day to 600 kg/day on average and (3) Large Scale Recycling Dealers (LSRD): On an average the amount of RSW collection by each LSRD is greater than 600 kg/day.
The SSRDs purchase recyclable waste from waste collectors'. They sort, clean and sell the processed wastes to the MSRDs. The MSRDs purchase waste paper or plastic from different SSRDs, accumulate the processed waste and sell those to LSRDs. The LSRDs collect waste from all MSRDs, accumulate and sell it to recycling industries (RIs).

(d) Recycling Industries (RIs): Final destination of the materials collected by different actors starting from feriwalas, tokais comes to RIs through a chain of dealers like SSRDs, MSRDs and LSRDs. The RIs recycle the processed waste and finally sell it to market.

4. RESULTS AND DISCUSSIONS

4.1 Waste Paper Recycling

The schematic mass balance of waste paper recycling in Khulna is shown in Figure 2. From this study it was observed that 12.73 ton/day of waste paper (26% of total generated waste paper) were recovered for recycling of which 10.12 ton/day was collected through the waste collectors and 2.61 ton/day was collected by the SSRD directly from sources as shown in Figure 2. In the recycling chain SSRD, MSRD and LSRD were found to reuse paper for writing or covering food or recycle it to paper packets of 7.09 ton/day, 2.44 ton/day and 0.87 ton/day, respectively. The rest amount of 2.33 ton/day was sold to the paper RI for final recycling as shown in Figure 2. Due to lack of source separation and mixing with other biodegradable waste, 36.67 ton/day of waste paper was found to be unrecovered.

There is one paper recycling industry in Khulna. It was found to consume scrap paper to reprocess the paper product. 2.33 ton/day of waste paper such as corrugated cardboard, high grade paper and mixed paper were found to recycle by the RIs. Paper RIs were found to purchase additional post consumer waste paper based on fiber strength**,** fiber yield and brightness according to the type of product produced. RIs purchased waste paper from LSRDs and recycled it by sorting, cleaning, cutting into small pieces, washing, mixing with water and finally by rolling over a roller. From these recycled papers new products such as packet of sweets, hardboard, and bookbinder covers, cartons, shopping bags etc were produced. The flow diagram and the pictures of various steps of waste paper recycling process in the paper RI of Khulna are shown in Figure 3 and Figure 4, respectively.

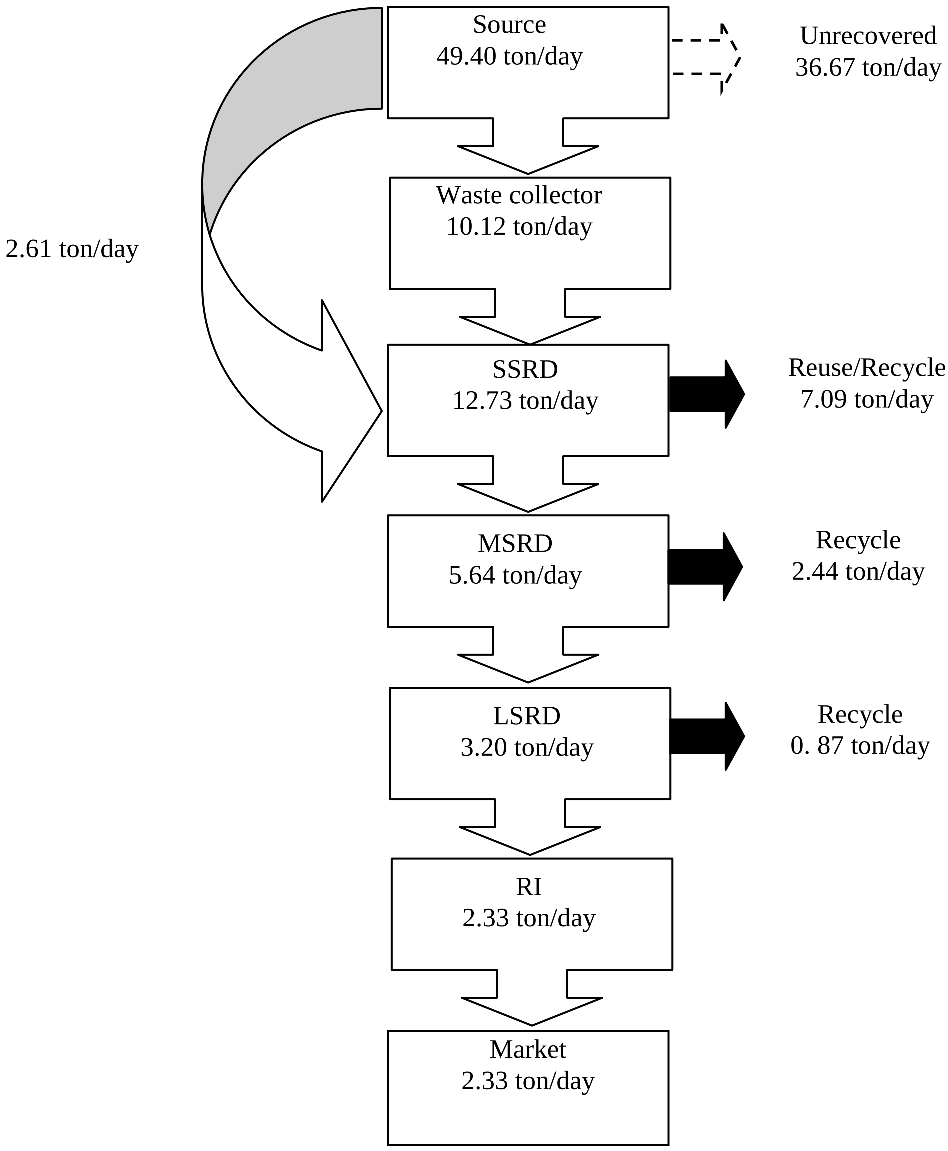


Figure 2: Schematic mass balance for waste paper recycling activity in Khulna

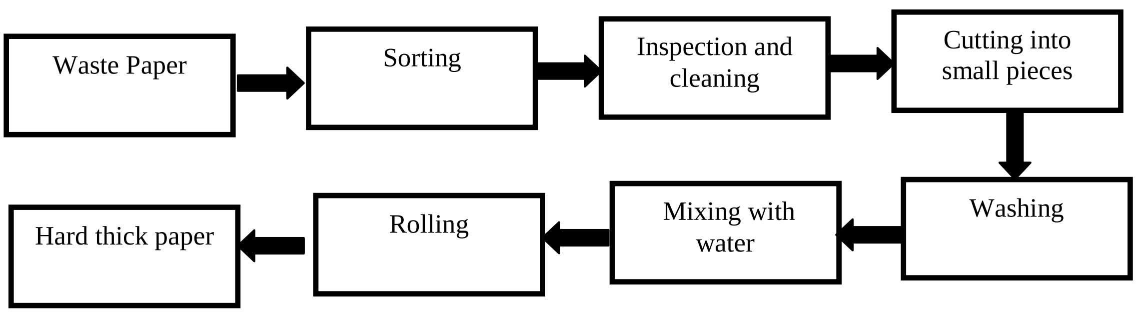


Figure 3: A typical flow diagram for waste paper recycling process in Khulna

(a) Waste paper (b) Cutting (c) Washing

(d) Mixing with water (e) Rolling (f) Product

Figure 4: Waste paper recycling process in a paper recycling industry in Khulna

4.2 Waste Plastic Recycling

The schematic mass balance of waste plastic recycling in Khulna is shown in Figure 5. From this study it was observed that 11.12 ton/day of waste plastic (69% of total generated waste plastic) were recovered for recycling of which 4.85 ton/day was collected through the waste collectors and SSRDs and 6.27 ton/day was collected by the MSRDs directly from sources as shown in Figure 5. Due to lack of source separation and mixing with other biodegradable waste, 4.98 ton/day of waste plastic was found to be unrecovered. In the recycling chain MSRDs were found to reuse plastic bottles for storing water of 3.61 ton/day and sell the remaining processed waste plastic to LSRDs and RIs as shown in Figure 5.

The study finds that there are 11 plastic recycling industries in Khulna and they recycle 7.51 ton/day of waste plastic. The plastic recycling process in RIs in Khulna is as follows.

(a) Sorting: Collected plastics are sorted according to their physical properties such as color, thickness and hardness.

(b) Washing: Sorted or broken materials are washed. The chips are washed using hot water, detergents to remove labels, adhesives and dirt and a centrifugal separator is used to separate the flakes from the dirty water, paper and debris.

(c) Separation: After they are washed, if the mixture does not separate readily a series of hydro cyclones may be required for both light and heavy streams, with the processes tailored to mix of bottles.

(d) Drying: After separation, a spin dryer is used to remove free water and the flakes are then dried with hot air to reduce moisture content to about 0.5 percent.

(e) Coloring**:** After drying, the chips are mixed with colors.

(f) Melting: The flakes are fed into the extruder at large diameter end of the screw and compressed as they are carried toward the extrusion die. The combined heat from flow friction and supplemental heating bands causes the resin to melt and volatile contaminants are vented from the mixture. Immediately before the die the melted plastic passes through a fine screen that removes remaining solid impurities; this step is known as melting.

(g) Making pellet and product: The molten plastic is then formed into strands. The strands are cooled in water, and then chopped into uniform pellets. Pellets are used to form a new product of the designed shape by application of heat in a mould. Varies of necessary products are produced such as toy, water pot etc on the basis of local market demand. Sometimes manufacturing companies of Dhaka buy the plastic pellets from RIs of Khulna to make new products.

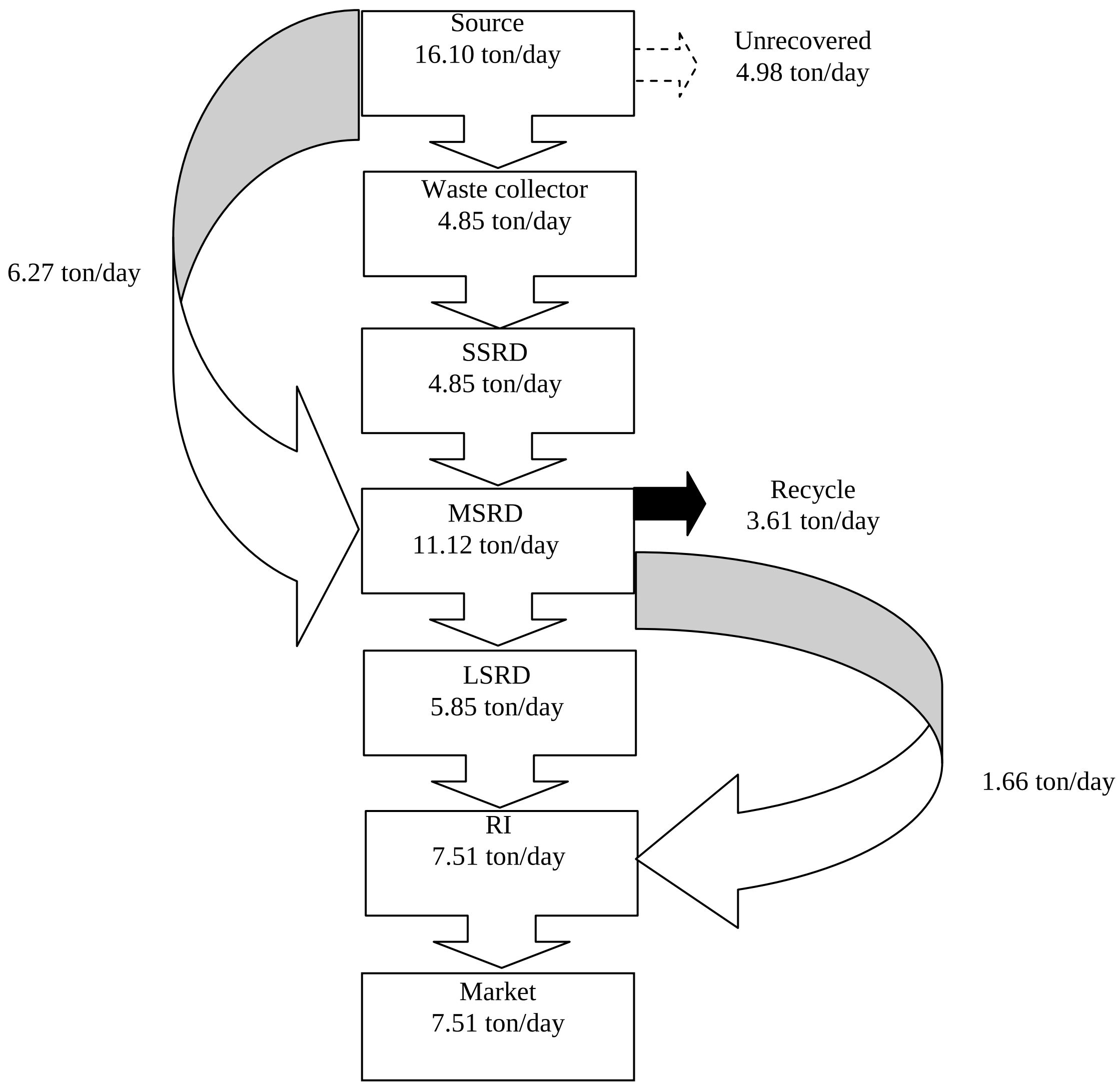


Figure 5: Schematic mass balance for waste plastic recycling activity in Khulna

The flow diagram and the pictures of various steps of waste plastic recycling process in RI of Khulna are shown in Figure 6 and Figure 7, respectively.

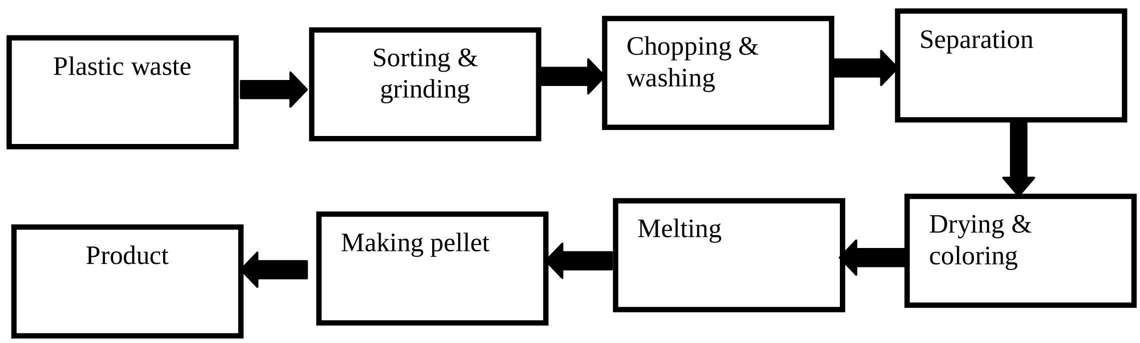
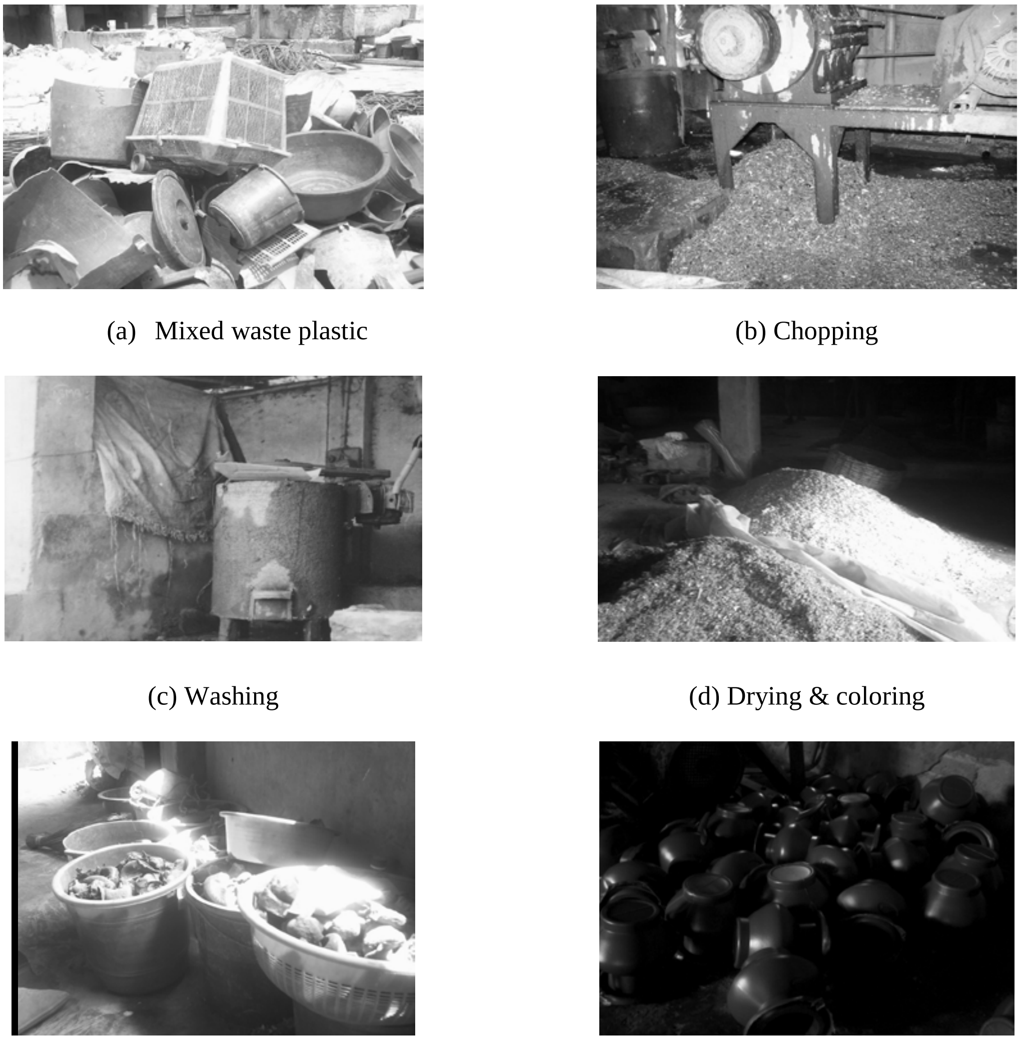


Figure 6:Typical flow diagram of waste plastic recycling process in Khulna



(a) Mixed waste plastic (b) Chopping

(c) Washing (d) Drying & coloring

(e) Making pellet (f) Product (water pot)

Figure 7: Plastic recycling process in a recycling industry in Khulna

4.3 Income Generation and Expenditure Savings

The recycling activity of waste paper and plastic is going on in Khulna not considering those as waste but as a source of income. The prices of separated and processed waste paper and plastic at different dealers and industries were obtained from the field survey as shown in Table 1. The profit margin added to the prices of the wastes in the waste recycling chain from the SSRD level to RI level varies between 40 to 80% for paper recycling and from 23 to 100% for plastic recycling, depending on the recycling potential of the material, its ease of availability and the demand in the market. The highest profit margin occurred in PET bottles, which were found to have a long life span and are readily recyclable and reusable.

The Khulna City Corporation (KCC) is responsible for the waste management in Khulna. KCC spent 288,03,000 taka during the 1998-99 financial year for solid waste management and collected only 80 tons/day. Thus the expenditure per ton per day for solid waste management in Khulna was 986.40 BDT (Enayetullah and Sinha, 2000). It was found that the informal sector was recycling 23.85 tons of waste paper and plastic per day in Khulna. Therefore, it can be estimated that by removing these solid waste, the informal sector is saving revenue 8,586,859 BDT/year.

Table 1: Prices of processed waste paper and plastic at different recycling levels

Type of waste		Price at SSRD (BDT/Kg)	Price at MSRD (BDT/Kg)	Price at LSRD (BDT/Kg)	Price at RI (BDT/Kg)	Profit Margin
Paper	white	10	12	14	-	40%
	mixed shredded paper	6	8	9	10	67%
	cartons and brown packing papers	5	6.5	8	9	80%
Plastic	Bottles	1	1.25	1.5	-	50%
	Hard plastic(containers etc)	22	23	25	27	23%
	PET bottles (mineral water bottles)	5	8	9.5	10	100%

5. CONCLUSIONS

The existence of the recyclable waste, mainly the waste paper and plastic, which are light weight and easily accessible, has opened up a wide possibility for the informal sector in Khulna to utilize it. Although the recycling of solid waste is not included in the waste management policy of local government, it has become a main source of income for many waste collectors, dealers and industries of the informal sector. This informal sector recycling pattern creates a market of recyclable solid waste and value addition occurs for the waste paper and plastic in the recycle stream. The profit margin for paper recycling varies between 40 to 80%, whereas for plastic recycling it varies between 23 to 100%. About 26% of the totally generated waste paper and about 69% of the totally generated waste plastic are recycled daily by the informal sector in Khulna city. Due to this recycling of waste paper and plastic of together 23.85 ton/day, the informal sector is saving the expenditure of solid waste management of 8,586,859 BDT/year.

In order to achieve proper solid waste management and improve recycling rates, one of the major challenges of Bangladesh is how to best effort with the informal sector to improve their working conditions, technologies and efficiency in recycling. Proper storage and source separation system can be adopted for recovering useable and recyclable paper and plastic. Moreover, solid waste management strategies in Bangladesh should be reorganized to include a separate collection and processing system for the recyclable solid waste to avoid mixing with biodegradable waste. That can work parallel with the traditional systems operated by the informal sector for waste separation, processing and final recycling. This approach results into not only the reduction of quantities of wastes to be disposed of but also increase employment and thus income for the disadvantaged urban poor.

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PRESENT SCENARIO OF SECONDARY DISPOSAL SITE FOR MUNICIPAL SOLID WASTE MANAGEMENT IN KHULNA CITY AND OPTIMIZING ROUTES FOR FINAL DISPOSAL USING GIS

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ABSTRACT

Municipal Solid Waste (MSW) is the useless and discarded materials considering the place produced mainly by the anthropogenic activities. With the rapid growth of population and technological activities amount of MSW is also increasing in the cities of developing countries like Bangladesh. Khulna is one of the largest metropolitan and industrial cities of Bangladesh. But in this city, there is no systematic time and route plan for the storage, collection and disposal of MSW from secondary disposal site (SDS) to the ultimate disposal site (UDS). There are more than 40 (forty) SDSs are available from where Khulna City Corporation (KCC) vehicles are collected the MSW and approximately 1200 community bins, located on roadsides throughout the city. KCC places the 26 (twenty-six) Haul Containers (HCs) and some permanent concrete/masonry of different wards in this city. This research concentrates to investigate the present situation of SDSs and to develop an optimized route plan for collection and disposal of MSW from SDS to UDS using GIS. To investigate the existing situation of SDSs questionnaire, interview and ovservation surveyes were conducted and the feedback was recorded by the author. Moreover, transportation facilities, types of vehicles were investigated and finally Lat-Lon of SDSs, KCC garage and UDS were recorded using GPS. Here, After collecting all the relevant information from questionnaire survey then analyzed using SPSS and optimized route plans were developed in ArcGIS using Network Analyst tool. Therefore an attempt is made to propose a sustainable plan through which the existing facilities are properly utilized by providing an appropriate route plans having minimum travel distance which contribute to ensure effective MSWM in Khulna city.

Keywords: *Secondary disposal site, municipal solid waste, geographic information system (GIS),network analyst, optimized route.*

1. INTRODUCTION

Khulna is the third largest metropolitan city and famous industrial city of Bangladesh. It is located in the south-western part near the Sundarbans, the largest mangrove forest of the world (Alamgir et al. 2005; Rafizul et al. 2009), shown in Figure 1. It is situated below the tropic of cancer, around intersection of latitude 22.49º north and longitude 89.34º east (KCC 2009). Total area of KCC is 47.0 sq. km. comprising 31 wards (BBS, 2001). It lies along the Bhairab River. The city stands on the bank of Rupsha and has an important river port. It is connected by river, road and rail to the major cities of the southern Gangetic delta. It is a linearly developed city extending from southeast to northeast along the Bhairab-Rupsha River and Khulna-Dhaka road.

MSW collections in Khulna city are mainly done by Khulna City Corporation (KCC), Non-government organization (NGOs) and community based organization (CBOs) (Ahsan et al. 2005). Door-to-door collection systems are adopted for MSW collection from generation sources and then dispose major portion of it to the nearest SDS. From there KCC vehicles collect the MSW and transport it to the UDS. Some NGOs transfer their collected organic wastes to composting plant. Whilst KCC has some limited numbers of non-motorized vehicle those are mainly used for the collection of MSW from community bins located at roadside, home side, near market and transfer to SDS. But there is no route plan to ensure effective and time saving MSW collection. Regarding to this, this study investigates the present situation of SDSs by questionnaire, interview and observation survey which was conducted and ten the feedback was recorded. Furthermore, transportation facilities, types of vehicles were investigated and finally the co-ordinates of SDSs, KCC garage and UDS were recorded using GPS. Aspatial data were analyzed using SPSS and optimized route plans were developed in ArcGIS using Network Analyst tool.

Figure 1: Location of Khulna city as study area

2. PRESENT SCENERIO OF MSW MANGEMENT IN KHULNA CITY FROM OVSERVATION

KCC generally manages the MSW and recently, some NGOs, CBOs and private organization are working for door-to-door collection (Rafizul et al. 2009). Door-to-door collection systems are introduced recently for MSW collection, mainly from households, and then dispose major portion of it to the nearest SDS (Alamgir et al. 2005). KCC collects these MSW and transfer it to the UDS and they are facing very complicated situations for the management MSW. Due to severe financial constraints, lack of motivation, absence of effective legislation to protect the environment, absence of route plan, the MSW has becoming a threat for city dwellers, planners and other concerned stakeholders (WasteSafe 2005). The management tiers of MSW can be categorized and hence discussed briefly in followings.

2.1 Primary collection

MSW are collected from generation sources by NGOs, CBOs and KCC by door-to-door collection systems and most of the cases owner by himself disposes it to the nearest community bins/SDS/open land/road sides/drains (Ahsan et al. 2005). Whilst KCC has some limited numbers of non-motorized rickshaw vans and hand trolley those are mainly used for the collection of MSW from community bins and transfer to SDS.

2.2 On-site storage

On-site storage is the SDS and transfer station which receives MSW from primary source and transfer to the designated location for processing/recycling/treatment and mostly for ultimate disposal (Jahan et al. 2011). The wide variety of types and shapes of community bins are built by the civic bodies and/or KCC, which are located on the roadsides at frequent intervals. The present scenarios of common types of SDS for MSW in Khulna city is shown in Figure 2.

2.3 Secondary collection and transportation

The functional element of collection includes not only the gathering of MSW and recyclable but also the transportation after collection, to the location where the collection vehicle is emptied (Chowdhury et al. 2008). KCC is responsible for collecting MSW from SDS and transported it by motorized vehicles/trucks and finally disposed in the designated UDS. Although NGOs and CBOs collect MSW from households/ generation points and dump it to the SDS, they do not take responsibility for the collection & transportation of MSW, which they deposited in the nearby SDS (Ahsan et al. 2005). Conservancy department of KCC setup the time-schedule and types of vehicle for collection and transportation of MSW. Generally, collection vehicles such as dump truck, normal truck, open truck, tractor with trolley, tipping truck (container carrier), de-sledging vacuum tanker with tractor, power tiller with trolley stands in the road nearest to the SDS for operation. Typical collection methods from SDS and MSW heaped transport vehicles because there is no optimized route plan for the vehicles.

Figure 2: Present scenario of common types of SDS in Khulna city

3. PRESENT SCENARIO OF SDS FROM QUESTIONNAIRE SURVEY

A direct questionnaire and interview survey have been conducted among 200 responsible people, local leaders, stakeholders' and related organizations to investigate the present situation of SDSs in and around Khulna city with a pre coded questionnaire. After collecting the relevant feedback from survey then analyzed through SPSS. Study on the responsible people, local leaders, stakeholders' and related organizations about their perception and feedback to SWM system at SDSs and the way other people respond to it. In case of sample selection this survey was conducted with the decision to choose at 31 SDSs of Khulna city giving priority to the affected communities. A 5% of the total households from each area were covered by the study. For questionnaire survey all the SDSs were numbered randomly and the total number of questionnaire administered was 200. Answers from city dwellers and other stakeholders were recorded and hence analyzed using SPSS. Result reveals that among 200 about 124 (62%) people has SDS, 76 (38%) has no SDS near their area which is presented in Figure 3.

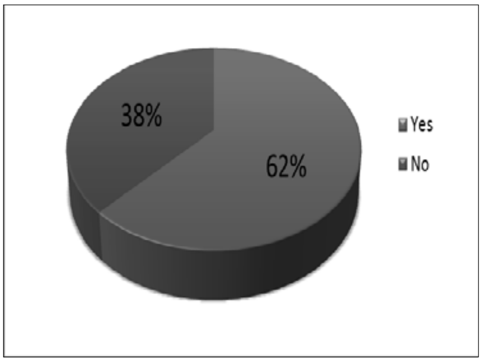

Figure 3: Presence of SDS near respondent's area

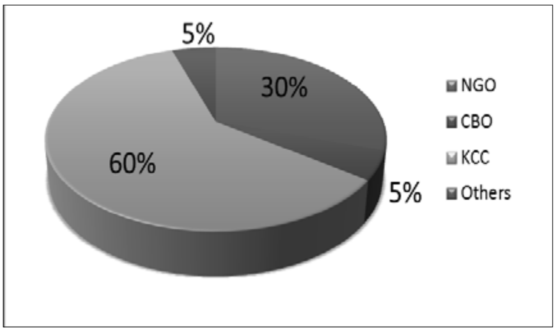

Figure 4: Collectors of MSW

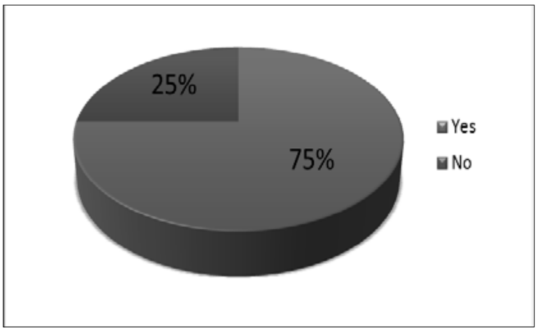

Figure 5: Presence of dustbin within 500m

From the survey it is noticed that, NGO collects 60 (30%), CBO 10 (5%), KCC 120 (60%) and others 10 (5%) of MSW from the household as shown in Figure 4. Here it is interesting to note that 150 (75%) has dustbin, where 50 (25%) has no dustbin within 500m of their locality as shown in Figure 5.

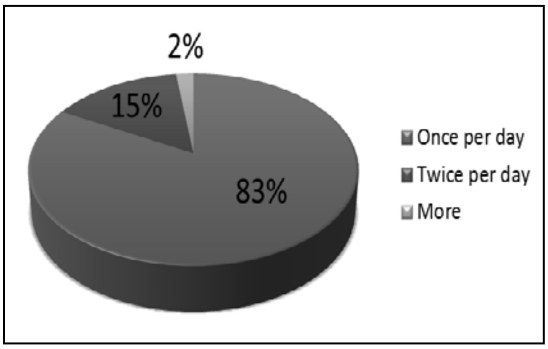

Figure 6: Collection schedule of MSW by KCC

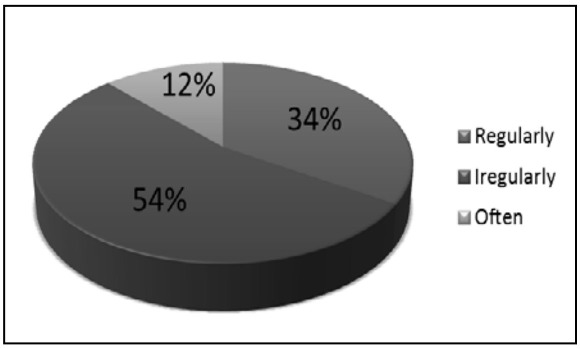

Figure 7: Cleaning schedule of SDS by KCC

The survey result also reveals that KCC collects MSW from SDS 166 (83%) once per day, 30 (15%) twice per day and 4 (2%) more shown in Figure 6 and KCC clean SDS regularly said by 68 (34%) people, irregularly by 108 (54%) and often by 24 (12%) respondents as shown in Figure 7. It is interesting here that, present situation of SDS is very good only said by 20 (10%) people, good 20 (10%), average 110 (55%) and bad 50 (25%) people as shown in Figure 8. That means most of the people are unsatisfied to MSW system.

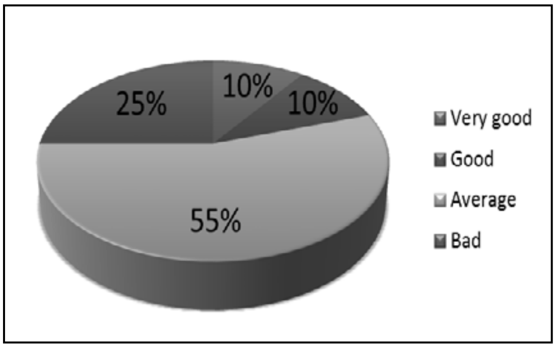

Figure 8: Satisfaction about collection of MSW from SDS

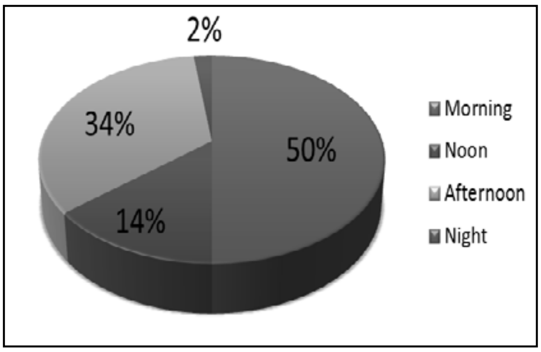

Figure 9: Collection time of MSW from SDS by KCC

Furthermore, based on questionnaire survey it can be depicted that, KCC collects waste from SDS was said at morning by 100 (50%) people, at noon 28 (14%), afternoon 68 (34%) and said at night by 4 (2%) people as shown in Figure 9. However, 86 (43%), 32 (16%), 60 (30%) and 22 (11%) against at morning, noon, afternoon and night, respectively, responsive that KCC should collect the MSW from SDS at the proposed time i.e most of the people wants that KCC should collect the MSW from SDS at morning shown in Figure 10. Moreover, biggest problem around SDS in and around Khulna city is odour was said by 90 (45%) people, dirty streets 56 (28%), rubbish heap 40 (20%) and was said flies by 14 (7%) people as shown in Figure 11.

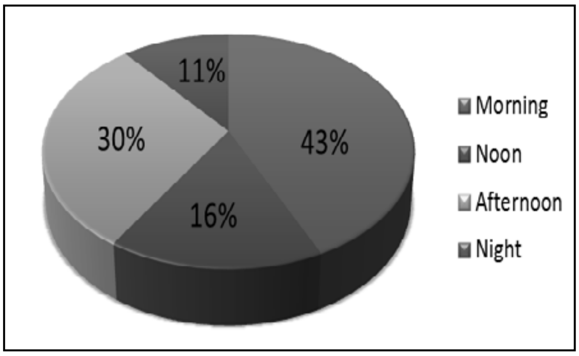

Figure 10: KCC should collect MSW from SDS

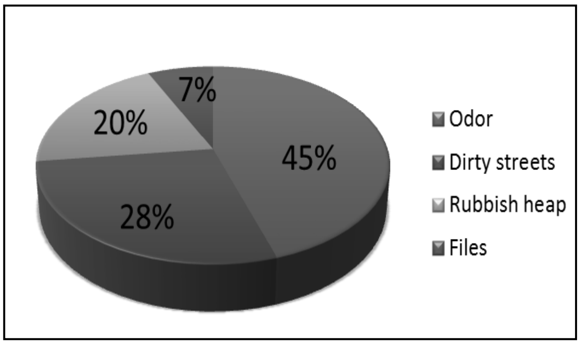

Figure 11: Problems around to SDS

Here, it is interesting to note that questionnaire survey reveals for improving the present situation of SDS people said it is possible through 44 (22%) KCC, 40 (20%), 88 (44%) and 28 (14%) people and shown in Figure 12. Moreover, type SDS is better in and around Khulna city was responded that 30 (15%) container, 70 (35%), 82 (41%) and 18 (9%) as shown in Figure 13.

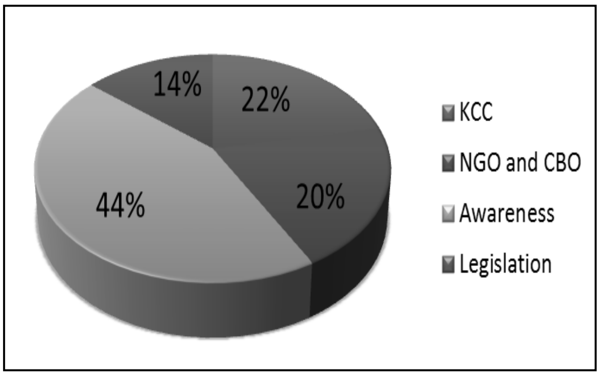

Figure 12 How to improve the present situation of SDS

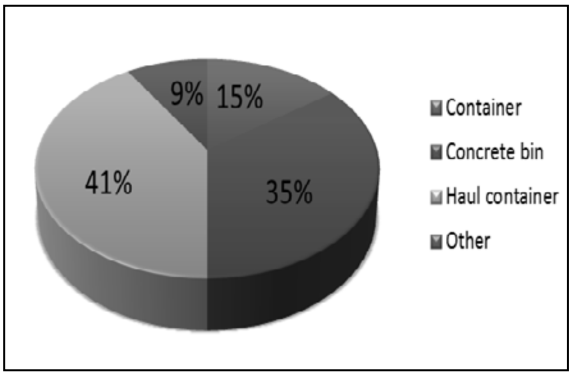

Figure 13 Which type of SDS is better

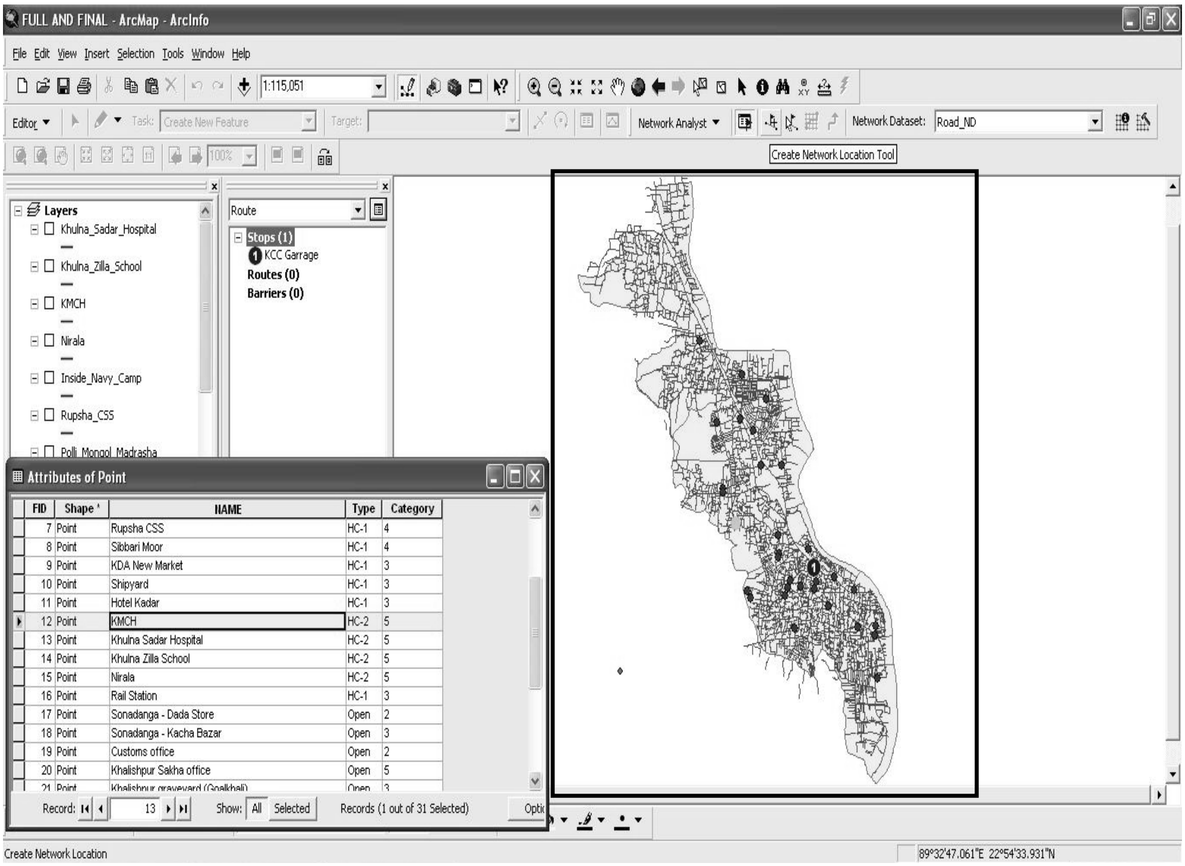

Figure 14: Case study area map with vector data in ArcGIS

4. OPTIMIZING ROUTES FOR COLLECTION AND DISPOSAL OF MSW USING GIS

(Geographic Information System) GIS can make the analysis of the situation along with a future trend. In MSW system, GIS proposes the collection of data from different sources and formation of map layers like, MSW generation map of different areas, existing MSW collection and disposal pattern from SDS to UDS, existing and optimized route plan of MSW lifting, etc (Ghosh et al. 2005; Thanh et al. 2009). There are many suggestions in this spatial planning proposal which are said to be considered while working in GIS, they are (i) identification of exact location of MSW bins with GPS demarcating on the base map; (ii) maintaining a record of SDSs; (iii) a map showing the collection and disposal routes in different areas; (iv) a map showing the distances between the bins; (v) location of the SDSs; (vi) record of available vehicles and equipment for MSW management; (vii) allocating a unique number to all the SDSs so it can be easily and quickly located in case of any complaint registered or planning and maintenance; (viii) maintaining a record about the type of SDSs and (ix) record of the responsibilities and assignment of work, equipment's, vehicles etc. of the MSW maintenance and also the logistics information about the transportation involved in the system.

4.1 Case study area map in GIS

The case study map is the form of GIS. The detailed map of case study area had the information about the boundary and detailed road network stream. These maps were imported in the ArcMap of ArcGIS and geo-referenced perfectly. Different information was spatially located on the maps in point; line and polygon features and the database were updated with new data. The detailed base map of case study area with vector data information in different features is seen in Figure 14.

FID	Shape *	NAME	Type	Category
12	Point	KMCH	HC-2	5
13	Point	Khulna Sadar Hospital	HC-2	5
14	Point	Khulna Zilla School	HC-2	5
15	Point	Nirala	HC-2	5
20	Point	Khalishpur Sakha office	Open	5
30	Point	Inside Navy Camp	HC-2	5
7	Point	Rupsha CSS	HC-1	4
8	Point	Sibbari Moor	HC-1	4
0	Point	Goalkhali Navy Collony	HC-1	3
1	Point	Goalpara Electricity Plant	HC-1	3

Figure 15: Attributes and sample data of SDS in ArcGIS

4.2 Spatial and Attributes

GIS data is any type of information that holds both the spatial information and attributes. GIS data is used in this way to store information about large geographic areas. For example, a municipality will likely have property information, illustrating individual land parcels or lots geographically. For each one of these parcels, the municipality will also store attribute information such as the contact information of the owner and the street address of the home. When using shape files, the data can be presented in 3 different forms: points, lines and polygons. For this municipal example, land parcels will be represented by polygon data. In this study, the SDSs were fed as points. Figure 15 represents attributes of SDS and some sample data on SDS in ArcGIS where SDS is located by point.

4.3 Route Optimization by Network Analyst in GIS

Network analyst is a popular analysis technique for route optimization. Network Analyst can discover the obvious way to get from one location to another or the best way to visit several locations. The locations can be specified interactively by placing points on the screen, simply by entering an address, or by using points in an existing feature class or feature layer. The very best route can be determined for the order associated with locations as specified by the user. Alternatively, Network Analyst can determine the best sequence to visit the locations (Ramasamy et al. 2003; Vijay et al. 2005). Whether finding a simple route between two locations or one that visits several locations, people usually try to take the best route. Hence, the best route can be defined as the route that has the lowest impedance, where the impedance is chosen by the user. Hence, some conditions were place into consideration for route optimization of (i) starts from KCC garage, collect MSW from SDS(s) and stops at UDS; (ii) optimum travel distance; (iii) least road requirement for vehicle movement; (iv) specific vehicle for specific type and category of SDS and (v) then routes were optimized using network analyst of ArcGIS. Here, it is interesting to note that the information about the optimized route of SDS at different study area to UDS with recommended vehicle is also presented in Table 1. Furthermore, some sample optimized routes using network analyst of ArcGIS of SDS at Hotel millennium, railway station, KMCH, Sibbari moor, Rupsha bus stand, Boyra area, Nirala, Khualishpur Sakha Office, KDA new market and Khulna shipyard to UDS are presented in Figures 16 to 25, respectively.

Table 1: Summery of optimized route for SDS to UDS with vehicle recommendation

SDS Location	Type	Route length (m)	Recomeded Vehicle
Khulna Medical College Hospital	HC-2	11507	Double HC
Khulna Sadar Hospital	HC-2	9132	Normal
Khulna Zilla School	HC-2	10454	Double HC
Nirala	HC-2	8217	Double HC
Khalishpur Sakha office	Open	15460	Normal
Inside Navy Camp	HC-2	13458	Double HC
Rupsha CSS	HC-1	12381	Single HC
Sibbari Moor	HC-1	8823	Single HC
Goalkhali Navy Colony	HC-1	17126	Single HC
Goalpara Electricity Plant	HC-1	18737	Single HC
Banorgati	HC-1	7791	Normal
Polli Mongol Madrasha	HC-1	7892	Normal
Hotel Millennium	HC-1	9110	Normal
PTI Moor	HC-1	9262	Single HC
KDA New Market	HC-1	9796	Single HC
Shipyard	HC-1	14840	Single HC
Hotel Kadar	HC-1	8026	Normal
Rail Station	HC-1	9063	Normal
Rupsha Bus-stand Moor	Open	12160	Normal
Near Islam Commissioner's House	Open	7791	Normal
Tutpara	Open	11692	Small
Boyra area (Bazar Area + Police Line)	Open	13801	Normal
Sonadanga Kacha Bazar+Dada Store)	Open	10188	Normal
Goalkhali + Doulatpur	Open	21288	Normal
Khalishpur + Customs office	Open	17467	Normal
Khan Jahan road + Shaheber Kobor Khana	Open	8145	Normal

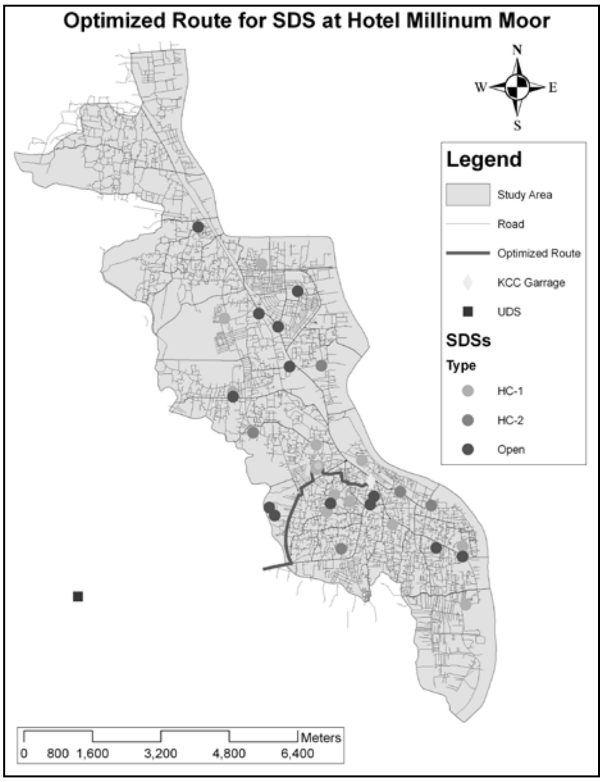

Figure 16: Optimized route of SDS at Hotel Millennium moor

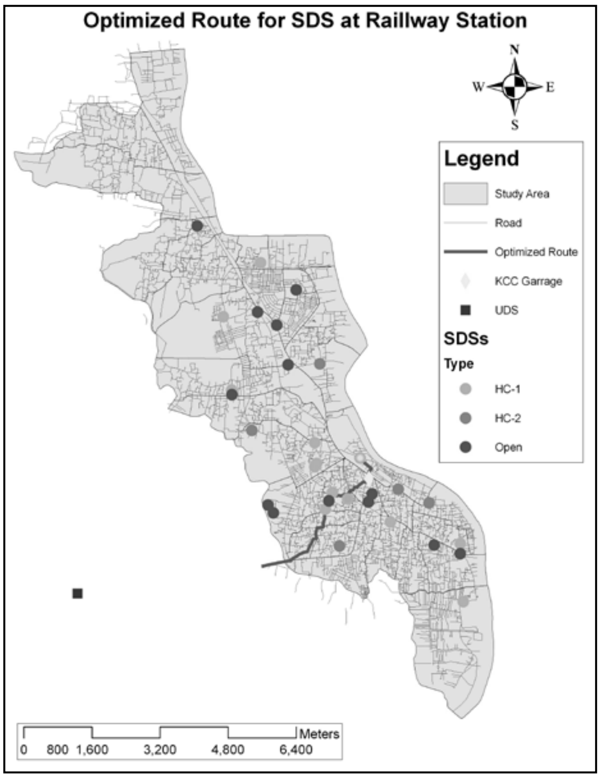

Figure 17: Optimized route of SDS at Railway station

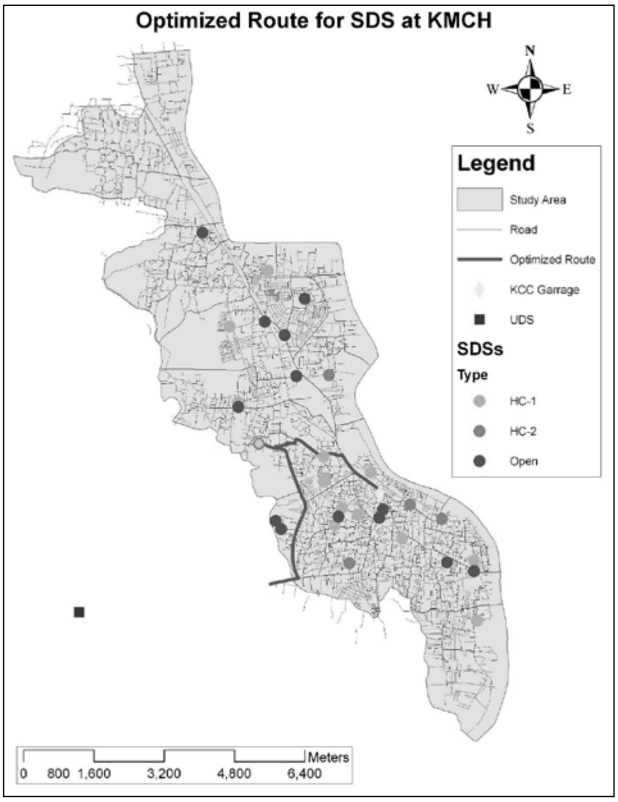

Figure 18: Optimized route of SDS at KMCH

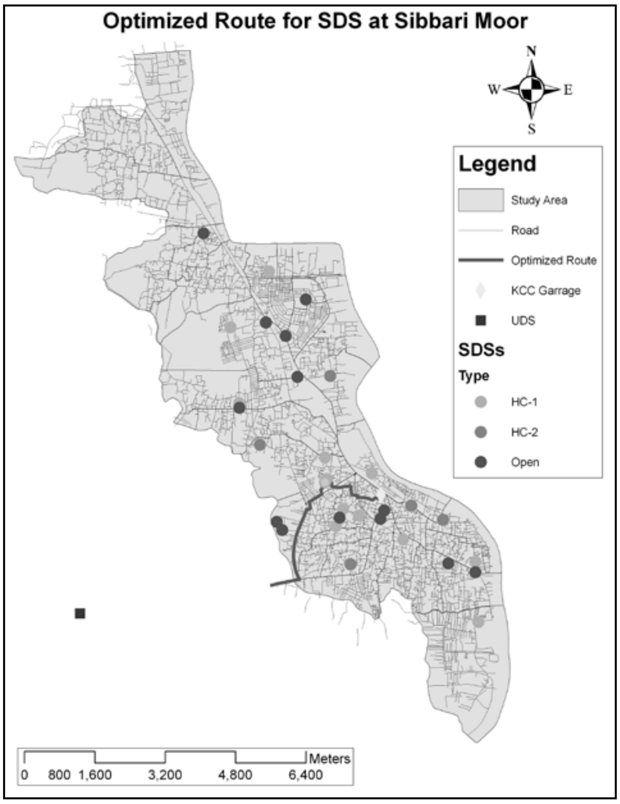

Figure 19: Optimized route of SDS at Sibbari Moor

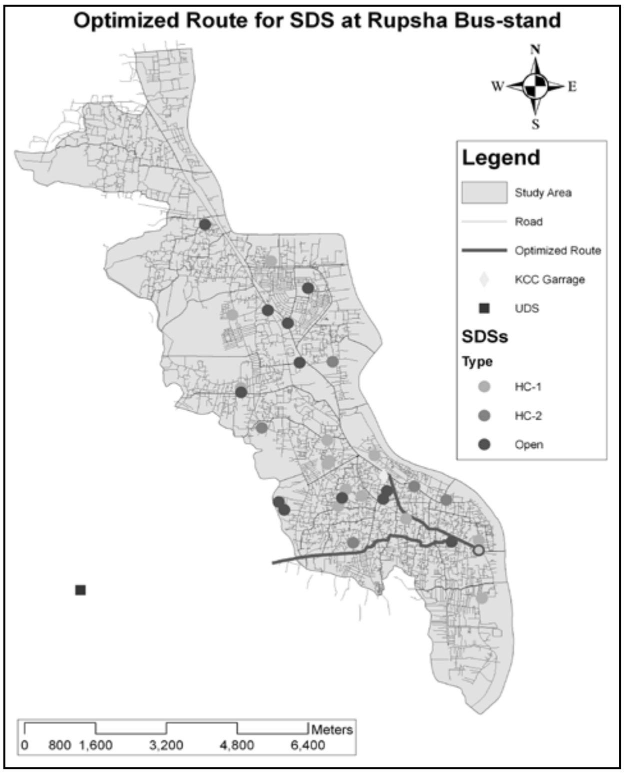

Figure 20: Optimized route of SDS at Rupsha bus stand

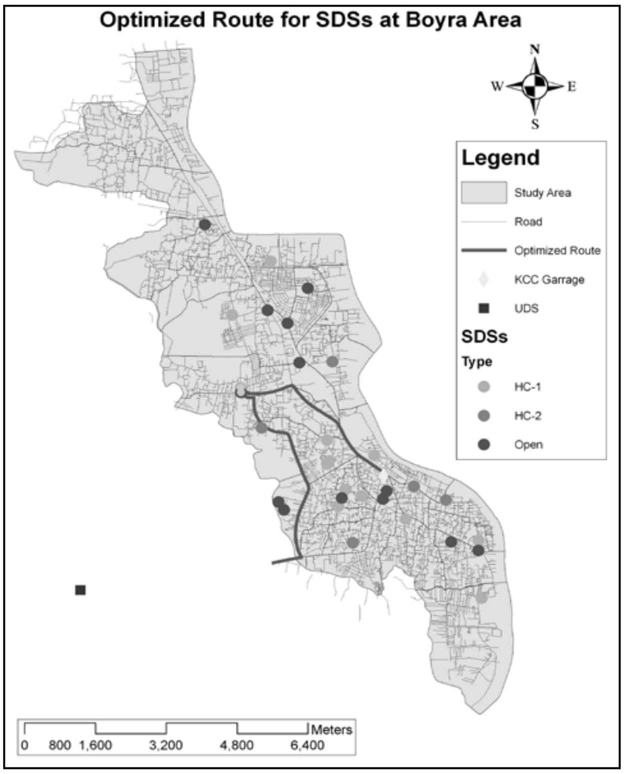

Figure 21: Optimized route of SDS at Boyra area

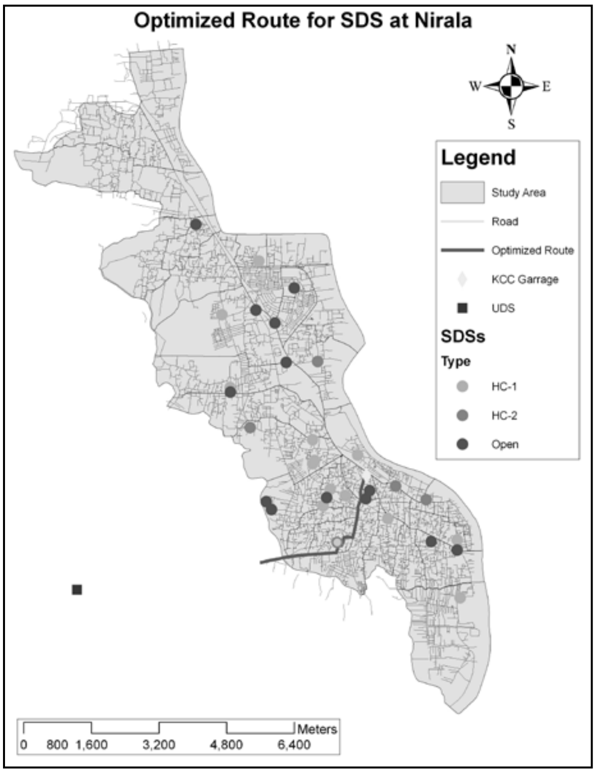

Figure 22: Optimized route of SDS at Nirala

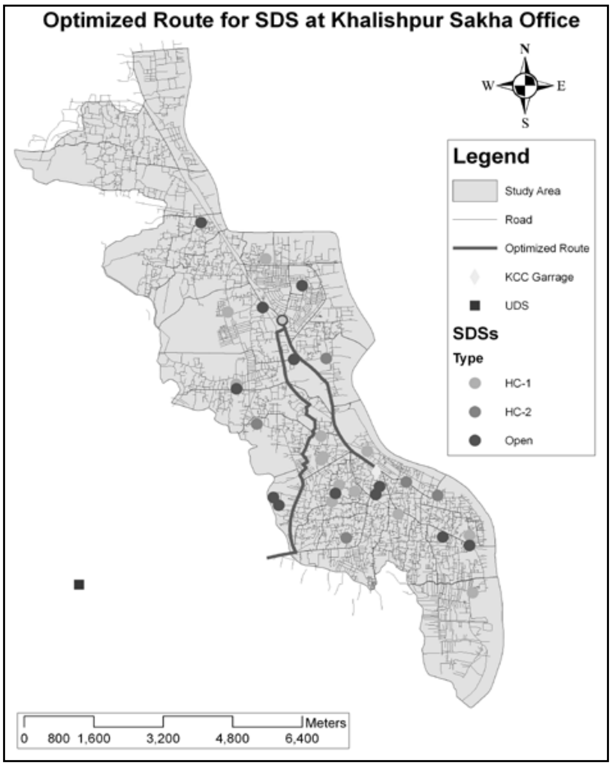

Figure 23: Optimized route of SDS at Khalishpur Sakha Office

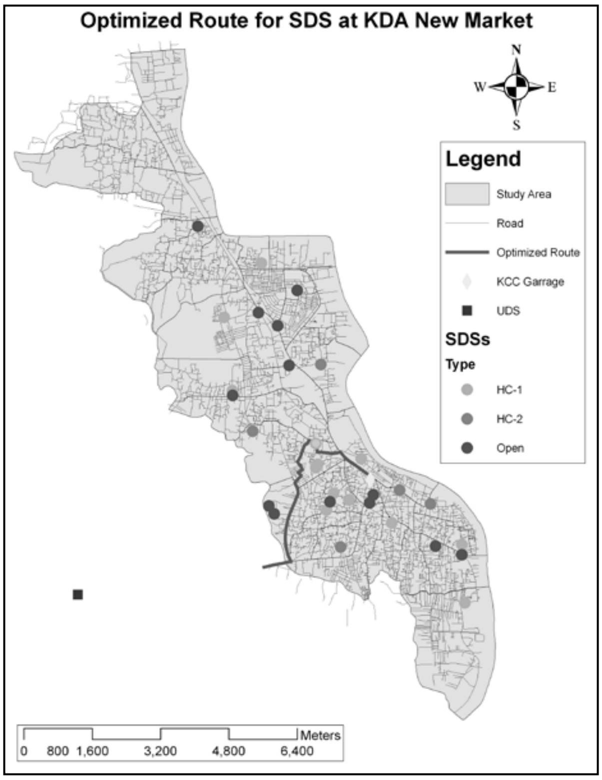

Figure 24: Optimized route of SDS at KDA new market

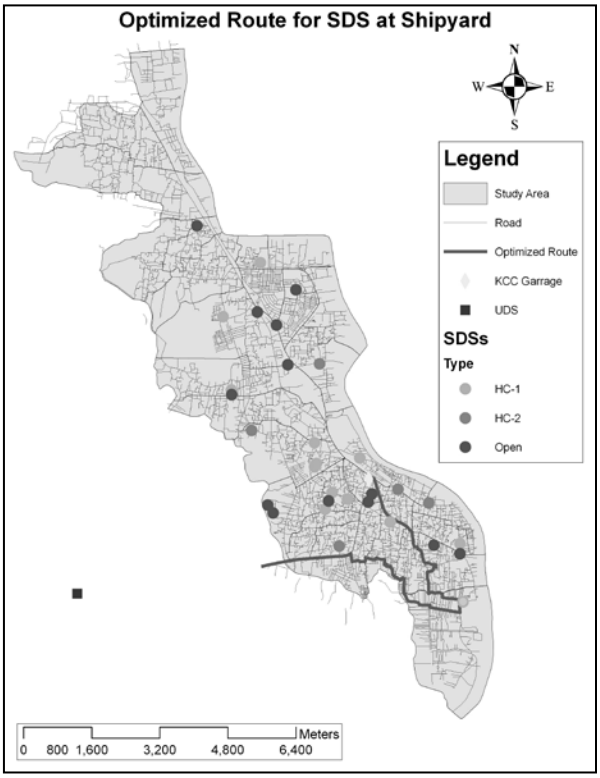

Figure 25: Optimized route of SDS at Khulna Shipyard

5. CONCLUSIONS

This study intends to identify the optimized collection and disposal route of MSW from SDS to UDS in Khulna city. There are different routes for the three categories of SDSs and four types of MSW collection vehicles. These statistics were given to have the idea of the new MSW management system by using the proposed route network. It can be a applied tool for MSW transport, fuel consumption, work distribution amongst the vehicles for load balance and generation work schedules for both employees and vehicles. But KCC should follow these routes and should support these routes by necessary manpower and equipments. They should maintain the optimized routed to ensure effective collection and disposal of solid wastes. Furthermore, they should consider the arrangement of backup support which will help to ensure continuous collection and disposal in emergency through the optimized routes and further development of route optimization using GIS.

ACKNOWLEDGEMENTS

Authors would like to express thanks to all the persons who helped in this study by giving data and information.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ANALYSIS OF LEACHATE POLLUTION INDEX AND FORMULATION OF SUB-LEACHATE POLLUTION INDICES OF SOLID WASTE LANDFILL IN BANGLADESH

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ABSTRACT

The principal concern of this study is to concentrate the contaminant potential of solid waste landfill in Bangladesh. To these endeavors, leachate samples from four selected solid waste landfill namely Dhaka, Chittagong, Khulna and Rajshahi were sampling and the relevant parameters required for evaluating the leachate pollution index (LPI) were measured and monitored in the laboratory. Three sub-leachate pollution indices(sub-LPIs) in terms of LPI in organic pollutant (LPI_{or}), LPI in inorganic pollutant (LPI_{in}) and LPI in heavy metal (LPI_{hm}) as well as overall LPI had been developed and reported by the author. Result reveals that LPI_{or} was ranging from 56.22 to 36.86 (Dhaka); 42.43 to 30.41 (Rajshahi); 50.98 to 35.12 (Khulna) and 55.30 to 37.54 (Chittagong), while LPI_{in} was ranging from 50.57 to 34.5 (Dhaka); 40.27 to 29.8 (Rajshahi); 50.21 to 37.19 (Khulna) and 50.04 to 34.54 (Chittagong); as well as LPI_{hm} 40.4 to 27.5 (Dhaka) 35.58 to 25.78 (Rajshahi); 34.22 to 24.9 (Khulna) and 38.6 to 27.5 (Chittagong). Moreover, it depicts that overall LPI was ranging 50.01 to 30.68 (Dhaka); 38.48 to 25.20 (Rajshahi); 35.18 to 13.45 (Khulna) and 40.43 to 25.41 (Chittagong) solid waste landfill. Results showed that Dhaka landfill had the highest sub-LPIs and overall LPI than that of other studied landfills. Finally, it can be depicted that overall LPI was significantly higher for all the selected landfill and proper treatment will be necessary before discharging the leachate into the water bodies.

Keywords*: Landfill, contaminant potential, aggregation function, leachate pollution index, sub-leachate pollution indices.*

1. INTRODUCTION

Landfill is the most common waste disposal method in many countries for solid wastes, and it is a basic requirement in all countries. As a result of the serious environmental problems associated with abandoned dump sites and the high costs of clean-up measures to deal with the contaminated sites, almost all countries have introduced regulations to safeguard the water aquifers from the leachate generated from the landfills (Rafizul et al. 2011). Leachate is characterized by its high content of organic constituents, metals, acids, dissolved salts and microorganisms (Fueyo et al. 2003; Orta de Velasquez et al. 2003). Containing hundreds of different chemicals, the characteristics of landfill leachate vary significantly with respect to its composition, volume, and the presence of the biodegradable matter and with time (Anderottola and Cannas 1992; Chu et al. 1994). Leachate constitutes a flow that is highly aggressive and dangerous to the environment, with a contamination potential exceeding that of several industrial-waste materials (Orta et al. 1999). The use of indices in the management of the ecosystem is well accepted. It allows for the representation of a complex set of information about ecosystem variables in a simple fashion (Zandbergen and Hall 1998). A technique to quantify the leachate contamination potential of landfills on a comparative scale by using an index known as the leachate pollution index (*LPI*) has been developed and reported elsewhere (Kumar and Alappat 2003).

A stock market index is a good indicator of the overall performance of the market. However, an individual with a Stock portfolio will be more interested in the performance of individual stocks or stock sectors. In a fashion similar to the stock market index, the LPI provides an overview of the leachate contamination potential of a landfill on a comparative scale. However, it fails to communicate the dominating pollutants present in the leachate as for performance of individual stocks or stock sectors in the stock market. In an effort to effectively communicate the dominating pollutants present in a leachate sample, it was decided to subgroup the pollutants considered in the leachate pollution index. The formulation and applications of the sub-indices of the LPI (sub-LPIs) and the overall LPI is presented in this paper. Using the leachate composition of a municipal landfill site in UK, obtained from literature (Last et al. 199), three sub-LPIs and the overall LPI have been calculated and are reported.

2. MATERIALS AND METHODS

In this study, to derive the individual and overall pollutant rating of four selected solid waste landfill namely Dhaka at Matuail (sanitary landfill), Chittagong (open dump), Khulna at Rajbandh (sanitary landfill) and Rajshahi (open dump) in Bangladesh based on *LPI* as well as sub-LPI, the detailed procedure advocated by Kumar and Alappat (2003) was followed and hence discussed in followings.

2.1 Concept of Leachate Pollution Index

In an effort to develop a system for comparing the leachate pollution potential of various landfill sites in a given geographical area, 80 panelists consists of academicians in environmental science and engineering; environmental regulatory authority officials and scientists; consulting engineers; and members of International Solid Waste Association (ISWA) conducting the necessary survey around the world. The survey was conducted using multiple questionnaires to formulate LPI based on Rand Corporation's Delphi Technique (Dalkey 1968).

2.2 Pollutant Variables Selection

From literature, *fifty (50)* commonly reported leachate parameters were selected for their possible inclusion in the LPI. *In questionnaire 1*, the panelists were introduced to the possibility of preparing a tool in the form of *LPI*. Moreover, the panelists were also asked to consider *fifty (50)* leachate parameters for their possible inclusion in the proposed LPI. Panelists were also requested to add any variables to the list of *fifty (50)* parameters, which, they feel shall also be included in the *LPI*. They were also asked to designate all the parameters as follows: (i) do not include (ii) or undecided and (ii) or include.

Panelists were also requested to rate each parameter marked 'include' according to the significance of its contribution to overall leachate pollution. The rating was to be done on a scale of *'1' to '5'*. The value *'1'* and '5' were to be used for the parameter that has lowest and highest relative significance to the leachate contamination, respectively. The *eighteen (18)* selected pollutants and the significance obtained for them are given in Table 1.

Table 1: Significance and weights of the pollutant parameters *(after Kumar and Alappat 2003)*

Sl. No.	Pollutant	Significance	Pollutant weight
1	Total Chromium	4.057	0.064
2	Lead	4.019	0.063
3	COD	3.963	0.062
4	Mercury	3.923	0.062
5	BOD_5	3.902	0.061
6	Arsenic	3.885	0.061
7	Cyanide	3.694	0.058
8	Phenol Compound	3.627	0.057
9	Zinc	3.585	0.056
10	pH	3.509	0.055
11	Total Kjeldahl Nitrogen	3.367	0.053
12	Nickel	3.321	0.052
13	Total Coliform Bacteria	3.289	0.052
14	Ammonia Nitrogen	3.250	0.051
15	Total Dissolved Solids	3.196	0.050
16	Copper	3.170	0.050
17	Chlorides	3.078	0.049
18	Total Iron	2.830	0.045
	Total	63.165	1.000

2.3 Development of Pollutant Variable Rating Curves

In the third questionnaire, a selected group of panelists was requested to develop rating curves for all the *eighteen (18)* selected variables as shown in Figures 1 to 3. This was done by providing graph sheets to the panelists. On the graph sheets, levels of leachate pollution (sub index score) from *'0' to '100'* were indicated on the ordinate of each graph, while, various level of concentrations of particular variable, up to the maximum limits reported in literature, were marked along the abscissa. The panelists were requested to draw a curve on each graph, which represented leachate pollution produced by the various concentrations of each lechate pollutant.

The panelists were requested to start the curves for each pollutant variable with a minimum value of *'5'* of leachate pollution even if there is no contamination from the pollutant to the overall leachate pollution. This was done to ensure that multiplicative aggregation function can be used at the later stage, if required, and the minimum value of *'5'* units of leachate pollution will ensure that the LPI value does not result in zero even if some of the pollutants do not show any pollution. Therefore, the theoretical range of *LPI* is from *'0' to '100'*.

The responses received on graph sheets have been used to produce a set of 'average curves', one for each pollutant variable. As all the curves are obtained from the survey, they are implicit non-linear functions for which no mathematical equations can be given. In each figure, the bold line shows the arithmetic mean of all the panelists' curves, while, the x axis error bars indicate the *90%* confidence limit. Approximately *75%* of the panelists' curves fall within this area.

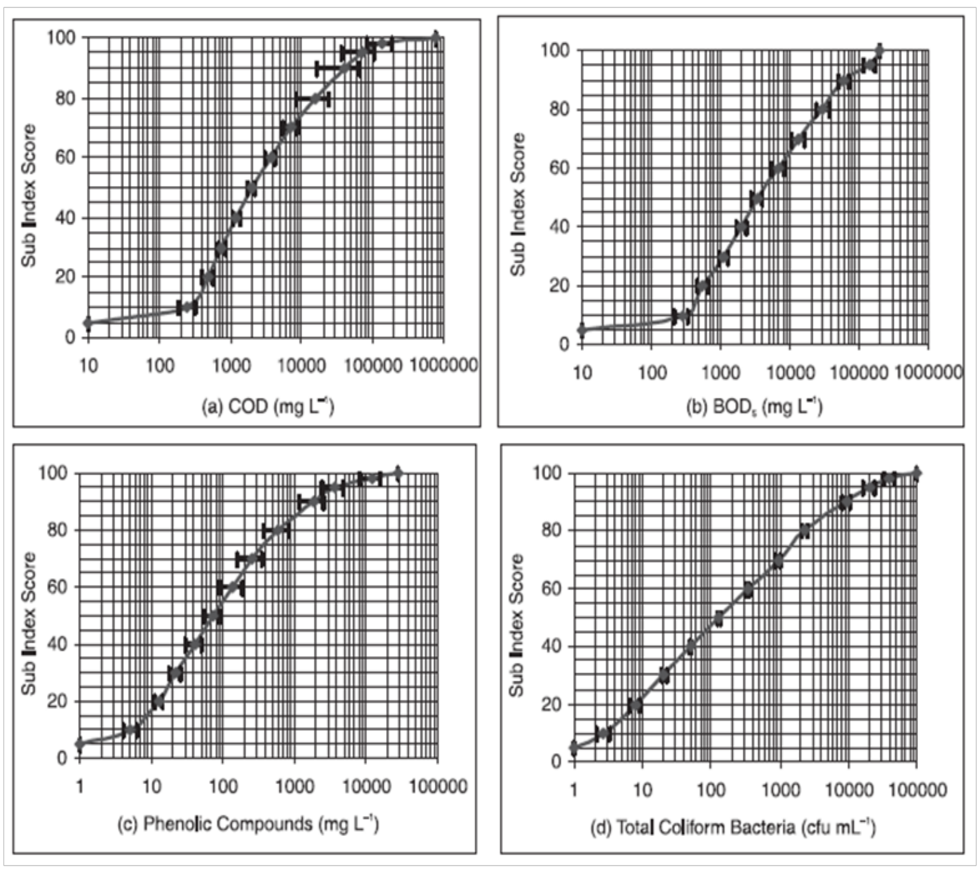

Figure 1: *LPI* organic (LPI_{or}) sub-index average curves with 90% confidence limit for (a) COD; (b) BOD_5; (c) phenolic compounds and (d) total coliform bacteria

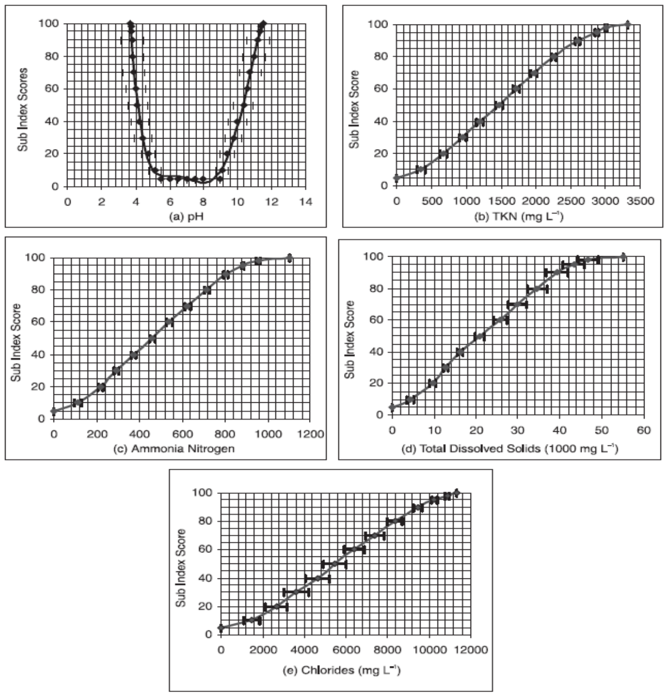

Figure 2: *LPI* organic (LPI_{in}) sub-index average curves with 90% confidence limit for (a) pH; (b) total kjeldhal nitrogen; (c) ammonia nitrogen; (d) total dissolved solid and (e) chlorides

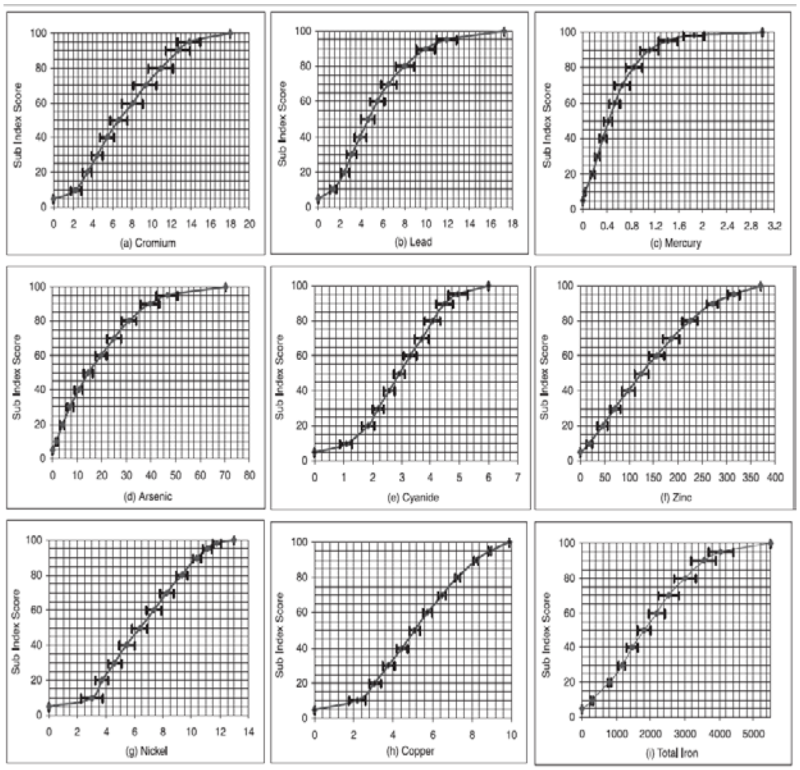

Figure 3: *LPI* organic (LPI_{hm}) sub-index average curves with 90% confidence limit for (a) chromium; (b) lead; (c) mercury; (d) arsenic; (e) cyanide;(f) zinc; (g) nickel (h) copper; and (i) total iron (all concentration in mgL^{-1}) *(after Kumar and Alappat, 2003).*

2.4 Variable Aggregation

The weighted sum linear aggregation function was used to sum up the behavior of all the leachate pollutant variables. The various possible aggregation functions were evaluated by Kumar and Alappat (2003) to select the best possible aggregation function. The analysis of the six short-listed aggregation function was performed to arrive at the best possible aggregation function. However, Panelists suggested that if the concentrations of the *eighteen (18)* selected variables are known, the following *Equation (1)* is used. Otherwise, *Equation (2)* is used.

$$LPI = \sum_{i=1}^{n} w_i p_i \quad (1)$$

LPI = the weighted additive leachate pollution index
w_i = the weight for the I^{th} pollutant variable
p_i = the sub-index value of the I^{th} leachate pollutant variable, number of leachate pollutant parameters
n =18 and $\sum w_i = 1$.

$$LPI = \frac{\sum_{i=1}^{n} w_i p_i}{\sum w_i} \quad (2)$$

Pollutant parameter for which data is available in this study, m < 18 and $\sum w_i < 1$

3.0 SUB-INDICES OF LEACHATE POLLUTION INDEX

To make LPI more informative and useful among the scientific community and field professionals, the LPI may be subdivided into three sub-indices. The three sub-indices are based on the leachate characteristics and will indicate the dominant pollutants present in a given landfill leachate. Leachate can be characterized in terms of its physical, chemical and biological composition. The physical properties of leachate include colour, odour, solids, temperature, etc., whereas the chemical properties can be categorized into organic and inorganic constituents. The biological constituents include various viruses and pathogenic organisms present in the leachate. Other pollutants of concern are biodegradable organics, heavy metals and other dissolved inorganic. The eighteen leachate pollutant variables selected for the LPI are grouped into three components so as to formulate three sub-LPIs.

3.1 LPI Organic (*LPIor*)

Organic compounds are normally composed of a combination of carbon, hydrogen and oxygen, with nitrogen in some cases. The principal groups of organic substances include proteins, carbohydrates, fats and oils. Urea is another organic compound that may join the leachate from the runoffs from surrounding agricultural areas. However, it decomposes so rapidly that the undecomposed urea is seldom found in leachate. Apart from that, leachate may contain a large number of different synthetic organic molecules ranging from simple to extremely complex in structure, such as surfactants, VOCs and agricultural pesticides. The presence of these substances, has complicated leachate treatment because many of them either cannot be or are very slowly decomposed biologically. Laboratory methods commonly used to measure the gross amount of organic matter include: biochemical oxygen demand (BOD), chemical oxygen demand (COD) and total organic carbon (TOC). In this group, the pollutants selected for sub-LPI are BOD_5, COD and phenol compounds. Since only one Biological pollutant, total coliform bacteria, is included in LPI, this is also included in this subgroup. The weight factors for the pollutants in *LPI*or have been recalculated on a scale of 1 as if *LPI*or is an absolute index and are presented in Table 2 and also in Figure 1.

The biodegradability of the leachate varies with time and thus the *LPI*or subgroup provides important information regarding the biodegradability of the leachate at a given time. Changes in the biodegradability of the leachate can be monitored by checking the BOD5/COD ratio. For a young landfill, the BOD5/COD ratio may be in the range of 0.5 to 0.6 or higher. Ratios in this range and above may be taken as an indication that the organic matter in the leachate is readily biodegradable, whereas the BOD5/COD ratio in old and matured landfills may be in the range of 0.02 to 0.2 suggesting that the organic matter in the leachate is not readily biodegradable. The high BOD5 and COD values may also be of importance when the leachate is to be transported and treated at the municipal wastewater treatment plants to ascertain whether or not the wastewater treatment plant can take the extra organic load. The sub-index rating curves along with 90% confidence limits (shown as *x*-error bars) for the pollutants included in this subgroup are shown in Figure 1to 3.

Table 2: Weight factors of leachate parameters based on *sub-LPI*

Index	Parameters	Weight factor
LPI organic (LPI_{or})	COD	0.267
	BOD	0.263
	Phenol compounds	0.246
	Total coliform bacteria	0.224
	Summation	1.000
LPI inorganic (LPI_{in})	pH	0.214
	TKN	00000.2060
	Ammonia nitrogen	0.198
	Total dissolved solids	0.195
	Chlorides	0.187
	Summation	1.000
LPI heavy metal (LPI_{hm})	Total chromium	0.125
	Lead	0.123
	Mercury	0.121
	Arsenic	0.119
	Cyanide	0.114
	Zinc	0.11
	Nickel	0.102
	Copper	0.098
	Iron	0.088
	Summation	1.000

3.2 LPI inorganic (LPI_{in})

Chlorides, alkalinity, various forms of nitrogen, phosphorous, sulphur, pH, heavy metals, gases like hydrogen sulphide and methane, etc. constitute the inorganic component of leachate. Due to their toxicity, certain cations (including heavy metals) are of great importance in the treatment and disposal of leachate and hence they have been categorized into a separate group of sub-indices. The LPI inorganic component consists of chlorides, pH, ammonia nitrogen, and total Kjeldhal nitrogen. One of the physical constituents of the LPI, total dissolved solids (TDS), is also included in this group. The weight factors for the pollutants in *LPI*in have been recalculated on a scale of 1 as if *LPI*in is an absolute index and are reported in Table 2.

The sub-index rating curves along with 90% confidence limits (shown as *x*-error bars) for the pollutants included in this subgroup are shown in Figure 2. Chlorides present in the leachate are usually not attenuated by soil and are highly mobile under all conditions and thus have special significance as the tracer element of leachate plume joining the ground water aquifers. This sub-LPI therefore has its own significance and importance in deciding leachate treatment systems. This sub-LPI can also be helpful in siting new naturally attenuation (NA) landfill (the landfills which do not have any liners and rely on natural attenuation of the leachate) that may have similar leachate characteristics in the same region. Studies however indicate that even small NA type landfills (waste volume up to 50 000 yd3) may impact groundwater (Friedman 1988; Bagchi1994). Further, the high TDS present in leachate may pose difficulties in the biological treatment of leachate.

3.2 LPI heavy metal (LPI_{hm})

Many metals such as chromium, lead, zinc, nickel, copper, iron and mercury are important constituents of leachates produced from landfills. Many of these metals are also classified as priority pollutants. The US Environmental Protection Agency has identified approximately 129 priority pollutants in 65 classes to be regulated by categorical discharge standards.Priority pollutants (both inorganic and organic) were selected on the basis of their known or suspected carcinogenicity, mutagenicity, teratogenicity, or high acute toxicity. Two non-metal priority pollutants, arsenic and cyanide have also been included in this sub-group. Many of the organic priority pollutants are also classified as volatile organic compounds. The weight factors for the pollutants included in this sub-index have been recalculated on a scale of 1 as if *LPI*hm is an absolute index and are reported in Table2. The sub-index rating curves along with 90% confidence limits (shown as *x*-error bars) for the pollutants included in this subgroup are shown in Figure 3.

The presence of heavy metals in the leachate is of great concern for the unlined landfills in which the ground water table is not deep with respect to the landfill base. The heavy metal s are removed by the ion exchange reactions as the leachate moves through the soil, and therefore the depth of the unsaturated soil strata below the landfill base decides the capacity of the soil to retain the heavy metals (Bagchi 1994).The high index scores of this group indicate that the leachate treatment technology adopted shall be prudent for taking care of the removal of heavy metals from the leachate stream.

The three sub-LPI scores can be calculated separately as individual indices using equations (1) and (2) depending on the availability of information about the leachate characteristics. The values *n* and *m* in equations (1) and (2) now correspond to the number of pollutants included in the sub-LPI and number of pollutants for which the data is available.

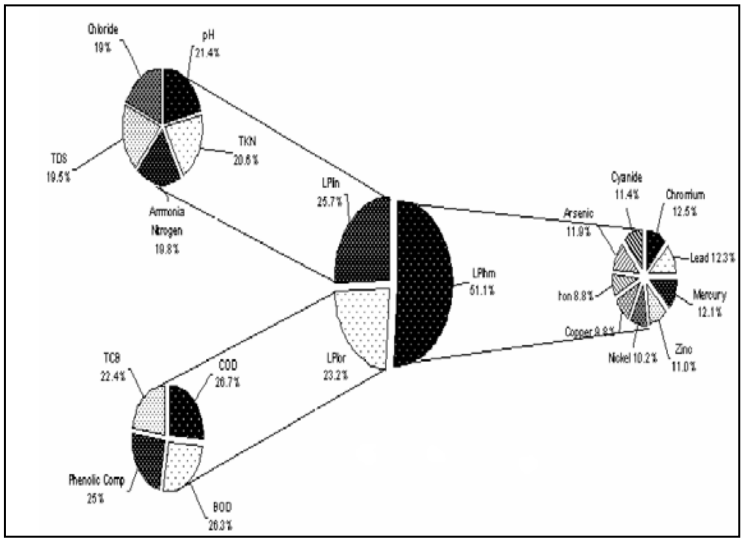

Fig. 4: Three LPI subgroups and the pollutants in the three subgroups with percentage weight factors.

4.0 CALCULATION OF SUB-*LPI* AND OVERALL *LPI*: CASE STUDY

For calculating of s*ub-LPI* and overall *LPI*, the following steps were followed:

1. The concentration of eighteen (18) leachate parameters required for *Sub-LPI* and overall *LPI* were measured and monitored in the laboratory (*Table 3 and 4, column 4*).
2. Evaluating of sub-index scores or individual pollutant rating (*pi*) of all the pollutants included in *Sub-LPI* and overall *LPI* are noted from Figures 1, 2 and 3 with respect to the concentration of pollutant parameter in leachate (*Table 3 and 4, column 5*).
3. Overall pollutant rating as sub-pollution indices for organic ($\boldsymbol{LPI}_{\mathrm{or}}$), inorganic ($\boldsymbol{LPI}_{\mathrm{in}}$) and heavy metals ($\boldsymbol{LPI}_{\mathrm{hm}}$) are calculated using the weight factors given in Table 2 based on the aggregation function given in Equation (1) (*Table 3 and 4, column 6*).
4. Finally, the aggregation of the three sub-*LPIs* gets the overall *LPI*. The three sub-*LPI* values are aggregated to calculate the overall *LPI* using the following Equation (3).

$$\boldsymbol{LPI} = 0.232\boldsymbol{LPI}_{or} + 0.257\boldsymbol{LPI}_{in} + 0.511\boldsymbol{LPI}_{hm} \quad (3)$$

Where *LPI* is the overall ***LPI***, $\boldsymbol{LPI}_{\mathrm{or}}$ is the sub-leachate pollution index of organic component; $\boldsymbol{LPI}_{\mathrm{in}}$ for inorganic component and $\boldsymbol{LPI}_{\mathrm{hm}}$ for of heavy metal component in leachate.

Here, it can be noted that Equation (3) has been derived based on the weight factors of the eighteen pollutants included in the overall LPI and their contribution to each sub-*LPI* (Figure 4). Figure 4 depicts the component of organic, inorganic and heavy metal fraction is 23.2, 25.7 and 51.1% for evaluating the overall *LPI* these fractions is used.

Table 2 Characteristics of leachate and *LPI* of Dhaka and Chittagong solid waste landfill

Index (1)	Parameters (2)	Weight factor, w_i (3)	Pollutant concentration, c_i (4)		Individual pollutant rating, p_i (5)		Overall pollutant rating, $w_i p_i$ (6)	
			Dhaka	Chittagong	Dhaka	Chittagong	Dhaka	Chittagong
LPI_{or}	COD	0.267	9100	6100	75	63	20.03	16.82
	BOD_5	0.263	4100	3050	56	50	14.73	13.15
	Phenol	0.246	5	4.8	10	12	2.46	2.95
	TCB	0.224	13500	10000	92	86	20.61	19.26
	summation	**1.000**					57.82	52.19
LPI							**57.82**	**52.19**
LPI_{in}	pH	0.214	8.3	8.5	5	5	1.07	1.07
	TKN	0.206	4200	4000	100	100	20.60	20.60
	NH_4-N	0.198	900	550	70	58	13.86	11.48
	TDS	0.195	2000	1723	60	40	11.70	7.80
	Chlorides	0.187	2600	2100	15	12	2.81	2.24
	summation	**1.000**					50.04	43.20
LPI							**50.04**	**43.20**
LPI_{hm}	Chromium	0.125	21	18.5	100	95	12.50	11.88
	Lead	0.123	0.45	0.45	7	6	0.86	0.74
	Mercury	0.121	0.4	0.4	45	45	5.45	5.45
	Arsenic	0.119	0.01	0.01	5	5	0.60	0.60
	Cyanide	0.114	1.4	1.3	12	12	1.37	1.37
	Zinc	0.11	18	17.8	6	7	0.66	0.77
	Nickel	0.102	0.048	0.078	5	5	0.51	0.51
	Copper	0.098	5.8	5.6	48	50	4.70	4.90
	Total Iron	0.088	5.5	4.7	5	5	0.44	0.44
	summation	**1.000**			801	744	27.08	26.64
LPI							**27.08**	**26.64**
	Derived *LPI*						***40.11***	***36.82***

All values in mg/L except pH and total coliform unit (cfu/100ml); COD=chemical oxygen demand, BOD_5=biological oxygen demand, TKN=total kjendhal nitrogen, TCB= total coliform bacteria and TDS=total dissolve solid, NH_4-N=Ammonia nitrogen.

Table 3 Characteristics of leachate and *LPI* of Rajshahi and Khulna solid waste landfill

Index (1)	Parameters (2)	Weight Factor, w_i (3)	Pollutant concentration, c_i (4)		Individual pollutant rating, p_i (5)		Overall pollutant rating, $w_i p_i$ (6)	
			Khulna	Rajshahi	Khulna	Rajshahi	Khulna	Rajshahi
LPI$_{or}$	COD	0.267	6868	7120	68	70	18.16	18.69
	BOD_5	0.263	2890	2518	54	47	14.20	12.36
	Phenol	0.246	3.5	3.2	10	8	2.46	1.97
	TCB	0.224	9000	6000	87	86	19.49	19.26
	summation	**1.000**					54.31	52.28
LPI							**54.31**	**52.28**
LPI$_{in}$	pH	0.214	8.1	8.3	4	6	0.86	1.28
	TKN	0.206	3200	3000	100	98	20.60	20.19
	NH_4-N	0.198	450	400	47	43	9.31	8.51
	TDS	0.195	1150	888	12	7	2.34	1.37
	Chlorides	0.187	1700	1500	10	8	1.87	1.50
	summation	**1.000**					34.97	32.85
LPI							**34.97**	**32.85**
LPI$_{hm}$	Chromium	0.125	14	12.4	91	90	11.38	11.25
	Lead	0.123	0.45	0.45	5	5	0.62	0.62
	Mercury	0.121	0.3	0.3	40	40	4.84	4.84
	Arsenic	0.119	0.01	0.01	5	5	0.60	0.60
	Cyanide	0.114	1	1	10	10	1.14	1.14
	Zinc	0.11	12	11.2	5.1	4.9	0.56	0.54
	Nickel	0.102	0.04	0.032	4	4	0.41	0.41
	Copper	0.098	4	4.1	37	35	3.63	3.43
	Total Iron	0.088	4.2	4.6	5	5	0.44	0.44
	summation	**1.000**					23.60	23
LPI							**23.60**	**23.26**
	Derived *LPI*						***33.65***	***32.46***

All values in mg/L except pH and total coliform unit (cfu/100ml); COD=chemical oxygen demand, BOD_5=biological oxygen demand, TKN=total kjendhal nitrogen, TCB= total coliform bacteria and TDS=total dissolve solid, NH_4-N=Ammonia nitrogen.

5. RESULTS AND DISCUSSIONS

In this study, to evaluate the contaminate potential of selected solid waste landfill in Bangladesh, the significance and variation of leachate parameters in *LPI*, individual and overall *LPI* as well as the variability of leachate parameters involved in *sub-LPI* and overall *LPI* has been analyzed and hence discussed in followings.

5.1 Significance of Leachate Parameters Involved in *Sub-LPI* and Overall *LPI*

***Organic Parameters**:* Foo (2009) showed that young landfill leachate is characterized by high BOD_5 (4000~13000 mg/L) and COD (30000~60000 mg/L). It has been demonstrated (Tatsi et al. 2003) that young leachate may have BOD_5 as high as 81000 mg/L. Moreover, Li (2002) stated that increase in age of landfill is decrease the BOD_5 and COD. However, in this study, BOD_5 was 900-5000, 1000-6000, 1800-4000, 2000-4000, and range of COD was 2000-10000, 2200-8000, 1500-9000, 1500-9500 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

Phenol Compounds: Christensen et al. (2001) report the concentration of phenol compounds in a landfill between 1–2100 μg/L. However, in this study, the phenol was ranging 3-8, 3-7, 2.5-5 and 2.5-6 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

Inorganic Parameters: A study conducted by Christensen et al. (2001) observed that leachate is generally found to have pH between 4.5 and 9, moreover, pH of young leachate is less than 6.5, while, old landfill leachate has pH higher than 7.5 (Abbas et al. 2009). However, in this study, pH was ranging 6-9, 6-8.5, 6-8.5 and 6-8.5 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

TDS compromise mainly of inorganic salts and dissolved organics. According to Koshy (2002) TDS is one of the parameters taken into consideration for licensing discharge of landfill leachate in many countries. In this study, TDS was ranging 800-5600, 800-3000, 600-2500 and 500-2500 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

According to Kulikowska et al. (2008) ammonium represents the major proportion of total nitrogen. In comparison to soluble organics, the release of soluble nitrogen from waste into leachate continues over longer period. However, in this study, Nitrogen was ranging 300-1200, 300-800,200-600,280-750 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

According to Deng and Englehardt (2007) the chlorides may range between 200-3000 mg/L. for a 1-2 year old landfill the concentration decreases for a landfill greater than 5-10 years old. However, in this study, chlorides was ranging 1000-3500, 1500-3000, 1000-2500, and 100-3000 for the solid waste landfill of Dhaka, Chittagong, Rajshahi and Khulna, respectively.

Heavy metal: According to Jones et al. (2006) the solubility and mobility of metals may increase in presence of natural and synthetic complexing ligands such as EDTA and humic substances. According to Baun and Christensen et al. (2004) less than 30%, typically less than 10% of the total metal concentration is present in free metal ion forms and the rest is present in colloidal or organic complexes. Jensen and Cristensen (1999) found that 10-60% of Ni, 30-100% Cu and 0-95% Zn were constituted in colloidal function. The solubility in metals can also increase because of the reducing condition of leachate. In this study, Cr 15-20, 17-24, 12-17 and 12-16, Lead 0.4-0.5 for all four landfills, Mercury 0.3-0.5, 0.3-0.45, 0.25-0.35 and 0.25-0.32, Arsenic 0.01-0.03 for all four landfills, Cynide 1-1.5, 1-1.45, 0.5-1.2, and 0.5-1.2, Zinc 13-20, 14-20, 13-17, and 13-16, Nickel 0.05-1, 03-0.075, 0.03-0.06, and 0.03-.05, Copper 3-6.5, 2.5-6, 2-4.5, and 2.5-4.5, total Iron 4.5-6.5, 3.5-6, 3.5-5 and 3.5-5.5 for solid waste landfill of Chittagong, Dhaka, Khulna and Rajshahi, respectively.

5.2 Sub-*LPI* and Overall *LPI*

Table 2 and 3 illustrates the calculation of sub-LPI and overall *LPI* for a particular period of leachate sampling for Dhaka and Chittagong as well as Khulna and Rajshahi landfill, respectively. Tables 1 and 2 depicts the component of organic fraction in leachate for all the selected landfill was found highest against the other counter fraction i.e. inorganic and heavy metal fraction. Consequently, the Dhaka, Chittagong, Khulna and Rajshahi landfill had the highest *sub-LPI* of organic content than that of *sub-LPI* of inorganic and heavy meat is provided in Figure 5. Figure 5 reveals that Dhaka landfill had not only the highest LPI_{or} but also the overall *LPI* than that of other studied landfill. Moreover, result reveals that LPI_{or} was ranging from 56.22 to 36.86 (Dhaka); 42.43 to 30.41 (Rajshahi); 50.98 to 35.12 (Khulna) and 55.30 to 37.54 (Chittagong), while LPI_{in} was ranging from 50.57 to 34.5 (Dhaka); 40.27 to 29.8 (Rajshahi); 50.21 to 37.19 (Khulna) and 50.04 to 34.54 (Chittagong); as well as LPI_{hm} 40.4 to 27.5 (Dhaka) 35.58 to 25.78 (Rajshahi); 34.22 to 24.9 (Khulna) and 38.6 to 27.5 (Chittagong). Moreover, it depicts that overall LPI was ranging 50.01 to 30.68 (Dhaka); 38.48 to 25.20 (Rajshahi); 35.18 to 13.45

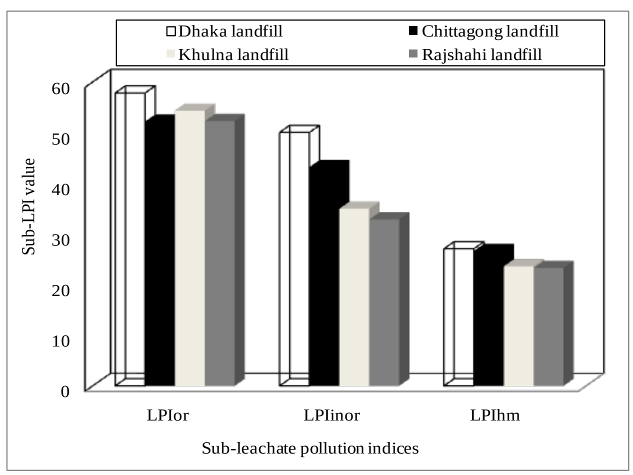

Figure 5 Variation of sub-leachate pollution indices of landfill

(Khulna) and 40.43 to 25.41 (Chittagong) solid waste landfill. Based on Tables 1 and 2, it can be seen that the derived *LPI* for the Dhaka landfill was the highest, while, for Rajshahi landfill the lowest is shown in Figure 6. The higher *LPI* (40.14) for the Dhaka landfill further indicates that the MSW has not yet stabilized. This is also evident from the high BOD_5 and COD values.

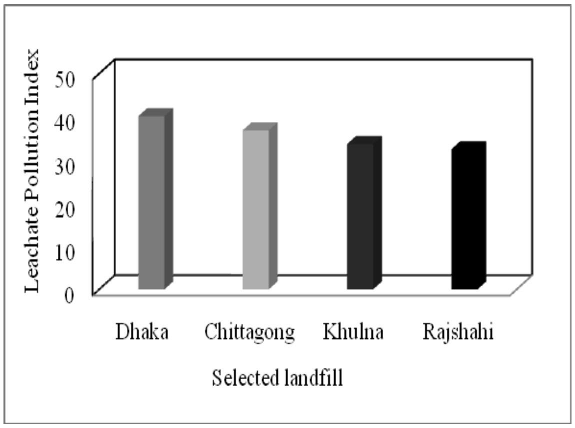

Figure 6 Variation of *LPI* of landfill.

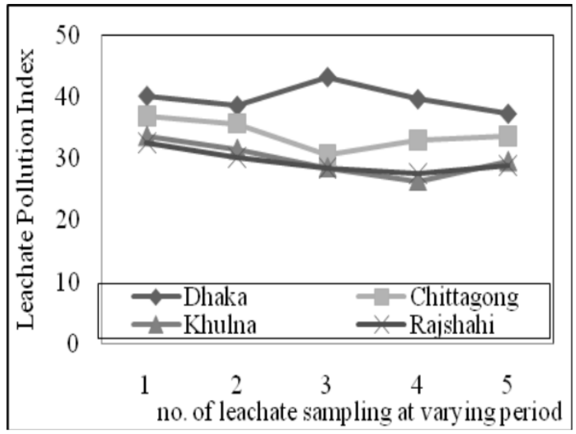

Figure 7 Variation of overall LPI of landfill against sampling period.

Here, it is important to note that standards for the disposal of treated leachate to inland surface water as per the Management and Handling Rules (The Gazette of India 2000) should not exceed 2.0, 0.1, 250, 0.01, 30, 0.20, 0.20, 1.0, 5.0, 5.5-9.0, 100, 3.0, 50.0, 2100, 3.0 and1000 for Cr, Pb, COD, Hg, BOD_5, As, Cn, Phenol, Zn, pH, TKN, Ni, NH3-N, TDS, Cu and Cl^- concentration and their corresponding overall *LPI* of 7.38. The comparison of the leachate characteristics with the standards set for the disposal of treated leachate verifies the fact that the leachate generated from Dhaka landfill is highly contaminated and the values of *LPI* for all the selected landfill exceed the *LPI* of treated leachate of 7.38. The high *LPI* demands that leachate generated from the selected landfill should be treated. The low value of *LPI* (28.86) for the leachate of Rajshahi landfill indicates the relatively lower contaminant potential. However, the individual contaminants shall meet the state discharge standards before discharge of leachate into any surface water body.

Moreover, the relatively highest *LPI* of 29.54, 33.64 and 37.25, of the three landfill of Khulna, Chittagong and Dhaka, respectively, are comparable and indicates that the leachate should be treated before discharging. Moreover, it can be depicted that the comparatively lower *LPI* for the Rajshahi landfill sites are attributable to low concentrations of heavy metals in the Leachate (Table 3). Landfill age also plays an important role in leachate characteristics and hence, influences the *LPI* value (Kurniawan 2006).

The variation of overall *LPI* of the selected landfill against leachate sampling period is shown in Figure 7. Based on Figure 7, it can be seen that in case of Dhaka landfill, the *LPI* was found higher than that of other studied landfill. The results indicate that the Dhaka landfill has high *LPI* value in comparisons with the three landfills of Rajshahi, Khulna and Chittagong and therefore, it has relatively more contamination potential. Based on the evaluated results, the *LPI* was found 40.14, 38.54, 43.21, 39.62, 37.25 for Dhaka Matuail landfill, 38.85, 35.63, 30.52, 32.87, 33.64 for Chittagong landfill, 33.67, 31.53, 28.54, 26.35, 29.54 for Khulna landfill and 32.48, 30.15, 28.45, 27.5, 28.86 for Rajshahi landfill against the leachate sampling period.

5.3. Interpretation of Parameters Involved In LPI in Solid Waste Landfill

The observed result reveals that COD, pH and Cu is high in Rajshahi landfill than that of Khulna landfill, while BOD_5 and TKN, chromium is significantly higher in Khulna landfill than Rajshahi landfill (Table 3). Moreover, The COD, BOD_5, Phenol, TCB, TDS and chloride were significantly higher for Dhaka landfill, in comparison to Rajshahi and Khulna landfill. A significant difference between individual and cumulative pollution ratings for both the landfill of Dhaka and Rajshahi, was observed due to the distinct difference in their concentrations. The concentration of heavy metals (Cu and Ni) was fairly similar for both the Rajshahi and Khulna landfill. Although these two landfills exhibited notable differences for chlorides and TCB, but the influence of individual and cumulative pollution ratings is insignificant. The Cr and Zn in Chittagong landfill were higher than that of Rajshahi landfill which resulted in significantly higher cumulative pollution rating.

As the individual pollution ratings of BOD_5, TKN, TCB, TDS and chloride were lower for Rajshahi landfill, the cumulative pollution rating of Rajshahi landfill was consequently lower. The concentrations of COD, TKN, TCB, TDS, chloride, pH, Cr, Pb, Zn and Fe represent a significant difference between Chittagong landfill and Khulna landfill. The higher COD, Zn, TKN, TDS, Pb, Ni and Fe of Dhaka landfill imply higher individual and cumulative pollution rating and consequently higher *LPI* than Chittagong landfill. Moreover, the TDS in Dhaka landfill was more than 1.25 times higher than Rajshahi landfill, and the cumulative pollution rating for TDS in Dhaka landfill was 3.0 times higher than Rajshahi landfill. The pH, Pb, Ni, Cu and Fe in Khulna landfill were higher than the Rajshahi landfill, which resulted in significantly higher cumulative pollution rating. All the concentrations except pH and Cu were lower for Rajshahi landfill than that of Dhaka, Chittagong and Khulna landfill has lowest individual and cumulative pollution rating and consequently lower *LPI*.

6. CONCLUSIONS

The component of organic fraction in leachate for all the selected landfill had highest against the other counter fraction i.e. inorganic and heavy metal fraction. Consequently, Dhaka, Chittagong, Khulna and Rajshahi landfill shows the highest *sub-LPI* of organic content than that of *sub-LPI* of inorganic and heavy metal. Moreover, Dhaka landfill had not only the highest *LPI* of organic fraction but also the overall *LPI* than that of other studied landfill. In contrast, result reveals that comparison of the leachate characteristics with the standards set for the disposal of treated leachate verifies the fact that the leachate generated from Dhaka landfill is highly contaminated and the values of *LPI* for all the selected landfill exceed the *LPI* of treated leachate of 7.38. The LPI provides a meaningful method of evaluating the leachate contamination potential of solid waste landfill. Finally, it can be concluded that highest *LPI* demands that leachate generated from the selected landfill should be treated before discharging in to the water bodies.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

REUSES OF SOLID WASTE IN RAJSHAHI CITY, BANGLADESH

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ABSTRACT

The Rajshahi City, the north western divisional city in Bangladesh is located between 24° 21‘ and 24° 25‘ north latitudes and between 88° 32‘ and 88° 40‘ east longitudes with an estimated population of 773,430 which is a lively, friendly and open city with many attractions. The Rajshah city generates approximately 350 tonnes of solid waste every day while the amount increases to 400 tones during summer. Of the total, only 210 tones are collected and dumped into the open dumping ground at Nawdapara. The remaining 140 tones of waste are dumped straight into drains, water bodies and open spaces or littered around. The RCC is also not responsible for collecting waste from households and in many mahallas the communities. Generation of waste is increasing day by day but waste management system is not effective. As a result, scrap metal, cement bags, paper, woods, cartons, cloths, foods, glass bottles, plastic goods are becoming a major stream in solid waste, which are being littered on landscape of Rajshahi city. In this paper, a survey on existing reuse of solid waste is conducted in the Rajshahi City, Bangladesh. An environmentally sound reuse of solid waste improves the economic condition of concerned people and minimizes environmental hazards in Rajshahi City, Bangladesh.

Keywords: *Solid waste, Collection, Resources recovery, reuse, Rajshahi City*

1. INTRODUCTION

Solid waste is the unwanted or useless solid or semi solid material generated from combined residential, industrial and commercial activities in a given area. It may be categorized according to its origin (domestic, industrial, commercial, construction or institutional); according to its contents (organic material, glass, metal, plastic paper etc); or according to hazard potential (toxic, non-toxin, flammable, radioactive, infectious etc). A huge quantity of solid waste generate in all the developed and developing countries per day. Solid-waste management may be defined as the discipline associated with controlling the generation, storage, collection, transfer and transport, processing, and disposal of solid waste in a manner that is in accordance with the best principles of health, economics, engineering, conservation, aesthetics, and other environmental considerations. Management of solid waste reduces or eliminates adverse impacts on the environment and human health and supports economic development and improved quality of life. A number of processes are involved in effectively managing waste for a municipality. Of them reuse of solid waste is not only environment friendly but only it increase economic condition of the concern area. Waste reuse of products is the method of waste prevention. It eliminates the production of waste at the source of usual generation and reduces the demands for large scale treatment and disposal facilities. Methods of waste reduction include manufacturing products with less packaging, encouraging customers to bring their own reusable bags for packaging, encouraging the public to choose reusable products such as cloth napkins and reusable plastic and glass containers, backyard composting and sharing and donating any unwanted items rather than discarding them.

All of the methods of waste prevention mentioned require public participation. In order to get the public onboard, training and educational programmes need to be undertaken to educate the public about their role in the process. Also the government may need to regulate the types and amount of packaging used by manufacturers and make the reuse of shopping bags mandatory. There are many environmental benefits that can be derived from the use of these methods. They reduce or prevent green house gas emissions, reduce the release of pollutants, conserve resources, save energy and reduce the demand for waste treatment technology and landfill space. Therefore it is advisable that these methods be adopted and incorporated as part of the waste management plan. So, reuse of solid-waste has become an important issue in Rajshahi City, Bangladesh and it needs to be resolved through an integrated community, private-sector, and policy-based approach. The objective of the study is determination the type, amount of reusable solid waste materials which improves the economic condition of the concerned people and minimizes environmental hazards in Rajshahi City, Bangladesh.

2. METHODOLOGY

2.1 Study area

The study area is Rajshahi City in Bangladesh, which is the divisional headquarters of the Rajshahi division as well as the administrative district which shown in figure 1. It lies between located between 24° 21‘ and 24° 25‘ north latitudes and between 88° 32‘ and 88° 40‘ east longitudes. Rajshahi is located in the north-west of the country and the city has a population of 773430 (male 52.4%, female 47.6%) (RCC website). It consists of 30 wards with a total area of 93.47 km2 and is situated on the northern banks of the river Padma (the downstream of the Ganges River in India). The climate of the city is generally marked by monsoons, high temperature, considerable humidity and moderate rainfall. It has a mean annual rainfall of below 1524 mm compared to a national average of 2540 mm (BBS, 2005).

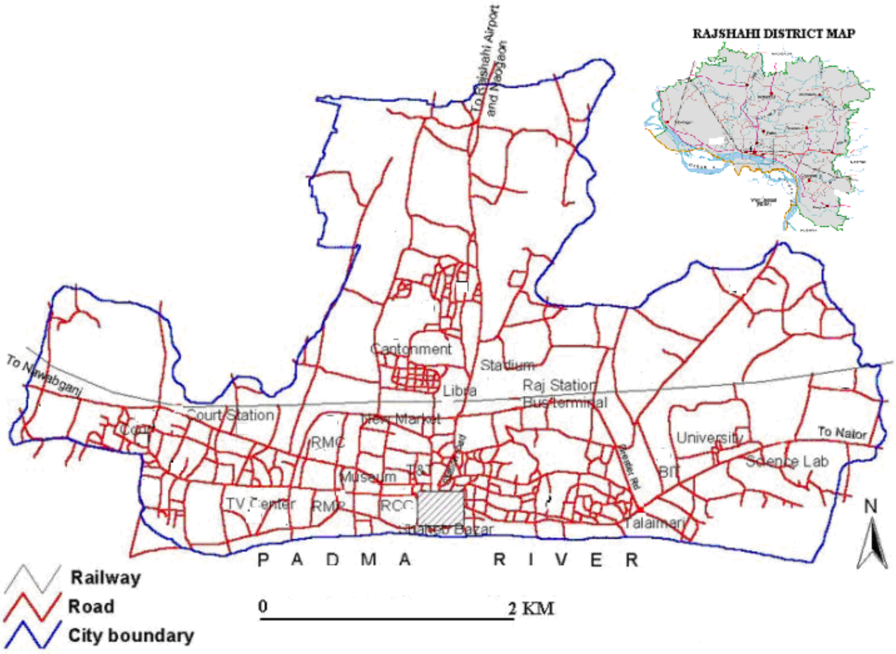


Figure 1: Map of the study area

2.2 Data collection

Both primary and secondary data have been collected to conduct this study. Primary data collected through field survey. The authors are from Rajshahi and their personal observation, living, and discussion with the professionals are other sources of primary data. Secondary data have been collected from different sources like journals, books, published documents and the Internet.

3. DISCUSSION

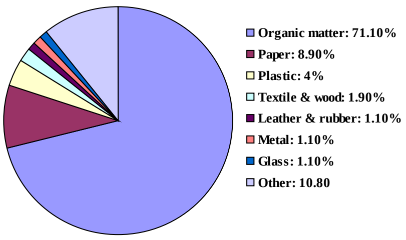
3.1 Waste recycling and disposal

The Rajshahi City Corporation (RCC) is the key government institution in waste managements in the city of Rajshahi. The present solid waste management practice being followed in the study area is based on the end-of-pipe approach, i.e. collect-transport-dispose. The sustainable way for solid waste management is based on 4 R's principle i.e. reduce, reuse, recycle and recovery of the waste. The Rajshah city generates approximately 350 tonnes of solid waste every day while the amount increases to 400 tones during summer. Of the total, only 210 tones are collected and dumped into the open dumping ground at Nawdapara. The remaining 140 tones of waste are dumped straight into drains, water bodies and open spaces or littered around. The composition was then categorized into eight major categories: organic matter, paper, plastic, textile and wood, leather and rubber, metal, glass and other (After AIT, 1991) as shown in Table 1. In the study area solid waste are recycled by mainly waste pickers which are directly involved waste collection, sorting and recycling are exposed to health hazards e.g respiratory problems and skin diseases are common. Scavengers suffer from serious occupational health risks. Because of manual handling and lack of protective clothing and equipment, they are undoubtedly exposed to various health risks (Wilson *et. at.*, 2006). Waste pickers are normally injured during collection of saleable materials from the dustbin. In the case of waste from hospitals and clinics, it was found that among total professional injuries 42% was needle injury caused during pushing injection or blood drawing from patients at the laboratory, 24% was sharp injury during cleaning and rest 34% was injury during ampule breaking (Nasima and Miah, 2004).

Table 1: Waste composition category (after AIT, 1991)

Waste category	Waste compostion
Organic matter	Waste from foodstuff such as food and vegetable refuse, fruit skin, stem of green, corncob, leaves, grass and manure.
Paper	Paper, paper bags, cardboard, corrugated board, box board, newsprint, magazines, tissue, office paper and mixed paper.
Plastic	Any material and products made of plastics such as wrapping film, plastic bag, polythene, plastic bottle, plastic hose and plastic string.
Textile & wood	Has its origin from yarn, wood and bamboo such as cotton, wool, nylon, cloth, desk, chair, bed board, toy and coconut shell.
Leather & rubber	Any material and products made of rubber or leather such as ball, shoes, purse, rubber band and sponge.
Metal	Ferrous and non-ferrous metal such as tin can, wire, fence, knife, bottle cover, aluminium can and other aluminium, foil, ware and bi-metal.
Glass	Any material and products made of glass such as bottles, glassware & light bulb.
Other	Yard waste, tyres, batteries, large appliances, and sanitary products, medical.

The waste composition for the entire waste stream of Rajshahi City in the year of 2010 is shown in figure 2. The percentage composition of waste combined from all locations was about 71.10% organic matter, 8.90% paper, 4.0% plastic, 1.9% textile and wood, 1.10 % leather and rubber, 1.10% metal, 1.10 % glass and 10.80 % other waste. The biodegradable fraction (organic matter) is normal very high as compared to other fractions, essentially due to the use of fresh vegetables and foods. . Figure 3 represents the contribution of different sources in total generation of MSW, where nearly 77.20 % of generated waste came from the residential sector, 18.60 % came from the commercial sector, 1.20 % from the institutional sector, street sweeping 1.20% and rest 1.80% from other sectors.



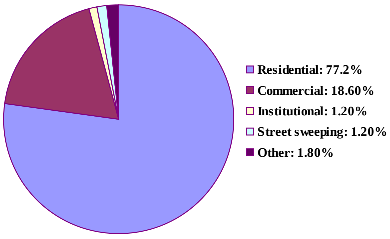


Figure 2: % composition of Solid waste in RCC   Figure 3: Contribution of various sources in total generation

3.2 Waste as a resource

The key is to think differently about the products we make, design, buy, and use. So that manufacturers, retailers, and consumers think about and treat by-products of their activities and products at the end of their useful lives as *resources* rather than waste. Businesses can gain a competitive edge by recognizing that creating waste is both economically and environmentally efficient. Treating waste as a resource based on the three principles of sustainability and resource conservation. a) **Treating waste as a resource reduces pollution:** shifting waste management practices toward those that better manage waste as a resource, such as waste reduction, reuse, recycling, or composting, leads to reductions in the amount of pollution released to the environment, including greenhouse gases. b). **Reducing waste saves money and creates jobs:** reducing and eliminating the generation of waste often creates significant cost savings by conserving raw materials and using resources more efficiently in the production of products. c). **Materials in waste often have value:** if certain materials are separated prior to disposal, these materials can be reused, recycled, or recovered for their highest and best use. For example, the organic portion of garbage can be kept separate in one's kitchen before it gets to the curb, and then processed into compost, which can be used as a high-quality soil amendment. As it seen in the various city of developing countries, this can bring money and jobs back to the local community.

Recycling and reclamation are now strongly promoted for conservation of resources and prevention of environmental degradation. Bangladeshi cities often collect only 40% to 50% of waste generated, with open dumping the only disposal method available (Wilson et.al., 2006). In Bangladesh, wastes having some market value are being reclamation or salvaged in three stages. In the first stage, housewives separate refuse of higher market value such as papers, bottles, fresh containers, old cloths, shoes etc. and sale them to street hawkers. Mostly children of slum dwellers known as “Tokai” carry out the second stage salvaging. They collect different items of low market value from waste collection bins. The items include broken glass, cans, cardboard, waste papers, rags, plastics, metals and miscellaneous commercial wastes discarded by households. Scavengers at final disposal sites do the third stage of salvaging when municipal trucks unload fresh refuse. The reclaimed materials reach the waste and materials shop through street hawkers who purchase old materials directly from households and through solid waste collectors who reclaim the materials from bins and final disposal sites. These reclaimed materials require intermediate processing like washing, drying and sorting. The refuse dealers separate the materials in proper form and sell them to consumers as well as supply them to appropriate processing for reuse as raw materials. The most common consumer products recycled in RCC area include aluminum beverage cans, steel food and aerosol cans, HDPE and PET bottles, glass bottles and jars, paperboard cartons, newspapers, magazines, and corrugated fiber board boxes, rubber sponge. PVC, LDPE, PP and PS are also recyclable, although these are not commonly collected. These items are usually composed of a single type of material, making them relatively easy to recycle into new products. The recycling of complex products (such as computers and electronic equipment) is not collected for recycling due to more difficult for reusing. In RCC area huge quantity of phencidil glass bottle are found in the solid waste. These glass bottles are collected by scavenger and it gathered in Nomovadra (in front of Bangladesh Sericulture Board) where sorting for sell. All the markets are situated in the in Nomovadra (in front of Bangladesh Sericulture Board) where reusable wasre are gather for further process. Carton papers and steel food cans are also sorting and transporting here industry for new material produce. Table 2 has been shwon that Various waste materials and products for reuse in Rajshahi city.

A lot of money saved through its waste reduction, reuse, and pollution prevention assistance. After successful waste reuse that that act as *waste as a resource* not only protect the environment but they also gain an economic value. Materials exchange alliance resulted in a lot of money exchange of solid and hazardous waste. These exchanges saved businesses huge amount of money in avoided purchase and disposal costs. A survey from personal communication and estimates that for each ton of MSW reduced, businesses save 10,000 Tk. in avoided material purchases. Specific material savings include textiles and aluminum at 5,000 Tk. per ton, food at 2,000 Tk. per ton, and office paper at 1,500 Tk. per ton in the study area. Rajshahi City discarded over 20 million Tk. of white paper, office paper, cardboard, mixed paper, and newsprint; 3 million Tk. of plastic containers; 5.50 million Tk. of aluminum and other metals, and 1.50 million Tk. of glass in every year. These materials when completely recycled are worth 85 million Tk. at current market value. Instead, however, it cost roughly 45 million Tk. to manage these materials as waste rather than as resources. Once these materials are discarded, they can no longer generate future revenue streams, except for the materials that go to a waste-to-energy facility. Rajshahi City reuse industry adds significant value to our country’s economy. More than one-tenth of the economic activity related to recycling and reuse is generated by remanufacturing secondary materials into new products. Recycling equals jobs. If present reuse of solid waste trends continues, by 2020, approximately 8,000 people will be employed in the value-added recycling manufacturing industry.

Table 2: Various waste materials and products for reuse in Rajshahi city

Waste category	**Waste composition category**	**Ammount/day**	**Price(Taka)**
Cement bag	Made from paper/ plastic	1200 Nos.	5 /No
Plastic bag	Bags of rice, fertilizer, flour, pulse,	800 Nos	8/No
Cartons	Paper bags, cardboard, corrugated board, box board	500 Kg	150/ Kg
Paper	Old newspaper, magazines	1000 Kg	10/ Kg
Old Book	Newsprint, magazines, tissue, office paper and mixed paper	700 Kg	5/ Kg
Glass	Unbroken glass bottles	1000 Nos.	15 /Kg
Plastic bottles	Mineral water bottles, soft drinks bottles, oil bottles etc	3000 Nos.	20 / Kg
Metal	Tin can, wire, fence , aluminium can	300 Kg	25 / Kg
Leather & rubber	shoes, purse, rubber band and sponge	100 Kg	2 / Kg

Waste are being reused or recycled from hospital and/or dustbin which known as medical waste. Information was gathered on items they collect and sell from inside hospital and dustbin. Waste pickers and cleaners/aya collect recyclable item from the roadside, dustbin, hospital ward, operation theatre, ICU and pathology unit. These items were water bottle, iron/tin paper/carton/box, ampoule/vial, poly bag/plastics, saline bag/set, blood bag/set, canola/catheter, suction tube/rice tube, syringes, glass/broken glass, and surgery blade in the study area. Waste pickers sold those items. It was noticed that aya/cleaners also collect and sell few items informally without informing hospital authority. Nevertheless, all respondents stated abovementioned items as those are sold informally. Table 3 showed items sold from hospital waste. It can be seen that most of the items are plastic items except paper, glass and iron (Table 3). It was found that quite a good proportion of saline bag and syringes are sold. Therefore, there is a risk of illegal re-use of syringe and saline bag.

Table 3: Price (Tk/Kg) of item sold by waste picker of medical waste in RCC

Item	**Price (Tk/Kg)**
Syringe	10-20
Saline bag	10-24
Water bottle	7-10
Paper	2-5
Other plastic items	10-20
Broken glass	1.5-5
Metal container	4-10

4. CONCLUSION

Scrap metal, cement bags, paper, woods, cartons, cloths, foods, glass bottles, plastic goods are becoming a major stream in solid waste, which are being littered on landscape of Rajshahi city. In this paper, a survey on existing reuse of solid waste is conducted in the Rajshahi City, Bangladesh. Reuse of solid waste in Rajshahi city is completely under the informal private sectors. Housewives separate refuse of higher market value such as papers, bottles, fresh containers, old cloths, shoes etc. and sale them to street hawkers. Mostly children of slum dwellers known as “Tokai” carry out the second stage salvaging. They collect different items of low market value from waste collection bins. 100 shops are are available to buy the reusable solid waste. In this way, without any help of the authority some people who live on this profession engaged to clean reuse of some selected solid waste. So, reuse of solid waste saves money, materials exchange, creates jobs, save resources and lost revenues in the Rajshahi City. An environmentally sound reuse of solid waste improves the economic condition of concerned people and minimizes environmental hazards in Rajshahi City, Bangladesh.

5. RECOMMENDATIONS

Planned and organization reuse and recycling initiated by CBOs and fostered by RCC should be introduced involving householders, Tokai etc. Due to considerations should be given to the public initiatives and local people’s participation at every stage of solid waste management to make it sustainable. In this regard, the following recommendations should be implemented for sustainable reuse of solid waste in Rajshahi City, Bangladesh.

- Enhance public and private partnerships to increase the collection of additional recyclables, provide markets, and purchase recycled products.
- In order to ensure the success of these markets, consumers and businesses must purchase products with recycled content.
- Provide assistance to start-up and expanding businesses in Rajshahi City that use recycled material in their products. Assistance includes technical, financial, and marketing support.
- Provide updated information for recycling markets to local units of government and the public.
- Develop a financial performance benchmark for recycling companies.
- Encourage grant applications that focus on technology for reducing contaminants in paper recycling.
- Continue to pursue new market opportunities for used carpet in the state.
- Create opportunities with the manufacturing and business sectors for waste reduction and waste as a resource
- Increase reduction, reuse, and recycling by commercial sectors
- Reduction, reuse, and recycling efforts by non-residential sectors should be increased through technical assistance and financial incentives.
- Secure support of key trade and business associations and industry leaders.

- Incorporate recycling systems into new construction and remodeling projects.
- Accelerate the use and application of reusable transport packaging through technical and financial assistance.

ACKNOWLEDGEMENTS

The authors would like acknowledgement Md. Mojibor Rahman for encouragement write manuscript.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CHARACTERIZATION AND REMOVAL OF POLLUTANT IN LYSIMETER LEACHATE THROUGH THE SMALL SCALE BIOLOGICAL TREATMENT TECHNIQUES

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ABSTRACT

Landfill leachate is liquid that occasionally leak from municipal solid waste (MSW) deposited in landfill and enters the surrounding environment. The collection and treatment of leachate have become common practice in order to prevent environmental pollution. This paper concentrates the characterization and removal of pollutant in leachate generated from MSW deposited in landfill lysimeter through the small scale biological treatment techniques. In this study, biological treatment through aquatic reactors by duckweed *(Lemna minor), s*nail (*Pila globasa*), helencha (*Enhydra fluctuans*) and topapana (*Pistia stratiotes*) were carried out. Moreover, this study demonstrated that leachate is one of the major problems to overcome in managing the landfill and treatment has become common practice in order to prevent environmental pollution. To this attempt, treatment that has low cost, need less maintenance and environmental friendly was the target on how to treat the leachate. Here, it can be depicted that pollutant in leachate varied significantly for the biological treatment and it was more effective when the concentration of leachate was low and no single treatment showing the better performance for the removal of all pollutant.

Keywords: Landfill lysimeter, municipal solid waste, leachate, pollutant, removal efficiency.

1. INTRODUCTION

The term 'landfill' is used herein to describe a unit operation for final disposal of municipal solid waste (MSW)on land, designed and constructed with the objective of minimum impact to the environment. This term encompasses the other terms such as 'secured landfill' and 'engineered landfills' which are also sometimes applied to MSW disposal units (Tubtimthai 2003). The term 'landfill' can be treated as synonymous to 'sanitary landfill' of MSW, only if the latter is designed on the principle of waste containment and is characterized by the presence of a liner and leachate collection system to prevent ground water contamination. Sanitary landfill is one of the secure and safe facilities for the disposal of MSW; however, it needs high standard of environment protection in the operation of landfill (Davis and Cornwell 1998). Moreover, it is a well-suited method for managing of MSW all over the world and to investigate the performance of sanitary landfill the behavioral patterns namely; leachate generation, landfill gas (LFG) emissions etc. are required (Visvanathan et al. 2002). Lysimeter is a simulate form of sanitary landfill in the sense of control device. The word lysimeter is a combination of two Greek words *"lusis" means "solution"* and *"metron" means "measure"* and the original aim is to measure soil leaching (Rafizul et al. 2009a). Sanitary landfill plays a significant role for the disposal of MSW in most of the developing countries like Bangladesh (Alamgir et al. 2006 and WasteSafe 2005).

Chemically contaminated leachate is one of the by-products in landfill degradation reactions (Alamgir et al. 2006). A MSW landfill can be conceptualized as a biochemical reactor with MSW and water as the major inputs and with landfill gas and leachate as the principal outputs. Therefore, it is difficult to generalize the chemical composition of leachate that posses at a particular time of sampling. In addition, leachate quality may vary from time to time and site to site due to the variables such as waste composition, temperature, moisture content, climatic changes etc. (Alkalay et al. 1998). The collection and treatment of leachate have become common practice in order to prevent environmental pollution (Rafizul et al. 2009b). In addition, physico-chemical processes are used for the pretreatment of young leachate to make it amenable to biological treatment, and to hydrolyze some refractory organic compounds in older landfill leachate (Chain, 1977). In practice, a combination of physical, chemical and biological methods are usually used for the treatment of leachate since it is difficult to obtain satisfactory effluent water quality using any one of these methods alone (Kargi and

Pamukoglu 2003). To this attempt, treatment that low cost, need less maintenance and environmental friendly are the target on how to treat the leachate. This study has been tried to treat leachate by locally available aquatic reactor of preconditioned samples that does not require any expensive cost. From the findings it was observed that the characteristics of lysimeter leachate influenced significantly for different cap liner systems and the presence of CCLs as well as by applying the different biological treatment system and conditioning of leachate, the parameters was significantly changed against the control system.

2. METHODOLOGY ADOPTED

In the pilot scale treatment scheme, the adopted biological treatment through aquatic reactors by Duckweed *(Lemna minor)*, Snail (*Pila globasa*), Helencha (*Enhydra fluctuans*) and Topapana (*Pistia stratiotes*) was carried out as shown in Figure 1. Table 1 shows the leachate characteristics resulting from four samplings at two weeks interval for about 2 months from 1st August to 30th September 2009. Moreover, in the laboratory, leachate samples at varying concentrated were prepared through the dilution of raw leachate with tap water. At such stage, the prepared 100 concentrations of leachate means 100% of raw leachate was used and 50 concentration means 50% of tap water was mixed with 50% of raw leachate. Similarly 20 concentration of leachate means 80% of tap water was mixed with 20% of raw leachate. In addition, the leachate samples subsequent to pre-treatment through different aquatic reactors were analyzed based on the elapsed period from the treatment, leachate concentration (conditioning system) and removal efficiency.

Lysimeter leachate samples were analyzed for physico-chemical parameters of pH by pH meter (HACH, Model No. Sens ion 156), alkalinity by titration method, hardness by EDTA titrimetric method as well as COD by closed reflexive method as per the Standard Methods (APHA, 1998). However, the Ca Na, K and Mg ions were determined using flame atomic absorption spectrophotometer (VARIAN; AA/2400). In addition, Heavy metals viz., Cu, Cr, Cd, Ni, Pb, Mn, Fe and Zn were analyzed using spectrophotometer (HACH; DR/2400) as per the Standard Methods (APHA, 1998).

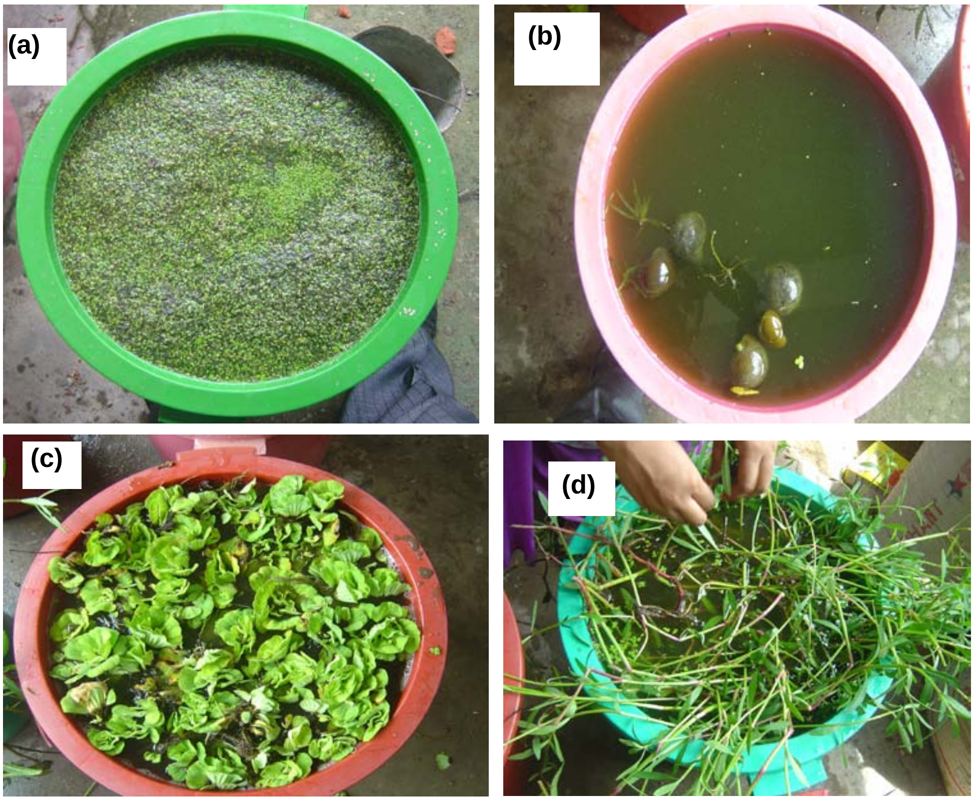

Figure 1 Different adopted biological treatment systems (a) duckweed *(Lemna minor)*, (b) snail (*Pila globasa)*, (c) topapana (*Pistia stratiotes*) and (d) helencha (*Enhydra fluctuans*).

Table 1 Initial quality of leachate samples

Parameter	mg/L
Chemical Oxygen Demand (COD)	3040
pH	9.33
Chloride	650
Total dissolved solid (TDS)	5580
Alkalinity	1600
Hardness	926
Calcium (Ca)	247
Sodium (Na)	2435
Iron	27.70
Copper (Cu)	0.98
Zinc (Zn)	1.38
Lead (Pb)	0.67
Nickel (Ni)	0.09

3. Results and Discussions

The leachate samples subsequent to pre-treatment through different aquatic reactors were analyzed based on the elapsed period from the treatment, leachate concentration (conditioning system) and removal efficiency and hence discussed in the following sections.

3.1 Effect on Leachate Concentration

In the laboratory, leachate samples at varying concentrated such as 100, 50 and 20 were prepared through the dilution of raw leachate with tap water as well as the effect and the consequences of different biological treatment were analyzed and hence discussed in followings.

Table 2 pH in 100 concentration of leachate at varying elapsed period

Elapsed period from treatment (weeks)	Concentration in control and treated leachate				
	Control	Topapana	Helencha	Duckweed	Snail
0	9.33				
7	9.24	9.22	8.97	9.1	8.27
14	9.20	9.12	8.93	9.06	8.33
21	9.23	9.1	8.9	8.93	8.43
28	9.22	9.11	8.85	8.88	8.31
35	9.28	9.07	8.79	8.72	8.28
42	9.18	9.0	8.45	8.59	8.2
49	8.92	8.69	8.37	8.7	8.27
56	8.85	8.51	8.28	8.69	8
63	8.72	8.42	8	8.45	7.87
70	8.44	8.14	7.87	8	7.65

3.1.1 Inorganic and Organic Compounds

The concentrations of inorganic and organic compounds in terms of pH, Cl^-, alkalinity, hardness, TDS and COD in leachate in relation to the increasing to elapsed period after treating with aquatic reactors were investigated and hence discussed in followings.

pH

The values of pH in leachate is considered to be the most significant parameter affecting leachate concentration in MSW landfill (Bilgili et al. 2007). Metal dissolution is enhanced at low pH which is one of the characteristics of young landfill leachate. MSW contains soil and organic matter that have significant sorptive capacity especially at high pH (Kjeldsen et al. 2002; Bozkurt et al. 1999). The values of pH in leachate after treated biologically at varying elapsed period provided in Table 2 and also in Figure 2. The Figure 2 illustrates the variation of pH with the increase of elapsed period from the treatment; to investigate the consequence of the different biological treatment systems by topapana, helencha, duckweed and snail. Based on Table 2 and Figure 2, it can be noted that pH decreased with the increases of elapsed period from the treatment for all the treatment systems until the end of this study. The Figure 2 also depicts that all the treatment systems provides the lower values of pH against of control system and the removal efficiency was found as positive for all the cases.

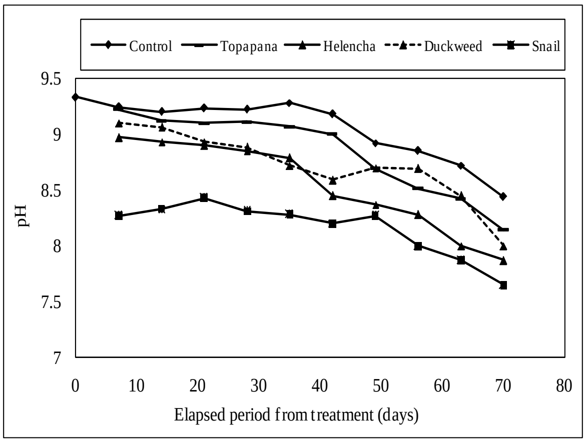

Figure 2 Variation of pH against of different treatment on leachate of 100 concentration

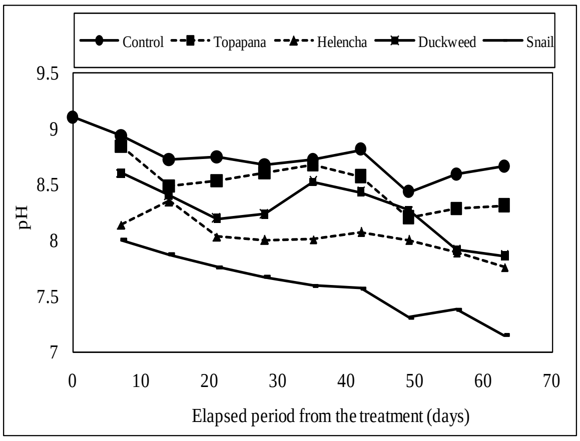

Figure 3 Variation of pH against of different treatment on leachate of 50 concentration

Moreover, the removal efficiency of snail was higher than that of other treatment systems. The values of pH for 50 and 20 concentration of leachate at varying elapsed period after applying biological treatment provided in Figures 3 and 4. However, the variation of pH at different biological treatment system for 100, 50 and 20 concentration of leachate at elapsed period 21 days provided in Figure 5. Here, it important to note that pollutant load (pH) decreases with the decreasing of leachate concentration. From Figure 5 it observed that pH fall down with the decreases of concentration of raw leachate by applying conditioning system. Moreover, the trend of 100 concentration of leachate sample having more values of pH than that of other two concentrated leachate samples. Diaz (1996) stated that preparation of leachate samples or prerequisite, such as dilution becomes effective to enhance the further treatment to some extent. It is evident from that more dilution condition of leachate, more enhance to the treatment capability for biological components and having low pH concentrations. However, Diaz (1996) also postulated that pH is the logarithmic hydrogen concentration of any liquid. Generally the pH value depends on temperature, amount of hydrogen ion etc. and it increases with the increase of hydrogen ion and the liquid becomes acidic.

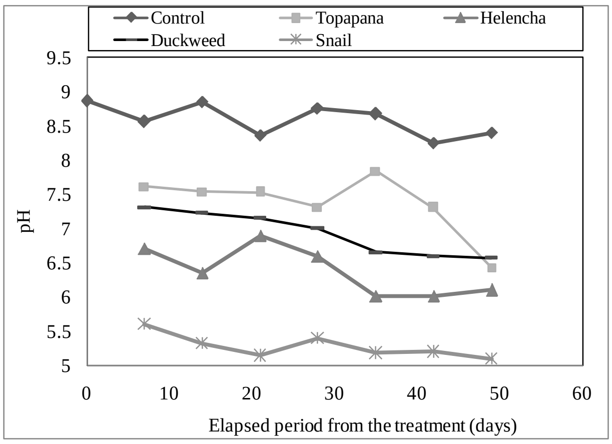

Figure 4 Variation of pH against of different treatment on leachate of 20 concentration

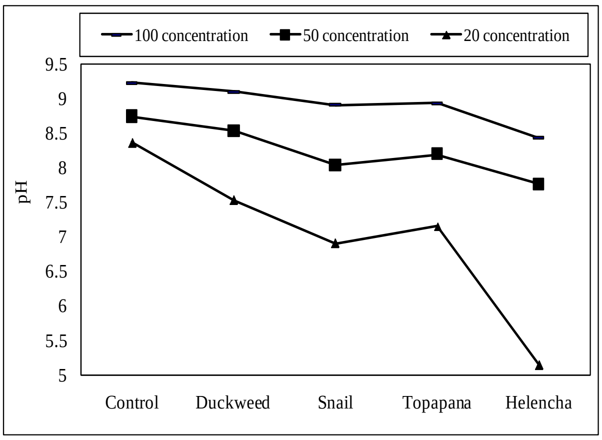

Figure 5 Comparison of pH among different treatment system after 21 days

COD Concentration

A decrease of COD occurs over the elapsed period of landfill and it can be attributed to a combination of reduction in organic contaminants and the increased biodegradation of organic compounds (Krug and Ham 1995). A constant decrease in COD is also expected as degradation of organic matter continues (Ehrig 1989). The values of COD in 100 concentration of leachate varies in relation to the increasing of elapsed period from the treatment systems by topapana, helencha, duckweed and snail untill the end of this study provided in Figure 6. This figure reveals that decreased with the increase of elapsed period and showing almost similar decreasing trend for all the treatment systems. In addition, the concentration of COD in case of control system was found as higher than that of the subjected treatment systems. Here, it can be noted that all the treatment systems showing positive and very efficient performance for the removal of COD concentration. However, the variation of COD at different biological treatment system for 100, 50 and 20 concentration of leachate at elapsed period 7 days provided in Figure 7. Here, it important to note that COD decreases with the decreasing of leachate concentration. From Figure 7 it observed that pH fall down with the decreases of concentration of raw leachate by applying conditioning system. Moreover, the trend of 100 concentration of leachate sample having more COD than that of other two concentrated leachate samples. COD is the measure of oxygen consumed during the oxidation of the oxidizable organic matter by a strong oxidizing agent. The determinations of COD are of great importance where BOD values cannot be determined accurately due to the presence of toxins and unfavorable conditions for growth of microorganism. The COD test remains a very important parameter in management and design of treatment plants because of its rapidity in determination.

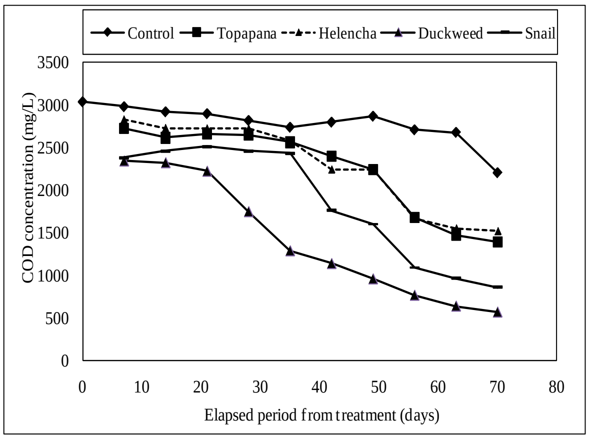

Figure 6 Variation of COD against of different treatment on leachate of 100 concentration

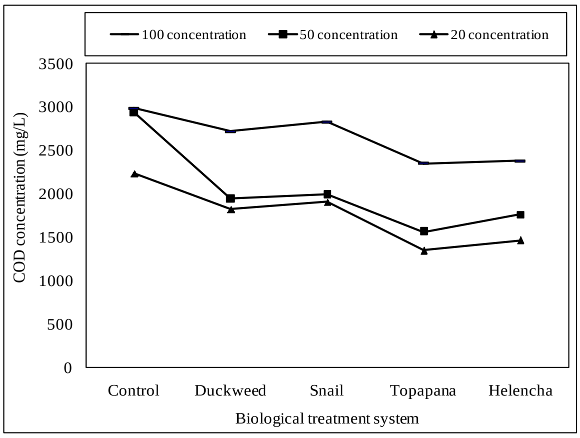

Figure 7 Comparison of COD among different treatment system after 7 days

3.1.2 Metals Concentration

The concentrations of metals in terms of Ca, Na, K, Mg, Cu, Zn, Pb and Ni in leachate in relation to the increasing to elapsed period after treating biologically were investigated in this study. The decreasing trend of Ca in leachate of 100 concentration in relation to the increasing of elapsed period after treating leachate biologically with locally available aquitic reactors of topapana, helencha, duckweed and snail provided in Figure 8. The Ca and Mg are two important ions in the evaluation of buffering capacity of leachate during the waste degradation in landfill (Karthikeyan et al. 2004). The Ca concentration has a tendency of complex formation with HCO_3^- and dissolved organic compounds and dissolution reactions involving calcite ($CaCO_3$) and may be siderite ($FeCO_3$) (Christensen et al. 2001). Thus, the dissolution process is closely linked to the dissolved carbonate compounds. Here, it is interesting to note that adopted biological treatment system deduced the values of Ca against the control system (without treatment) at varying elapsed period (Figure 8). Figure 8 reveals that Ca drops from 181-117, 212-147, 189-125.5 and 198.3-122 mg/L from the elapsed period 7-70 days, for topapana, helencha, duckweed and snail, respectively. For aparticular elapsed period say 49 days, the values of Ca reduces at 138, 166, 143.7 and 143 mg/L againist the initial quality of leachate for topapana, helencha, duckweed and snail, respectively. So, it can be concluded that topapara is more effective to remove Ca from leachate.

In contrary, the variation of Na in leachate of 50 concentration in relation to the increasing of elapsed period after treating with topapana, helencha, duckweed and snail evident in Figure 9. Figure 9 reveals that concentration of Na drops with the increasing of elapsed period from the treatment for all the cases of treatment system. Moreover, Figure 9 reveals that Na drops from 1456-752.6, 1845.4-1217.4, 1656.8-1002 and 1552-922 mg/L from the elapsed period 7-70 days, for topapana, helencha, duckweed and snail, respectively. For aparticular elapsed period say 63 days, the values of Na reduces at 867.4, 1332, 1032 and 976 mg/L againist the initial quality of leachate for topapana, helencha, duckweed and snail, respectively. So, it can be concluded that topapara is more effective to remove Na fro leachate. Moreover, all the applied treatment gives posive results for reducing Na in leachate.

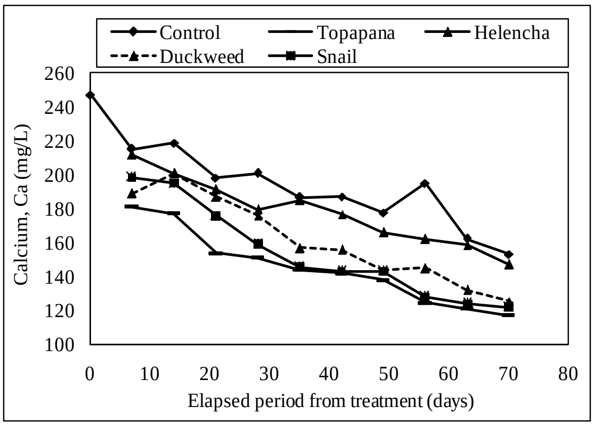

Figure 8 Variation of Ca against of different treatment on leachate of 100 concentration

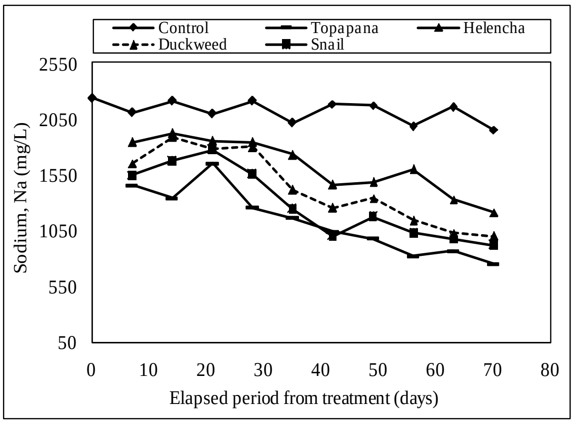

Figure 9 Variation of Na against of different treatment on leachate of 50 concentration

The variation of heavy metal of Cu in control sysyem and leachate of 100 concentration against of elapsed period after treating with topapana, helencha, duckweed and snail provided in Figure 10. The main processes for the low metal concentrations in leachate are sorption and precipitation. Solid wastes contain soils and organic matter, which have a significant sorptive capacity, especially at neutral to high pH prevailing in methonegenic leachate (Bozkurt et al. 1999). However, the solubility of the metals with sulfides and carbonates is low and sulfide precipitation is often cited as an explanation for low concentrations of heavy metal in leachate. Sulfides and carbonates are capable of forming precipitates with heavy metals of Cd, Ni, Zn, Cu and Pb. In general, sulfide precipitation is expected to dominate heavy metal attenuation as compared to complexation agents (Reinhart et al. 1998). Figure 10 reveals that concentration of Cu drops with the increasing of elapsed period from the treatment for all the cases of treatment system. Moreover, all the applied treatment gives posive results for reducing Cu in leachate. Figure 10 reveals that Cu drops from 0.94-0.66, 0.96-0.78, 84-0.62 and 0.90-0.65 mg/L from the elapsed period 7-70 days, for topapana, helencha, duckweed and snail, respectively. For aparticular elapsed period say 21 days, the values of Cu reduces at 0.92, 0.93, 0.77 and 0.87 mg/L againist the initial quality of leachate for topapana, helencha, duckweed and snail, respectively. So, it can be concluded that duckweed is more effective to remove Cu from leachate.

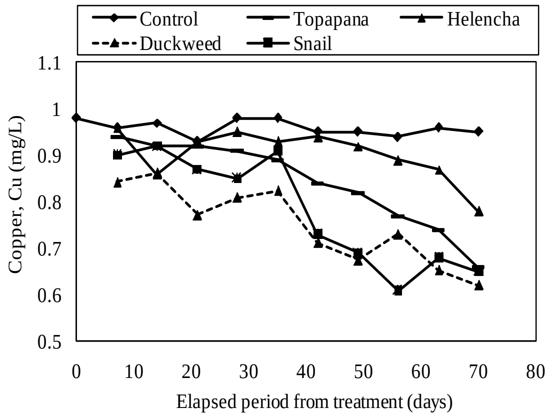

Figure 10 Variation of Cu against of different treatment on leachate of 100 concentration

The variation of Zn in leachate of 50 concentration and Pb in leachate of 100 concentration in relation to the variation of elapsed period provided in Figures 11 and 12, respectively. Figure 11 reveals that Zn in leachate drops from 0.98-0.54, 0.91-0.52, 67-0.28 and 0.72-0.31 mg/L from the elapsed period 7-70 days, for topapana, helencha, duckweed and snail, respectively. For aparticular elapsed period say 14 days, the values of Zn reduces at 0.92, 0.79, 0.63 and 0.71 mg/L againist the initial quality of leachate for topapana, helencha, duckweed and snail, respectively. So, it can be concluded that duckweed is more effective to remove Zn fro leachate. Moreover, based on Figure 12, it can be concluded that duckweed is more effective to remove Pb from leachate.

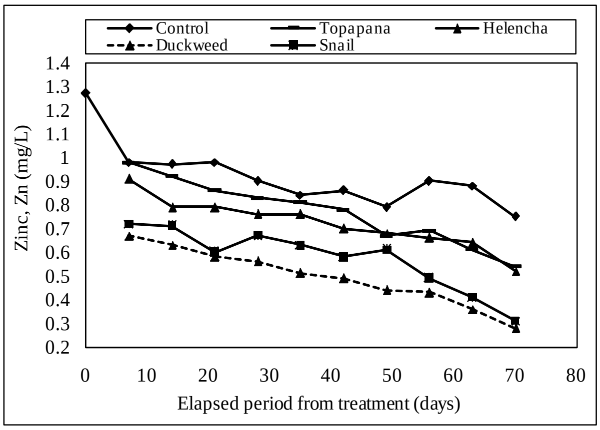

Figure 11 Variation of Zn against of different treatment on leachate of 50 concentration

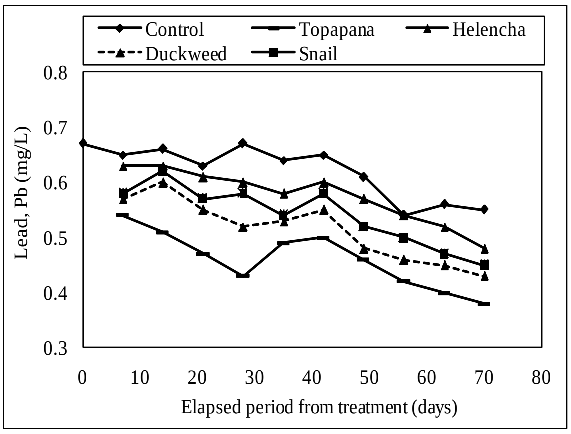

Figure 12 Variation of Pb against of different treatment on leachate of 100 concentration

3.2 Removal Efficiency of Pollutant in Leachate

The percentage of removal of different biological treatment systems based on the elapsed period from the treatment was analyzed and hence discussed in the following sections.

3.2.1 Inorganic and Organic compounds

The removal efficiency of pH ranging from 13-18% for different adopted biological treatment after the elapsed period of 70 days from the treatment for 100 concentration of leachate illustrates in Figure 13. The Figure 13 depicts that at 70 days of treatment, effective removal were achieved for topapana, helencha, duckweed and snail of 13, 16, 14 and 18%, respectively. Based on the findings it can be concluded that snail shows the highest performance of pH removal, which was obtained as 18%. Here, it can be noted that some of the treatment systems was very effective to remove some particular parameter like snail was more effective for the removal of pH in leachate. Experimental result reveals that for leachate of 100 concentration, the percent removal of pH ranging from 1-11, 2-11, 2-10, 2-11, 3-11,4-12, 7-11, 9-14, 10-16 and 13-18 % at the elapsed period 7, 14, 21, 28, 35, 42, 49, 56, 63 and 70 days for topapana, helencha, duckweed and snail, respectively. Here, it can be concluded that removal efficiency of pH increases with the increasing of elapsed period from the treatment shown in Figure 14. Figure 14 reveals that snail shows the highest removal of pH, while topopana having the lowest removal efficiency until the end of this study.

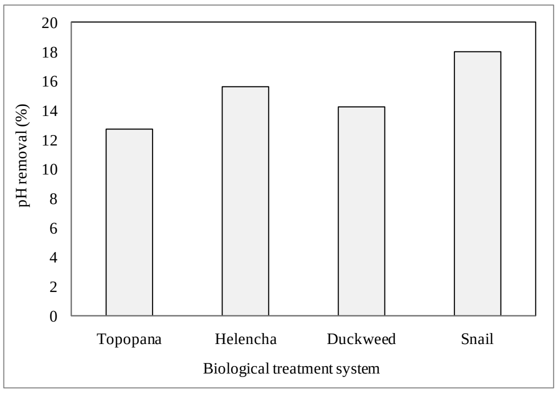

Figure 13 Reduction of pH in leachate of 100 concentration after 70 days from treatment.

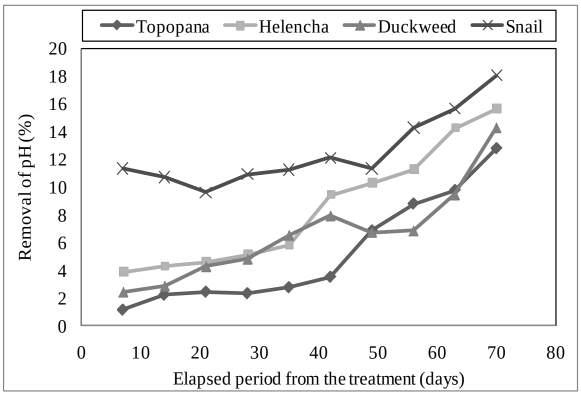

Figure 14 Percentage reduction of pH in leachate of 100 concentration at varying elapsed period

In contrary, removal efficiency of COD ranging from 51.55-81.84 % for different adopted biological treatment after the elapsed period of 63 days from the treatment for 50 concentration of leachate illustrates in Figure 15. The Figure 15 depicts that at 28 days of treatment, effective removal were achieved for topapana, helencha, duckweed and snail of 28.95, 25.43, 54.93 and 50.33%, respectively. Based on the findings it can be concluded that snail shows the highest performance of COD removal, which was obtained as 54.93%. Here, it can be noted that some of the treatment systems was very effective to remove some particular parameter like duckweed was more effective for the removal of COD in leachate. Here, it can be noted that removal efficiency of COD increases with the increasing of elapsed period from the treatment shown in Figure 16. Figure 16 reveals that duckweed shows the highest removal of COD, while helencha having the lowest removal efficiency until the end of this study.

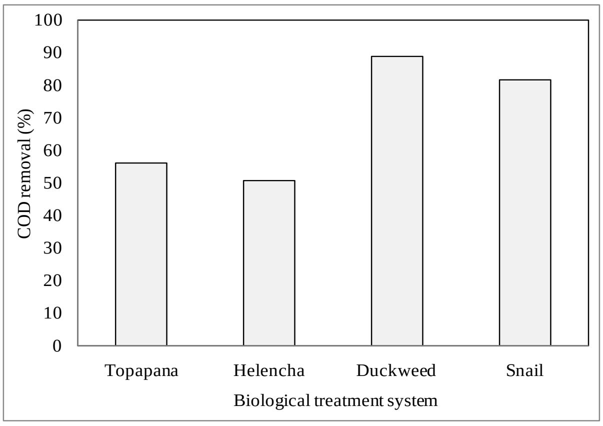

Figure 15 Percentage reduction of COD in leachate of 50 concentration after 63 days from treatment

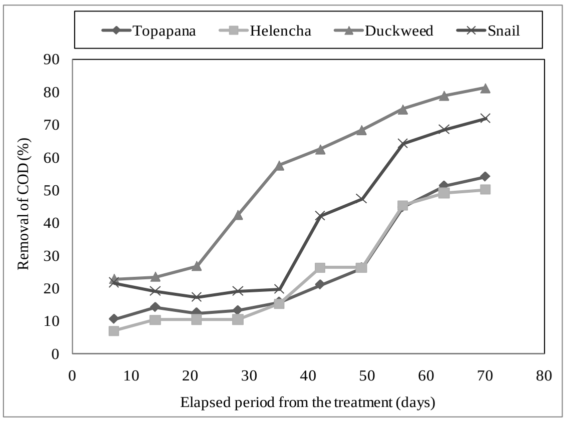

Figure 16 Percentage reduction of COD in leachate of 100 concentration at varying elapsed period

Figure 17 depicts the removal efficiency of TDS ranging from 20-47 % for different adopted biological treatment after the elapsed period of 28 days from the treatment for 100 concentration of leachate. At a particular elapsed period 70 days from the treatment treatment, effective removal was achieved for topapana, helencha, duckweed and snail of 28, 34, 51 and 59%, respectively. Figure 18 shows that snail had highest performance of TDS removal, which was obtained as 59%. Here, it can be noted that some of the treatment systems was very effective to remove some particular parameter like snail was more effective for the removal of TDS in leachate. Experimental result reveals that for leachate of 100 concentration, the percent removal of TDS ranging from 28-37, 20-41, 11-45, 20-47, 16-48, 22-48, 15-50, 26-53, 28-56 and 28-59 % at the elapsed period 7, 14, 21, 28, 35, 42, 49, 56, 63 and 70 days for topapana, helencha, duckweed and snail, respectively. Here, it can be concluded that removal efficiency of TDS increases with the increasing of elapsed period from the treatment (Figure 18). Figure 18 reveals that snail shows the highest removal of pH, while topopana having the lowest removal efficiency until the end of this study.

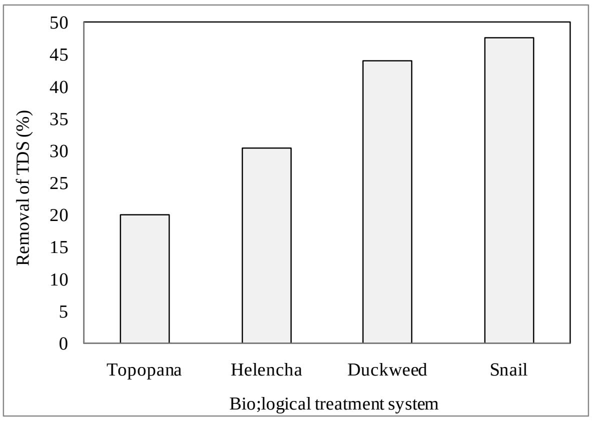

Figure 17 Percentage reduction of TDS in leachate of 100 concentration after 28 day from treatment

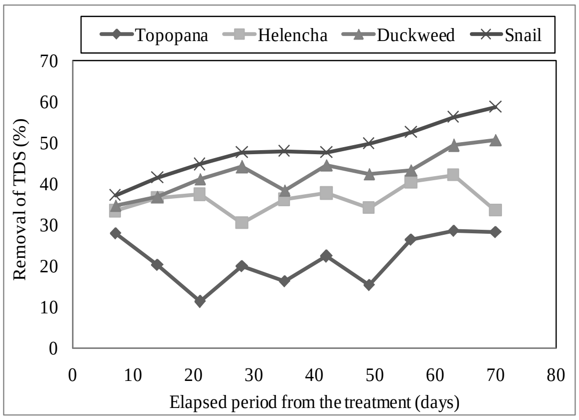

Figure 18 Percentage reduction of TDS in leachate of 100 concentration at varying elapsed period

3.2.2 Metals Concentration

The percent removal of metals in terms of Ca, Na, K, Mg, Cu, Zn, Pb and Ni in leachate in relation to the increasing to elapsed period after treating biologically was investigated in this study. The increasing trend of percent removal of Ca with the increasing of elapsed period provided in Table 3 and also in Figures 19 and 20. Here, it can be noted that removal efficiency increases with the increasing of elapsed (Table 3 and Figure 20). Figure 19 reveals that topopana was achieved to remove Ca at the highest of 53 % while, helencha having the lowest of 40% at the elapsed period of 70 days. In contrary, the removal efficiency for topopana of Ca in leachate at varying concentration shown in Figure 21 and it reveals that topopana was achieved the highest percent reduction of Ca when the concentration of leachate is 100, while removal is the lowest when concentration of leachate is 20.

Table 3 Percent reduction of Ca in leachate of 100 concentration after treating biologically

Elapsed period from the treatement (days)	Removal efficiency of Ca (%)			
	Topopana	Helencha	Duckweed	Snail
0				
7	27	14	23	20
14	28	19	19	21
21	38	23	24	29
28	39	27	29	36
35	42	25	37	41
42	43	28	37	42
49	44	33	42	42
56	50	34	41	48
63	51	36	47	50
70	53	40	49	51

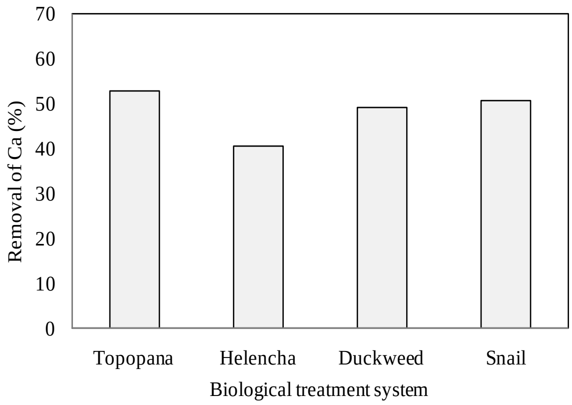

Figure 19 Percentage reduction of Ca in leachate of 100 concentration after 70 days from treatment

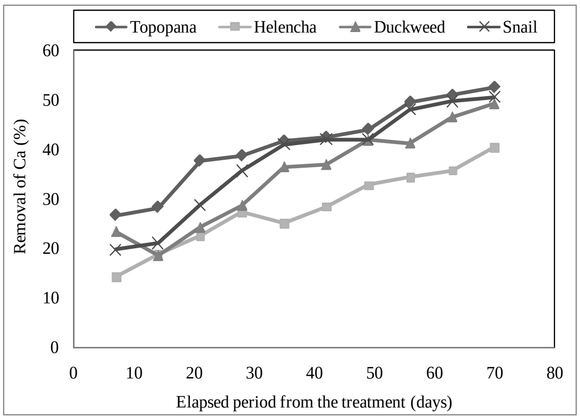

Figure 20 Percentage reduction of Ca in leachate of 100 concentration at varying elapsed period

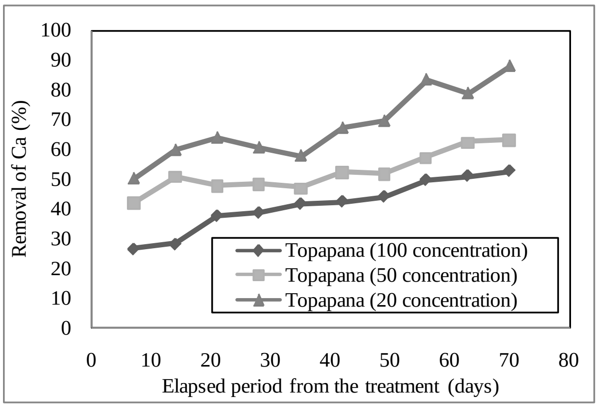

Figure 21 Percentage reduction efficiency of Ca in leachate for topapana at varying concentration

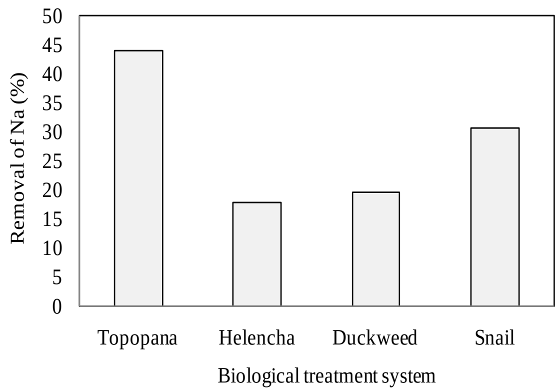

Figure 22 Percentage reduction of Na in leachate of 50 concentration after 28 days from treatment

In contrary, the percent reduction of Na in leachate of 50 concentration with topopana, helencha, duckweed and snail at the elapsed period of 28 days is provided in Figure 22. Figure 22 reveals that topopana was achieved to remove Na at the highest of 44 % while, helencha having the lowest of 18% at the elapsed period of 28 days. Moreover, the increasing trend of percent removal of Na with the increasing of elapsed period provided in Figure 23. Here, it can be noted that removal efficiency increases with the increasing of elapsed (Figure 23). Experimental result reveals that for leachate of 100 concentration, the percent removal of Na ranging from 9-17, 6-18, 6-16, 8-20, 6-19, 11-28, 11-23, 11-29, 11-36 and 12-40 % at the elapsed period 7, 14, 21, 28, 35, 42, 49, 56, 63 and 70 days for topapana, helencha, duckweed and snail, respectively. Here, it can be concluded that removal efficiency of Na increases with the increasing of elapsed period from the treatment (Figure 23). Figure 23 reveals that topopana shows the highest removal of Na, while helencha having the lowest removal efficiency until the end of this study. In contrary, the removal efficiency for topopana of Na in leachate at varying concentration shown in Figure 24 and it reveals that topopana was achieved the highest percent reduction of Na when the concentration of leachate is 100, while removal is the lowest when concentration of leachate is 20.

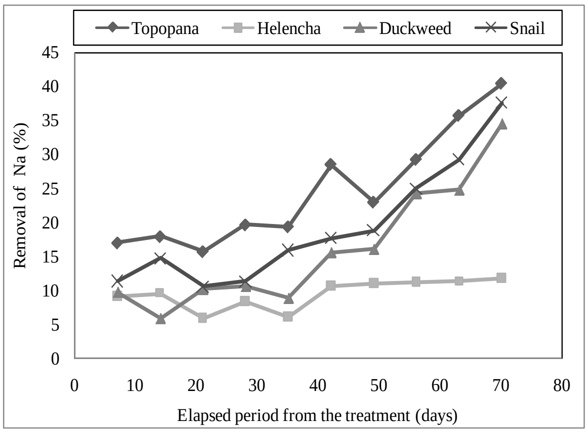

Figure 23 Percentage reduction of Na in leachate of 100 concentration at varying elapsed period

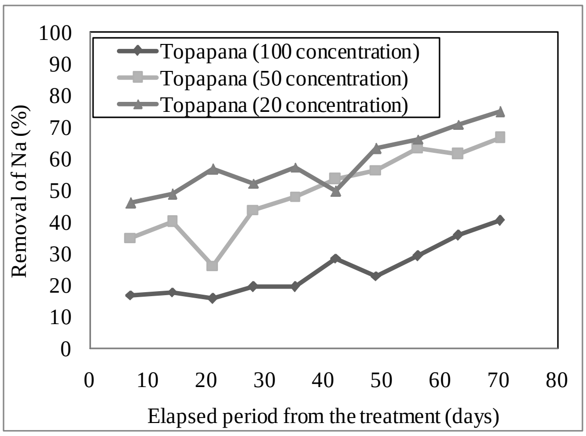

Figure 24 Percentage reduction efficiency of Na in leachate for topapana at varying concentration

In contrary, the removal efficiency of the values of Cu was ranging from 31.0 to 61.0% for the different adopted biological treatment after the elapsed period of 28 days from the treatment for 20 concentration of leachate as illustrates in Figure 25. The Figure 25 depicts that at 28 days of treatment, effective removal were achieved for topapana, helencha, duckweed and snail of 35.0, 31.0, 61.0 and 49.0%, respectively. Based on the findings it can be concluded that duckweed shows the highest performance of removal, which was obtained as 61.0% while, helencha having the lowest of 31% at the elapsed period of 28 days. Here, it can be noted that some of the treatment systems was very effective to remove some particular parameter like duckweed was more effective for the removal of Cu values. Moreover, the increasing trend of percent removal of Cu with the increasing of elapsed period for 50 concentration of leachate provided in Figure 26. Figure 26 reveals that duckweed shows the highest removal of Cu, while helencha having the lowest removal efficiency until the end of this study. In contrary, the removal efficiency for duckweed of Cu in leachate at varying concentration shown in Figure 27 and it reveals that duckweed was achieved the highest percent reduction of Cu when the concentration of leachate is 100, while removal is the lowest when concentration of leachate is 20.

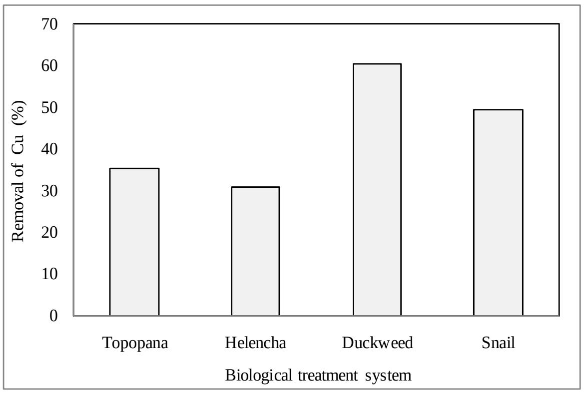

Figure 25 Percentage reduction of Cu in leachate of 20 concentration after 28 days from treatment

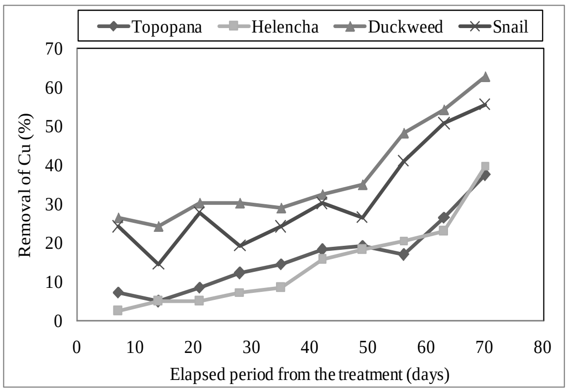

Figure 26 Percentage reduction of Na in leachate of 50 concentration at varying elapsed period

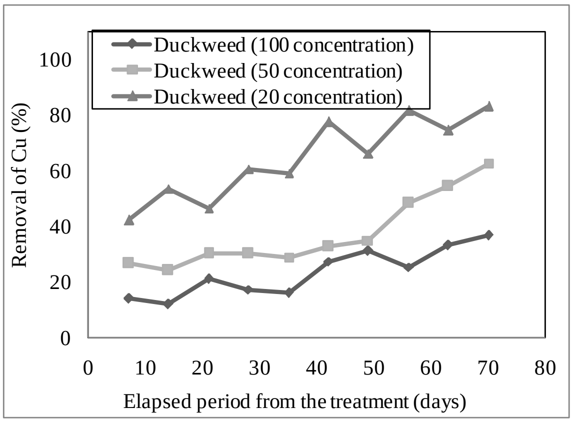

Figure 27 Percentage reduction efficiency of Cu in leachate for duckweed at varying concentration

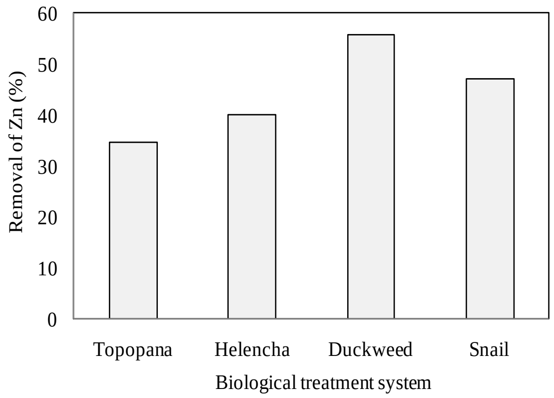

Figure 28 Percentage reduction of Zn in leachate of 50 concentration after 28 days from treatment

The percent reduction of Zn in leachate of 50 concentration with topopana, helencha, duckweed and snail at the elapsed period of 28 days is provided in Figure 28. Figure 28 reveals that duckweed was achieved to remove Zn at the highest of 56 % while, topopana having the lowest of 35% at the elapsed period of 28 days. Moreover, the removal efficiency of the values of Zn was ranging from 35.0 to 56.0% for the different adopted biological treatment after the elapsed period of 28 days from the treatment for 50 concentration of leachate. However, the increasing trend of percent removal of Zn with the increasing of elapsed period provided in Figure 29. Experimental result reveals that for leachate of 100 concentration, the percent removal of Zn ranging from 20-41, 30-43, 33- 44, 31-41, 31-40, 32-48, 33-54, 36-47, 37-53 and 41-54 % at the elapsed period 7, 14, 21, 28, 35, 42, 49, 56, 63 and 70 days for topapana, helencha, duckweed and snail, respectively. Here, it can be concluded that removal efficiency of Zn increases with the increasing of elapsed period from the treatment. Figure 29 reveals that duckweed shows the highest removal of Cu, while helencha having the lowest removal efficiency until the end of this study. In contrary, the increasing trend of removal efficiency of Pb in leachate of 100 concentration with the increasing of elapsed period from the treatment shown in Figure 30. Figure 30 reveals that topopana shows the highest removal of Pb, while helencha having the lowest removal efficiency until the end of this study.

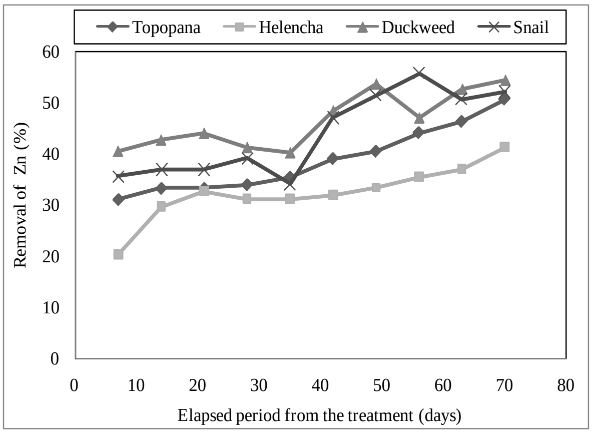

Figure 29 Percentage reduction of Zn in leachate of 100 concentration at varying elapsed period

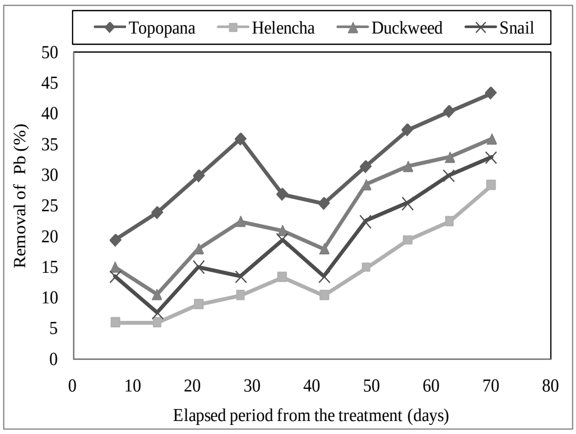

Figure 30 Percentage reduction of Pb in leachate of 100 concentration at varying elapsed period

4. Conclusions

The adopted biological treatment systems were more effective when the concentration was low and no single treatment system showing commonly the better performance for all the pollutant in leachate. Result showed that some of the treatment systems were very effective to remove some particular parameter. Here, it was noticed that snail was found to be more effective for the removal of pH, TDS and chloride, while, topapana for Ca, Na and Pb, helencha for Fe as well as duckweed for COD, Cu and Zn. This study has been tried to treat leachate by locally available aquatic reactor of preconditioned that does not require any expensive cost. Finally it can be concluded that locally available aquatic reactors can be used satisfactory for biological treatment of leachate.

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COMMUNITY PARTICIPATION IN CONSTRUCTION AND RENOVATION OF SANITARY LATRINES IN THE SLUMS OF KHULNA CITY

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ABSTRACT

Sanitary latrines contribute significantly in reducing diseases, ensure environmental sound health and improve socio-economic condition of the people. There are more than 520 slum and squatter settlements in the third largest industrial Khulna city where more than 20% people of the total city people of 1.5 million live. Poor people live in the slums and they often are unable to construct and renovate sanitary latrines by their own. They construct and renovate the latrines in share. About 43% and 31% households of the slums of Khulna city use community latrines and pit latrines in shares. Pit latrines are used by 2 to 3 families and community latrines by 15-30 families. Thus, community participation is found in selection of sites, choosing suitable technology, sharing in construction and renovation cost, monitoring and evaluation of construction and renovation works etc. all these improves the sanitary condition of the slums towards better life of the slum people. The paper highlights the ability and level of participation of slum community people in construction and renovation of latrines; problems of community participation and recommendations for ensuring community participation in construction and renovation of latrines.

Keywords: *Community participation, environmental sanitation, sanitary latrines, construction and renovation*

1. INTRODUCTION

Despite huge investments in sanitation during the United Nations International Drinking Water Supply and Sanitation Decade (1981-90), over 2.4 billion people still lack access to adequate sanitation services and an estimated 3 million children die each year of dehydration related to diarrhoea (WHO, 2000). According to the UN Habitat, an estimated 77% of the people in developing countries are expected to live in urban areas by the year 2025, and half of them in informal settlements; thus the provision of sanitation services in poor urban areas is a major challenge for the 21st century (Manase Gift., et. al, 2000). Bangladesh with a population of 140 million in a relatively small land is one of the world's most densely populated countries. Compounded by the desperately poor conditions in which many people live, in both rural and urban areas, lack of access to safe water and sanitation leads to high levels of water-related diseases throughout the country. As a result many people in Bangladesh find it impossible to escape the downward spiral of poverty and disease. Chattopadhyay (2003) in the paper titled “Sustainable Water Supply and Sanitation-City Planners Role” mentioned that it is the step into the 21st century, it is realized that the trend towards urbanization is posing over-increasing problems with respect to sanitation. The rate of growth of population, especially in the urban areas, is far exceeding that of rural areas in most countries. A vast majority of this urban population lives in slums, derelict areas of towns and cities, and sprawling peri-urban fringe areas. The level of services of sanitation is extremely poor in these areas. He also mentioned that while the direct effect of poor water supply and inadequate sanitation are diseases, the indirect effect amounts to lost earnings and lost educational opportunity for young people. Greater attention, better planning, better operation, maintenance, and management, for water supply and sanitation are desperately needed to improve the liveability of these settlements. He also mentioned that the World Bank and other bilateral donors are adopting a demand-responsive approach. It is recognized that for sustainability of a project, widespread stakeholder consultation is necessary. The questions of eligibility, choice of technology, cost sharing, and involvement of community, for operation and maintenance, have assumed significance. It is felt that the technology chosen should give the community the highest service level, which is willing to pay for, will benefit from, and has the institutional capacity to sustain. In the case of sanitation, NGOs are providing extension services with the aim of increasing poor people’s access to environmental sanitation to reduce the child mortality rate. Community participation develops sense of ownership and greater willingness to help programmes succeed. Training, orientation, planning meeting etc. with community participation for construction and renovation are important for making sanitation system sustainable. Thus, current strategies emphasize that it is better to do the work properly ensuring sustainability rather than meeting short-term targets and risking long-term failure. Plainly stated, this means that it is better to do it right rather than construct quickly without consulting user communities or checking environmental standards among other things.

2. OBJECTIVES OF THE STUDY

Objectives of the study are:

- To study the level of community participation in different aspects of construction and renovation of latrines in the slums of Khulna City
- To find out the problems of ensuring community participation in construction and renovation of sanitary latrines
- To draw some suggestions for enhancing community participation in the construction and renovation of sanitary latrines

3. METHODOLOGY

The study was conducted during 2008-2009 in three slums of Khulna City, namely Rupsha slum on Government khas land and Christian community land; Montu Colony slum on Bangladesh Railway land; and Kulibagan slum on Ispahani Company private land. Primary data for the study was collected through structured household questionnaire survey, interview with group and committee members of sanitation and related projects, and concerned GO-NGO staffs. A total of 384 households (22%) were randomly selected and surveyed from the slums, where 215 households were from Rupsha slum, 75 households were from Montu Colony slum and 94 households were from Kulibagan slum. Books, project reports and implementation guidelines, journals, newspapers etc. were reviewed as secondary data sources.

4. KHULNA CITY AND STUDY SLUMS

Khulna is the 3rd largest industrial city of Bangladesh. It is a divisional city and acts as regional hub of administrative, institutional, commercial and academic affairs. It is located on the banks of the Rupsha and the Bhairab rivers. The city covers an area of 45.65 square kilometers with a population of near about 1.5 million. There are 520 slums in Khulna city. Most of the slums of Khulna City have established after 1971, the year of independence of Bangladesh. About 73% slums have been established after 1971. Among 73% slums, 15.2% in 1971-75, 15.8% in 1976-80, 12.9% in 1981-85, 10.2% in 1986-90 and the rest 18% in 1991-2005. In the slums of Khulna city 188,442 poor and landless people live. Density of population per acre and per kilometer in the slum area is 538 and 132,988 respectively (CUS, 2006). Following three slums, located in three diverse locations of Khulna city are the study slums:

Rupsha Slum

Rupsha Slum is situated in Ward 22 of Khulna City Corporation. It is 2.5 kilometers on the north-east side from the Central Business District (CBD). It is on the west side of Rupsha embankment of BWDB which is along the western bank of River Rupsha. Total land area of Char Rupsha Slum is 4.0 acre. The strip of land just along the embankment is owned by the Government and the rest area after along the western side is owned by the Christian Baptist Church. The slum was established nearly in 1960s for settling the poor workers of nearby Dada Match Factory, Rupsha; Khulna Shipyard, Labonchara; and water transports namely boats, launches and steamers. Around 5,500 people live in around 1,000 households. The slum is bounded by BWDB embankment and Rupsha River on the east; Caritas and LGED office premises on the west; Rupsha Natun Bazar on the north; and Rupsha bus stand on the south.

Montu Colony Slum

Joragate Montu Colony Slum is situated in 21 Ward of Khulna City Corporation. The land is owned by the Railway Department of the Government of Bangladesh. It is established immediate after the liberation war of 1971. First time a population of around 75,000 lived here but it reduced at around 35,000 after an eviction during the Mujib regime. The slum also faced an eviction during the Zia regime. As the land is taken lease from the Department of Railway by Mr. Montu, a contractor of the area of that time, the area is named as Montu Colony. There are 350 households in the slum with a population of around 1,925. The 5 no. ghat (landing point at river bank) of Bhairab river was famous and busy for shipment of goods from cargo, launches, trawlers and boats to Khulna city and adjacent industrial area. In past, transport, goods landing and industrial laborers lived in this slum. The Montu Colony slum is bounded by Joragate Rail crossing on the west, Koiladepo slum and Bhairab river on the east, KCC Kitchen market (Kutcha bazar) and the road of 7 no. ghat on the north and Railway vacant land and *Natun Bastee* on the south.

Kulibagan Slum

Kulibagan slum is in Ward 3 of Khulna City Corporation. The Slum is located about one kilometer away from Daulatpur traffic island towards north at the western side of Khulna-Jessore highway. Area of the slum is 2.5 acre. Kulibagan is at the southern side of Moheswarpasa Modhodanga road which is originated from the Khulna-Jessore highway towards west. The slums are bounded by Shashibhusan High School at the south, Sadhubagan slum and Urban Primary Health Care Center of KCC at the north, Khulna-Jessore Highway on the east and Daulatpur Police Station on the west. Around 2,390 people live in around 435 households and workers mess of the slum. The land of the slum is owned by Ispahani Group of Industries. The slum is established in 1953 at the time of starting the Khulna Industries Ltd. The Ispahani Company allowed the laborers and staff of the Khulna Industries Ltd. to reside on the land.

5. COMMUNITY PARTICIPATION IN BANGLADESH NATIONAL POLICY ON SANITATION

Community or user-owned water and sanitation facilities requires an appropriate feeling of ownership, accepting voluntary functions, strong internal communication and good back-up from the outside including relevant training. Too often the 'coverage' figures given for water and sanitation facilities are not real. Ownership by one or a few powerful families may lead to good O&M but will defeat the purpose of the improved water supply and sanitation facilities by not providing safe water and sanitation facilities to the whole potential user group. Certain physical pre-conditions are also needed such as: good quality construction, acceptable quality of water, availability of spare parts and oil/petrol or electricity if relevant. Transparent and honest financial management are particularly important for sustaining community managed schemes. The Bangladesh National Policy for Safe Water Supply & Sanitation 1998 is clear in its focus on the community participation for sanitation management. It states that local government and communities shall be the focus of all activities relating to sanitation. All other stakeholders including the private sector and NGOs shall provide inputs into the development of the sector within the purview of overall government policy with DPHE ensuring coordination (LGD, 1998).

6. RESULTS AND THE FINDINGS

6.1 Organizations for Sanitation and Sanitation Projects in the Slums

Government bodies and NGOs implement some slum improvement programs in pursuit of meeting the needs of the poor slum people in the urban area. The organizations that absolutely work on sanitation are the sanitation organizations. There are some organizations that keep sanitation as one of the major components of their implemented development projects. List of sanitation and related projects implemented in the slums in Khulna City during the period 1985-2009 is enumerated in the Table 1. Once the previous sanitation projects and sanitation components do not sustain, repeated sanitation projects are needed for implementation in the slums.

6.1.1 Organizations for sanitation in Khulna City

There are specific government organizations (GOs) and NGOs in Khulna City that implemented and still now implementing some sanitation status improvement projects in the slum areas since 1985.

Government Organizations

LGED, KCC and DPHE have been working in Khulna City for slum status improvement since 1985 through SIP. WASA has also been established in Khulna City in 2008 for providing water and sanitation services in Khulna City. Before establishing KWASA, KCC was responsible for the water supply and sanitation services in Khulna City. All the staff of the Water Supply Department of KCC has been shifted in KWASA. Activities of KWASA at this primary stage are preparation of plans for water supply networks and infrastructures, taking initiatives for collection of arrear water bills and system development etc. Firstly, KWASA will work for water supply and gradually it will work for sanitation and drainage.

Non Government Organization (NGO)

World Vision (WV) mainly works with temporary slums that are of cluster type and poor in drainage, road and sanitation condition on government land which are under threat of eviction. People of such slums have lack of land to establish latrine for individual households. It has started its development interventions in the Khulna City slums around 10 years ago. It provides supports to establish sanitary latrines and construct low cost houses. All the groups of WV are of female. Each group consists of 30 women. Nabolok Parishad, a local NGO has been working in Khulna City with implementation of the ASEH project funded by WaterAid Bangladesh (WAB) since March 2005. WAB has been supporting both financial and technical supports to Nabolok for promotion of sanitation in poor urban areas especially in the slums.

Private Sector Organizations

In Khulna City there are a large number of profit making public sector organizations in the form of contractors, enterprises and shops for production, marketing, installation, maintenance and repairing of both low cost and high cost sanitary latrines. These private organizations also work as supporting organizations for installation, repair and maintenance of water supply systems and facilities.

6.1.2 Sanitation and related projects in the slums of Khulna City

A number of sanitation and related projects starting with Slum Improvement Project (SIP) since 1985 have been implemented in Khulna City. Brief of these projects with title, donors, implementation period, implementing agency and project area are shown in the Table 1.

Table 1: Sanitation and related projects in the slums in Khulna city

Sl.	Title of the Project	Donor	Period	Implementing Agency	Project Area/KCC Wards
1.	Slum Improvement Project (SIP) Phase I & II	GOB, UNICEF	1985-1995	LGED through KCC	All Wards (1 to 31)
2.	Urban Basic Service Delivery Project (UBSDP)	GOB, UNICEF	1994-1996	LGED through KCC	All Wards (1 to 31)
3.	Support for Basic Service in Urban Areas Project (SBSUAP) Component-Part II	GOB, UNICEF	January 2001-June 2006	LGED through KCC	All Wards (1 to 31)
4.	Urban Slums and Fringes Project	GOB, UNICEF	1999/2003-2006	LGED through KCC	19 Wards (3, 5, 7, 8, 9, 10, 12, 13, 15, 17, 18, 20, 22, 25, 26, 27, 29, 30 & 31)
5.	Local Partnership for Urban Poverty Alleviation Project (LPUPAP)- Part I	GOB, UNDP, UNCHS	January 2000-June 2006	LGED through KCC	All Wards (1 to 31)
6.	Urban Partnership for Poverty Reduction Project (UPPRP)	GOB, UNDP, DFID, UN-HABITAT	July 2006-June 2013	LGED through KCC	All Wards (1 to 31)
7.	Advancing Sustainable Environmental Health (ASEH)	WaterAid Bangladesh	April 2005-March 2008	Nabolok Parishad	All Wards (1 to 31)
8.	Micro-credit and Child Education Sponsorship Project	USAID	1998 to till now	World Vision	Slums and fringe areas

Source: KCC, DPHE and Nabolok Parishad Office, 2008.

Slum Improvement Project (SIP)

In view to fulfill the government planning and providing facilities to low income people of the country LGED with the help of UNICEF worked for providing basic urban service through the Slum Improvement Project (SIP) since 1985. Various service facilities have been provided to 45,000 families of 4 cities and 21 pourashavas. The facilities include establishment of tube-well, sanitary latrine, construction of footpath, drain, dustbin, street light, primary health care, satellite school, self employment human resources development, IGA skill development training and various package programs for improving basic services (PP LPUPAP, 2001). As a result slum dwellers of 225 slums of 25 municipalities/cities have improved their living condition and environment. With the rapid increase of urban population, demand for urban infrastructure and basic services has also been increased. But the facilities provided through SIP were limited compared to the increased demand. SIP was implemented in almost all the slums of Khulna City during the period 1985-1995. Some tube-wells and sanitary latrines were established in the slums. Households of the study slums could not clearly memorize about the sanitation facilities provided under the SIP. This is due to lack of effective community mobilization and community participation mechanisms in the project activities.

Urban Basic Service Delivery Project (UBSDP)

UNICEF assisted Urban Basic Service Delivery Project (UBSDP) established management and implementation structures in 4 city corporations and 21 pourashavas to address vulnerable population groups. The evaluation of this project done by Center for Urban Studies (CUS) highlighted the importance of implementing activities through local government structures. But there is no other regular structure for any of the local Govt. bodies in Bangladesh to deliver adequate basic services to the urban poor excepting a few slum development staff in

Dhaka City Corporation H.Q. The Urban Basic Service Delivery Project is an attempt to bridge the gaps for better coordination among the local Govt. authorities and development partners. The evaluation study recommended continuation of UBSDP and also its possible extension to another phase. CUS also recommended that UNICEF should support on advocacy campaign to strengthen the urban local governments and the civil society to work for improving the quality of life of the urban poor. There were different committees in the project-Central Coordination Committee (CCC), chaired by LGD-Secretary, Project Coordination Committee (PCC), chaired by respective Mayor/Pourashava Chairman and Project Implementation Committee (PIC), chaired by respective Ward Commissioners oversaw and implemented all UNICEF assisted project activities in the urban areas. Under the project 35 UDCs were established in Khulna City to perform basic health care services including Expanded Programme of Immunization (EPI), Vitamin A Capsule (VAC) distribution, growth monitoring, hygiene education, sanitation, IGA & education for children.

Support for Basic Service in Urban Areas Project (SBSUAP)

Support for Basic Service in Urban Areas Project (SBSUAP) was implemented in 206 Urban Development Centers (UDCs) and 21 Pourashavas functioning under UBSDP. The reach of UDCs was increased from 2,000 to 10,000 urban population by involving major NGOs and partners like ADB. Each UDC with existing staff provided basic health care services including Expanded Programme of Immunization (EPI), Vitamin A Capsule (VAC) distribution, growth monitoring, hygiene education, sanitation, IGA & education for children. UBSDP project staff with City Corporations/Pourashavas Health Assistants provided same services in the urban areas. UNICEF closely worked with UNDP which supported LPUPAP of LGD in the area of micro-credit and basic infrastructure development including sanitation facilities. UNICEF assisted SBSUAP mobilized national programmes resources of Health & Nutrition, Water and Environmental Sanitation (WES) for sanitation education & installation of tubewell/latrine etc., Child Development & Education, Protection, Planning & Communication Program towards meeting the basic needs of the children & women in the urban areas with special focus to the urban poor.

Local Partnership for Urban Poverty Alleviation Project (LPUPAP)

The Government of Bangladesh and UNDP implemented the Local Partnership for Urban Poverty Alleviation Project (LPUPAP) to support the efforts, energies and resources of the urban poor to get out of the cycle of poverty. The project began its operation in January 2000 and continued til June 2006. Subsequent delays in preparation of the second phase and concerns to maintain the momentum of field activities resulted in LPUPAP being further extended to June 2007. The funds from UNDP have totaled US$ 18.797 million. The government of Bangladesh contribution was Taka 60.6 million. LPUPAP worked in four city corporations-Chittagong, Khulna, Rajshahi, Barisal-and seven pourashavas-Kustia, Gopalganj, Bogra, Sirajganj, Mymemsingh, Narayanganj and Hobiganj. Under the project, the poor of the cities were organized and empowered to identify their own problems, find out solutions and implement their own programmes to address their needs. LPUPAP built the capacity of community organizations to establish creative partnerships with local government and with other agencies including NGOs and private sector businesses. LPUPAP adopted a community-based, demand driven approach with strong links to the local government through the elected representatives and government officials. Project staffs were in a facilitating role. The urban poor communities were the main project partners and community mobilization and empowerment began with the formation of Primary Groups (of about 20 families) and support to community management of regular savings, leading over time to a community-managed micro-credit fund. Primary Groups then grouped themselves into Community Development Committees (CDCs) representing 200-300 families. Of the Primary Groups, 90% were women’s groups and consequently women were strongly represented in the leadership of the CDCs. LPUPAP had three main components. A *Community Development Fund (CDF)* which financed the construction of basic infrastructure in the communities. The CDCs were supported to undertake participatory community action planning and the priority infrastructure was then constructed through community contracts signed between the local government and the concerned CDC. Thus, all construction was managed and overseen by the community leaders, majority women. LPUPAP experience is that such construction has been effective, efficient and of high quality. A *Poverty Alleviation Fund (PAF)* supported apprenticeship training, again planned, prioritized and managed by the community and implemented through contracts between the local government and the CDCs. A third component supported *Community Empowerment and Capacity* building of both communities and local government. In addition to more formal workshop and field level training, community empowerment and capacity building was strengthened by peer-to-peer learning between CDCs within and between towns. Elected representatives and local government officials accompanied community visits to other towns. This helped to develop their awareness of the benefits of the project procedures and their commitment to a community-managed approach. LPUPAP project was implemented in Rupsha and Kulibagan slums. The project had no intervention in Montu Colony slum. Training and capacity building has been a central pillar of LPUPAP. Right from the initial project orientation training for project staff, training has preceded and accompanied all aspects of project implementation. Staff trainings were provided on Community mobilization, Savings and credit, Community action planning, Community contract management: both infrastructure construction and apprenticeship training and Monitoring. Trainings for community were on Participatory urban appraisal, Community action planning, Gender and leadership, Savings and credit management, Accounting, PAF management, Apprenticeship programme, Community contract implementation and management, Construction of physical works and accounting, Operation and maintenance of physical works, Participatory monitoring and Auditing savings and credit (LPUPAP Final Report, 2007).

Advancing Sustainable Environmental Health (ASEH)

Nabolok Parishad implemented Advancing Sustainable Environmental Health (ASEH) project funded by WaterAid Bangladesh in all the wards of KCC during the period April 2005-March 2008. Giving priority of community participation in all spheres of the project is one of the strategies for attaining sustainability of the ASEH project. The project had direct control on technology options of latrines, ability to pay analysis of the community and ensure cost sharing of the community through maintaining bank accounts. ASEH has established Hardware Management Committees in the community of the slums. Inadequate fund for continuation of the sanitation project in future is the major limitation of the ASEH project. Community Situation Analysis (CSA), mass awareness, opinion sharing, knowledge level and practice level monitoring, performing hygiene promotion sessions in the courtyards, tea-stalls, mosques and schools etc. are the activities of ASEH with and for the community people.

Micro-Credit and Child Education Sponsorship Project

World Vision directly does not implement sanitation project in the slum areas. It works in the slums of temporary pattern on unauthorized or government or government khas land. It has been implementing Micro-Credit and Child Education Sponsorship Project in Montu Colony slum from around 1998. It provided the slum households that are the members of its Micro-Credit and Child Education Sponsorship Project with single pit latrines at subsidized rate.

6.2 Different Types of Latrines Used by Slum People of Khulna City

Table 2 reveals that about 55.5%, 20.1% and 15.2% households of the study slums used septic tank system community latrines, twin pit water sealed latrines and simple single pit latrines respectively. Around 74% and 57% households of Kulibagan and Rupsha slums used septic tank system community latrines. Maximum households (65%) of Montu Colony slum were found to use simple single pit latrines. In spite of intervention of sanitation development projects, 15% and 4% households of Rupsha and Montu Colony slum were found to use hanging latrines.

Table 2: Access to different types of latrine by households

Name of slums	**Types of the latrine**				**Total**
	Septic tank system community latrine	**Twin pit water sealed latrine**	**Simple single pit latrine**	**Hanging latrine**	
Rupsha slum	123	37	24	32	215
	57.0	17.0	11.0	15.0	100.0
Montu Colony slum	21	2	49	3	75
	28.0	3.0	65.0	4.0	100.0
Kulibagan slum	70	20	5	0	94
	74.0	21.0	5.0	0.0	100.0
Total	213	59	77	35	384
	55.5	15.2	20.1	9.2	100.0

Source: Household survey, 2008-2009.

6.3 Sharing Pattern of Latrines by Households

Most of the dwellers of study slums were found to use shared latrines. Almost all the twin pit and community latrines are used in shares. In general, twin pit latrines are used by 2 to 3 families and community latrines are used by 15-30 families. Table 3 reveals that in the study slums 42.7% and 31% households used community latrines and twin/single pit latrines in shares. Pit latrines were found to use mostly (by 80% households) in Montu Colony slum.

Table 3: Sharing pattern of latrines by households

Name of slum	Sharing pattern	Total		
	Self (Single pit latrine)	**Shared (Twin & single pit latrine)**	**Community (Community latrine)**	
Rupsha slum	4	34	62	215
	1.9	15.8	28.8	100.0
Montu Colony slum	8	60	32	75
	10.7	80.0	42.7	100.0
Kulibagan slum	5	25	70	94
	5.3	26.6	74.5	100.0
Total	17	119	164	384
	4.4	31.0	42.7	100.0

Source: Household survey, 2008-2009.

6.4 Community Participation in Construction and Renovation of Sanitary Latrines

Reducing burden to poor people, develop sense of ownership, develop sense of prestige and develop responsibility for repair & maintenance of latrines are mainly the reasons in favour of cost sharing for sanitation system development in the slums.

Table 4: Reasons in favour of cost sharing for sanitation system development in the slum

Name of slums	Percentage of households with reasons				Total
	Reducing burden to poor people	**Develop sense of ownership**	**Develop sense of prestige**	**Develop responsibility for repair & maintenance**	
Rupsha slum	129	32	11	43	215
	60.0	15.0	5.0	20.0	100.0
Montu Colony slum	60	3	5	8	75
	80.4	3.6	6.0	10.0	100.0
Kulibagan slum	53	22	12	6	94
	56.9	23.8	12.7	6.5	100.0
Total	243	57	27	57	384
	65.8	14.1	7.9	12.2	100.0

Source: Household survey, 2008-2009.

Table 4 reveals that 65.8% and 14.1% respondents of the study slums have mentioned respectively reducing burden to poor people and developing sense of ownership as the reasons in favour of cost sharing for sanitation system development in the slums.

Twin pit and community latrines required comparatively higher amount to share than the single pit latrines.

Table 5: Cost sharing by slum households for establishment of latrines

Name of slums	Percentage of households with amount of shared cost (In Tk.)			Total
	<500	**500-1,500**	**>1,500**	
Rupsha slum	16	177	22	215
	7.5	82.5	10.0	100.0
Montu Colony slum	55	20	0	75
	73.5	26.5	0.0	100.0
Kulibagan slum	0	53	41	94
	0.0	56.0	44.0	100.0
Total	71	250	63	384
	19.8	64.8	15.4	100.0

Source: Household survey, 2008-2009.

Table 5 reveals that most of the households i.e. 64.8% households shared cost ranges Tk. 500- Tk. 1,500 for establishment and construction of latrines in the study slum. About 73.5% households of Montu colony slum reported sharing less than Tk. 500 for establishing and construction of single pit latrines.

New construction cost of WatSan facilities is found higher than the renovation cost. This is why community people can contribute much more for renovation than new construction.

Table 6: Cost sharing by communities for construction and renovation of WatSan facilities under ASEH during 2007-2008

Sl.	Name of slum	Name and code of latrine	Total cost	Project grant	Community fund	% of Community contribution	Remark
1.	Rupsha slum	Hemayet and Sufir Community Latrine (Nabolok-ASEH-Reno-05 July 07)	12667	10777	1890	14.92%	Renovation
2.		BRAC School Bastee (Saat Bhai) Community Latrine (Nabolok-ASEH-Reno-07 Oct 07)	9644	8150	1494	15.49%	Renovation
3.		BRAC School Bastee (Jamal Goli) Community Latrine (Nabolok-ASEH-Reno-09 Nov 07)	12405	10500	1905	15.36%	Renovation
4.		BRAC School Bastee (Burjuk Khar Goli) Community Latrine (Nabolok-ASEH-Reno- Sep 07)	8484	7029	1455	17.15%	Renovation
Sub-total			43200	36456	6744	15.61%	
1.	Montu Colony slum	Montu Colony Community Latrine (Nabolok-ASEH-067/Feb/08)	75666	71526	4140	5.47%	New construction
2.		Montu Colony Community Latrine (Nabolok-ASEH Mardh/08)	71181	66321	4860	6.83%	New construction
Sub-total			146847	137847	9000	6.13%	
1.	Kulibagan slum	Kulibagan Community Latrine (Nabolok-ASEH-96/Oct'08)	176784	162168	14616	8.27%	New construction
2.		Kulibagan Community Latrine (Nabolok-ASEH-97/Oct'08)	92704	83620	9084	9.80%	New construction
3.		Kulibagan Community Latrine (Nabolok-ASEH-104/Oct'08)	176956	162112	14844	8.39%	New construction
Sub-total			446444	407900	38544	8.63%	
Grand Total			636491	582203	54288	8.53%	

Source: Field Survey, 2008-2009.

Table 6 shows that of the total cost of Tk. 6,36,491 for construction and renovation of 9 (nine) community WatSan facilities in the study slums under ASEH, Nabolok. Community people contributed Tk. 54,288 which is 8.53% of the total cost. Community people of Rupsha slum contributed 15.61% cost for renovation of 4 (four) WatSan facilities, whereas it shows that community people of Kulibagan slum contributed 8.63% cost for new construction of 3 (three) WatSan facilities in Kulibagan slum.

Figure 1: Constructed community latrine in Montu colony slum (2009), where community contribution was Tk. 14,616 and the ASEH grant was Tk. 1,62,168.

Figure 2: Renovated community latrine in Rupsha slum (2009), where community contribution was Tk. 1,455/= and ASEH grant was Tk. 7,029.

6.5 Areas of Consultation with Community for Construction of Latrines

Selection of place, cost sharing & accounts, purchasing of hardware & materials, overseeing construction works, repair & maintenance mechanism of latrines may be the areas of consultation needed with community for establishment of latrines in the slums. Table 7 reveals that 48.8%, 25.7% and 13.7% households respectively of the study slums have mentioned selection of place, cost sharing & accounts and repair & maintenance mechanism of latrines should be the areas of consultation needed with community for establishment of latrines in the slums.

Table 7: Areas of consultation needed with community for establishment of latrine

Name of slums	Percentage of households with areas of consultation					Total
	Selection of place	Cost sharing & accounts	Purchasing of hardware & materials	Overseeing construction works	Repair & maintenance mechanism	
Rupsha slum	103	17	19	32	43	215
	48.0	8.0	9.0	15.0	20.0	100.0
Montu Colony slum	56	10	2	2	6	75
	74.0	13.0	3.0	2.0	8.0	100.0
Kulibagan slum	15	52	12	2	12	94
	15.5	55.8	13.2	2.3	13.2	100.0
Total	173	79	34	36	61	384
	48.8	25.7	8.4	6.4	13.7	100.0

Source: Household survey, 2008-2009.

A significant portion of community people of the study slums was not informed the cost of establishment of latrines. Table 8 reveals that 65.4% households did not know the cost of establishment of latrines. Different sanitation projects of different organizations had different levels of community consultation during establishment of latrines in the slums. The levels of consultation can be categorized as good, moderate and poor. In the project and slum where there had good community consultation, community people had good knowledge on the cost of establishment of latrines. About 86.2% households of Kulibagan slum mentioned that they did not know the cost of establishment of latrines. About 35% households of Rupsha slum mentioned knowing the cost of establishment of latrines. About 59.7% households of Montu Colony slum have mentioned knowing the cost of establishment of latrines.

Table 8: Knowing the cost of establishment of latrines by households

Name of slums	Percentage of households with answers		Total
	Yes	No	
Rupsha slum	75	140	215
	35.0	65.0	100.0
Montu Colony slum	45	30	75
	59.7	40.3	100.0
Kulibagan slum	13	81	94
	13.8	86.2	100.0
Total	133	251	384
	34.6	65.4	100.0

Source: Household survey, 2008-2009.

7. PROBLEMS OF COMMUNITY PARTICIPATION IN CONSTRUCTION AND RENOVATION OF LATRINES

Povery, lack of knowledge on causes, consequences and remedial measures of using unsanitary latrines are the main problems of community participation in construction and renovation of latrines. The slum dwellers have very little technical know how on latrine preparation and installation technology. Besides, there had very little scope to be involved in the supervision activities for preparation and installation of latrines in most of the government projects. They could not bargain with the contractors and raise their voice to the project officers for improving quality of the latrines. Government officials and contractors were mainly involved in installation of sanitary facilities. Community involvement was bare minimum in installation of sanitary latrines, tubewells etc. LPUPAP and ASEH projects had better mechanism and practical interventions of community participation than other projects implemented in Khulna City.

8. RECOMMENDATIONS FOR ACCELERATING COMMUNITY PARTICIPATION IN CONSTRUCTION AND RENOVATION OF LATRINES

- Some community sanitation volunteers in the slums on construction/establishment, repair and maintenance of the latrines can be developed through providing training, orientation and attachment with the concerned organizations so that they can continue regular construction, repair and maintenance of the latrines in a sustainable manner.
- Construction, repair and maintenance committees for the latrines comprising Ward Councillors, GO-NGO representatives, community leaders and civil society can be formed in the slums. Provision of raising construction, repair and maintenance fund and regular meetings on functions of such committees can be considered towards sustainability of sanitation services.
- Optimum and efficient use of project fund for bringing quality of project interventions and sustainability can be ensured through discouraging corruption, irregularities and nepotism.
- Motivational awareness raising campaigns on ethical issues for checking corruption, irregularities and nepotism can be intensively organized for the GO-NGO staff, elected representatives and community leaders.
- Saving or paying a very insignificant amount at regular interval in a community fund can be an option of raising community fund for construction/establishment, repair and maintenance of the latrines. Biogas generation and sell it to nearby poultry farms, shops, households etc. can generate money for development and running of sanitation programs.
- Special steps for showing sincerity and rendering quality services of implementing agencies i.e. display of cost for installation of sanitation hardware in comparison with market value of major components, involvement of community representatives in planning and installation activities etc. can be taken to gain trust of the community to the implementing agencies.
- Provision to be kept under the future projects on supplying bangle version handouts/manuals on quality examination/control of hardwares of sanitation for easy understanding of the general community people or community level group/committee members, so that they can take part in verifying and controlling quality of hardwares of sanitations during the projects towards their sustainability.
- Quality Testing Unit (QTU) with committee comprising members of government officials, academics, public representatives, community leaders and researchers can be set up in KCC or KWASA or DPHE for testing quality of WatSan hardware.

9. CONCLUSION

Cost sharing by the community for construction of latrines has been recognized as an acceptable option both at international, national and local level. Reducing burden to poor people, develop sense of ownership, develop sense of prestige and develop responsibility for establishment, renovation and repair & maintenance of latrines are mainly the reasons in favour of cost sharing for sanitation system development in the slums. Twin pit and community latrines required comparatively higher amount to share than the single pit latrines. Saving or paying a very insignificant amount at regular interval in a community fund can be an option of raising community fund. Training, orientation, planning meeting etc. with community participation for construction and renovation are important for making sanitation system sustainable.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SALT BRICK INTERACTION ALLOWING SALT LADEN WATER SET BENEATH TO FLOW SPATIALLY WITHIN BRICKS

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ABSTRACT

Salt in Brick is a nuissance which can hardly be avoided in construction works. The prescence of salinity in brick results in efflorescence eventually. Salt laden water is transported through the numerous pore spaces of brick. Water gets evaporated whenever it gets available exposure of sunlight, humidiy,air etc. and finally leaves a white crusty matter as salt deposition called efflorescence. The auxiliary process which keeps this salt transport continuous, is termed as The Capillary Action. According to the research conducted; salt can be present in water near to brick made structures or may be present in brick itself since its manufacture. Three kinds of saline water were prepared in laboratory for research purpose using NaCl only. Normally available tapwater, Saline water of 1000 mg/l, & 35000 mg/l were prepared to imitate different practically available saline water. Brick samples (Average size: 9.75 in × 2.25 in × 2.10 in) were taken in different forms (Plastered, Bared & customized hindrance of evaporation) for watching the pattern of saline flow or interaction between salt and brick while flowing within. They were sunk in saline water of different concentrations. After 1 year they were withdrawn from the water containers and sliced teansveresely along its longitudinal direction to figure out level wise NaCl retention as well as to find out any affinity of saline accumulation in any specific zone. Research reveals some interesting information about salt-brick interaction.

Keywords: *Salinity, efflorescence, capillary action, slices, NaCl retention.*

1. INTRODUCTION

Building construction is chiefly dependent on brick in context of its substantial appearance, required material quantity and strength also. No other materials like cement, fine aggregates are required in such a huge quantity if other materials are not used in replacement of brick. The presence of salinity thus demands a detailed study regarding its quantity, nature of propagation through brick i.e. simply the brick-salt interaction. Retrospection shows massive usage of stones to serve the purposes of brick. Later with the advent of brick in construction works, problems were also concomitant. Brick is lighter and easy to handle in contrast of stones. Brick is sufficiently stiff to take the position of stone if any hard or voluminous substance is considered. Brick made its position at the pinnacle of demand due to its workable and uniform shape, construction and hauling economy. Brick is lighter because it is porous. Brick contains numerous pore spaces resulting from escaping of the entrapped air, water, organic particles during burning [Aziz, M. A. (1995)]. These pores permit water to intrude in. Water is not a problem unless it is salty. Salty water also proceeds through the brick pore spaces but leaves salt alone whenever or wherever it gets weather exposure. Brick sometimes contain salt previously. It proceeds further if water comes in contact. The research was conducted in Khulna, a region with salty groundwater in several places and in close proximity to salty watercourses. According to the geographic location of Khulna it houses many confluences of rivers flowing all over the Indian sub-continent (undivided) flowing with highly saline water which finally meets the Bay of Bengal. Most of the Brick manufacturing companies are situated near these watercourses. So presence of salt previously in brick is easily predictable. This case was treated separately. However presence of salt inside or intrusion of salt from outside both are distressing. None of these two kinds of occurrences does some good to brick and plasters overlain. The term Efflorescence [Kumar, S. (2001)] mentioned erstwhile, causes distressed brick and plaster material. Efflorescence [Kumar, S. (2001)] is also detrimental to reinforcements embedded inside concrete. Although no brick wall is reinforced with reinforcing steel, but it takes saline water or salt near concrete through capillary action which may cause intrusion in the concrete. However, only brick and plaster are the concern here. The co-existence of other potential distressing mechanisms such as the acid rain typically present in the industrial area can overlap and mask the effect produced by the salts raised by seawater capillary rise [Collepardi, M. (1989)]. Therefore it is worthy to look the quantity and pattern of salt transport through brick samples under controlled laboratory condition.

2. RESEARCH OBJECTIVES

1. Determinations of bricks' moisture absorption capacity those were set to absorb saline water of different (NaCl) concentrations. This deed assisted in obtaining the volume of pore spaces in bricks.
2. Determination of salinity (Chloride Content) of level wise brick slices after allowing controlled movement of saline solutions through the specific samples
3. Finding the brick salt interaction allowing salt laden water set beneath to flow spatially within bricks

3. METHODOLOGY

At the initiation of the experiment, real case was studied to apprehend the quantity of salinity intrusion inside the pore spaces of brick and plaster. A brick sample was collected from a structure exposed previously to salinity. Plaster overlaid and adjacent to that brick was also collected for experimental purpose. Both the brick and plaster was exposed to weather as for example sunlight, air and humidity. This was done for assessing the general salt retentivity of brick and plaster.

After studying the real happening; laboratory setups were run for watching the salinity (NaCl) movement pattern. Three water solutions were made for imitating three practical cases (35000 mg/L, 1000 mg/L and Normal tap-water). Sodium chloride only was used in purpose of research. It constitutes the maximum part of the seawater salinity (Almost 85-86 percent).

First class brick samples were collected from a locally available source. They were split into two halves of an approximate dimension of 9.75 in × 2.25 in × 2.10 in. One was sunk in the saline water container and another one was kept intact for future use and as a representative of the half that was sunk. Before sinking in the container, they were made free of moisture. A machine driven saw was used to split them into two halves. Some were wrapped around with plasters; some were wrapped around by plastic for light insulation so that no light can intrude in.

Figure 1: Experimental setup at a glance

After a yearlong observation brick samples were withdrawn from the container. Then they were cut to form slices so that level-wise accumulation pattern can be known.

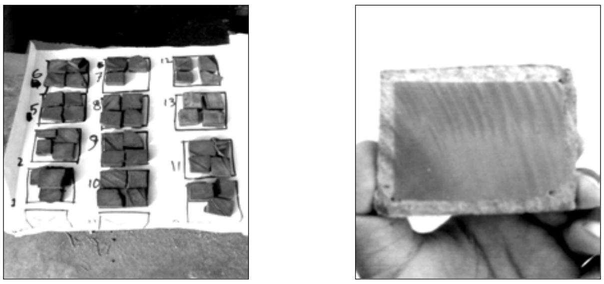


Figure 2: Typical Brick slices for Only Brick (Left) & Plastered Brick samples (Right)

The slices were tested for Chloride content to see whether there exists any accumulation pattern or any affinity to concentrate at any specific level.

The chloride content of the sample is determined by titration with a standard (0.0141N) silver nitrate ($AgNO_3$) solution in presence of K_2CrO_4 indicator. Chloride is precipitated as silver chloride with a reddish end point.

3.1 The Argenometric Method

The chemical reaction is as follows:

$NaCl + AgNO_3 = AgCl\downarrow + NaNO_3$

Calculation for obtaining Cl^- content:

Cl^- as mg/L = [(A-B) × N × 35460] ÷ ml of sample

Where, A = $AgNO_3$ required to titrate sample in ml
B = $AgNO_3$ required to titrate distilled water (0.2 ml)
N = Normality of $AgNO_3$ (0.0141N)

Dilution factor was used if required. A Dilution Factor (*DF*) is

$$DF = \frac{ml\ of\ original\ sample + ml\ of\ water\ added}{ml\ of\ original\ sample} \tag{1}$$

20 ml sample (diluted, if required) was used for titration.

3.2 Grinding of Brick Samples

The brick samples were grinded into smaller crumbs using hammer. They were reduced to sizes varying from 8-10 mm with a small amount of brick dust formed during hammering. This was done for a clear way out of the salt entrapped inside the pore spaces. It was ensured that no brick is lost during grinding. Any loss in brick crumbs could yield an erroneous value of saline content.

Grinded brick samples were then weighted to obtain their mass before sinking. Before grinding; they were dried in oven at about 105°C. Having their mass recorded, they were sunk again in water for 24 hours.

Plasters surrounding the brick of each level were set apart and tested separately. They were grinded also into fines. They were also kept sunken in water for 24 hours to leach out the salt properly.

The brick samples were then filtered for getting the clear water out. This clear water was taken for testing.

3.3 Sample Calculation for Determining NaCl percentage at each level

Sample ID: 1000 mg/l ODF
Level : 0
Cl^- content as mg/l = 1530 mg/l
Volume of Water Added for soaking = 31 ml
Dry mass of Brick Samples of that Level = 30721 mg

So, 1530 mg/l = 1.53 mg/ml
31 ml water contains = 1.53×31
= 47.43 mg

Therefore, 30721 mg Dry sample contained = $\frac{47.43}{30721}$
= 0.00154

Therefore Percentage of Cl^- content = 0.00154×100 %
= 0.154%

The percentage of NaCl content =0.154× $\frac{23+35.5}{35.5}$ = 0.254%

Later having calculated the % Cl^- content and % NaCl content; graphs were prepared depicting % Cl^- content VS Different Levels. This graphs assisted to better comprehend the salt movement and accumulation pattern i.e. the brick salt interaction.

4. RESULTS AND DISCUSSIONS

The results described herein cover the detailed salt accumulation pattern with a brief profile of each. The profiles include their porosity, volume, and amount of water they had absorbed, quantity of salt deposition on surfaces of brick samples. The study was performed totally in southern part of Bangladesh with a typical Bangladeshi weather. The degree of evaporation and subsequent efflorescence are highly susceptible to sun's position in the sky and it is evident that the east and west facing sides of a structure receive more sunlight in sunrise and sunset respectively. The experimental set ups were installed near to the window inside a room at the north-west corner. Northern attachment ensured the wind flowing over the surface. The experimental findings are as follows.

Before starting the experiment, real samples were collected to conceive the properties and real happenings regarding salt and brick. The findings are as follows:

Chloride Content in Bricks: 210 mg/l
Chloride Content in Plaster: 1028 mg/l

This test was run to assess the real case so that the exactitude of the artificial set ups can be evaluated. The samples were collected from an arbitrary location of a building previously exposed to salinity and Cl^- was tested following the procedure mentioned in section 3.1. It reveals that plaster retains a larger share of NaCl than bricks do while flowing through the intestitial pore spaces

4.1 Brick Samples Set to Absorb Salt Water (One Directional Flow)

Three sets of experimental setups were installed with three kinds of saline water having NaCl concentrations 35000 mg/l, 1000 mg/l, & Normal Tap water (Concentration varies between 700-800 mg/l). One set up was installed with full insulation of light and air peripherally so that no evaporation can occur in all other spatial directions except top. This set up is termed as One Directional Flow. Detailed information or brief attributes are presented in the Table-1

Table-1: Detailed Information of Brick Samples

Items	Set To Absorb 1000 mg/L Nacl	Set To Suck 35000 mg/L Nacl	Set To Suck Normal Tap Water
Brick Dimension	9.75”×2.13”×2.36”	9.75”×2.09”×2.35”	9.75”×2.14”×2.19”
Brick Volume ($in^{3)}$	48.88	47.82	45.76
Oven Dried Mass (g)	1193.6	1087.5	1028.8
SSD mass (g)	1477.0	1364.2	1271.8
Moisture Holding (%)	23.78	25.44	23.62
Volume of Void (in^3)	16.81	16.41	14.41
Porosity	0.344	0.343	0.315

Table-1 shows the brick attributes those were set to absorb different concentration of saline (NaCl) water. The test results of percent chloride retentions at each levels are presented in the table Table-2. The % Chloride content is presented as the Dry mass of brick slice of that level.

4.1.1 One directional flow (1000 mg/l NaCl)

Table-2: Test Results for 1000 mg/l ODF (One Directional Flow)

Level	Dry Sample	Initial Reading (ml)	Final Reading (ml)	DF (Dilution Factor)	Cl^- as mg/l	Water Added (ml)	% Cl^- content
0	30.72	19.5	35	4	1530	31	0.154
1	40.12	6.4	9.5	4	290	100	0.072
2	36.63	0	3.2	4	300	100	0.081
3	29.32	9.5	12.1	4	240	100	0.081
4	45.43	12.3	15.2	4	270	100	0.059
5	36.79	15.4	17.8	4	220	100	0.059
6	51.56	3.2	6.4	4	300	100	0.058
7	43.47	4	9.2	4	500	44	0.050
8	44.18	14	19.5	4	530	45	0.053
9	41.33	42.5	48	4	530	42	0.053
10	28.57	37.8	42.5	4	450	29	0.045
11	53.16	35	37.8	4	260	54	0.026

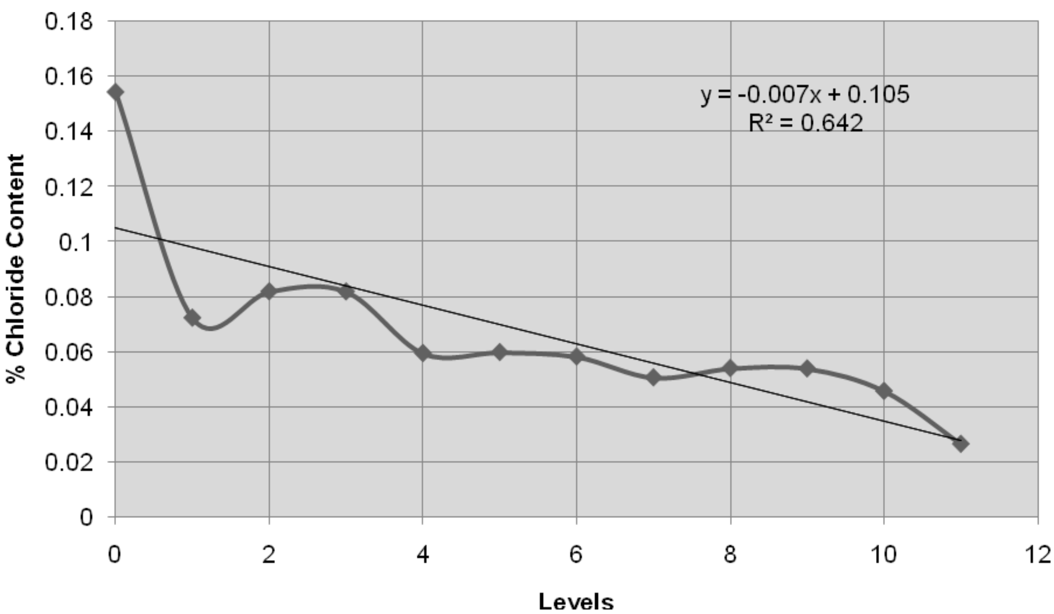


Figure-3: % Chloride Content VS Different Levels for Brick set to absorb 1000 mg/l NaCl in One Directional Flow

The data in Table-2 were found out after slicing off the bricks into different segments. After having the data in hand they were tested for Chloride content according to the section 3.1. The Chloride contents were then expressed in the percentages of the dry mass of the samples taken to soak for leaching their salinity. The soaked samples of each level are the representatives of that level which shows the salt flow pattern as well as salt accumulation at that level.

The plotted curve (Figure-3) depicts a decreasing trend with respect to the height of brick column samples. When the brick came in vicinity of the saline water it started absorbing saline water and after reaching the top surface of brick; subsequent evaporation had occurred. The bottom layer depicts higher accumulation of chloride and it gradually decreases towards upper level. If saline water is considered as a package of salt molecules then it can easily be apprehended that it loses more of its molecules during passing through the preceding layer. As a result the consecutive layer gets smaller share from the package and smaller accumulation occurs there. The surface tension of the denser solution does not allow moving upward so freely. Not only the surface tension, but also the unit mass of the sample does not allow moving freely like a lighter solution does.

A trend line has been found after analyzing the data observed. The trend line shows a decreasing pattern from bottom to the top. Table-2 depicts the level wise salt percentages. Here level 0 means the bottom level which got direct exposure to saline water front.

4.1.2 One directional flow (35000 mg/l NaCl)

The plotting for 35000 mg/l NaCl (Figure-4) also shows a decreasing trend with increasing levels. The supporting reason for the decreasing trend of sample set to suck 1000 mg/l also stands here.

Table-3: Test Results for 35000 mg/l ODF (One Directional Flow)

Level	Dry sample (g)	Initial Reading (ml)	Final Reading (ml)	DF (Dilution Factor)	Cl^- as mg/L	Water Added (ml)	% Cl^- content
0	169	42.3	48.2	100	14250	200	1.686
1	157.91	0	4.5	100	10750	200	1.361
2	185.5	4.2	10	100	14000	200	1.509
3	149.04	10	13.9	100	9250	200	1.241
4	136.83	14.4	16.7	100	5250	200	0.767
5	154.69	17	19	100	4500	200	0.581

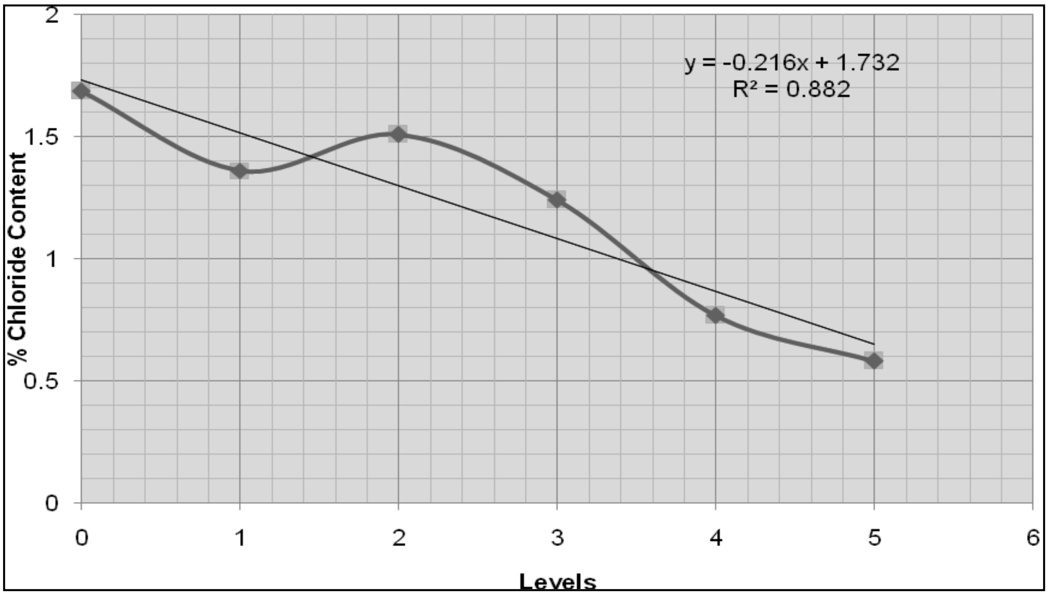


Figure-4: % Chloride Content VS Different Levels for Brick set to absorb 35000 mg/l NaCl in One Directional Flow

The unit mass of the 35000 mg/l NaCl solution is much greater than that of 1000 mg/l solution. The brick samples were taken from the same manufacturing co. So the volumes of the pore spaces were not the main concern here that could influence in the test result i.e. the salt transfer pattern. According to Table-1, it is evident that the pore spaces varied between 31-34% of the total volume the brick samples used for testing. Although the chloride content percentages did not vary with a consistent downfall; the overall trend was in a falling pattern. The percentage values of chloride content fluctuated at some intermediate levels. It is needed to instigate an investigation that the pore spaces inside brick samples are distributed uniformly or not. The concentration of saline water is the main fact here for this variable flow pattern. It will be clear to understand after viewing the result obtained from the test run for normal tap water (ODF). Here is the result and graphical representation.

4.1.3. One directional flow (Normal Tap Water)

Table-4: Test Results for Normal Tap Water ODF (One Directional Flow)

Level	Dry sample (g)	Initial Reading (ml)	Final Reading (ml)	DF (Dilution Factor)	Cl^- as mg/L	Water Added (ml)	% Cl^- content
0	51.05	29.4	32.5	4	290	100	0.057
1	49.93	10.5	13	4	230	100	0.046
2	42.55	26.5	29.4	4	270	100	0.063
3	43.37	18.1	21.4	4	310	100	0.071
4	38	21.4	26.5	4	490	100	0.129
5	41.84	13	18.1	4	490	100	0.117
6	15.71	36	39.1	4	290	100	0.185
7	49.32	0	10.3	4	1010	100	0.205

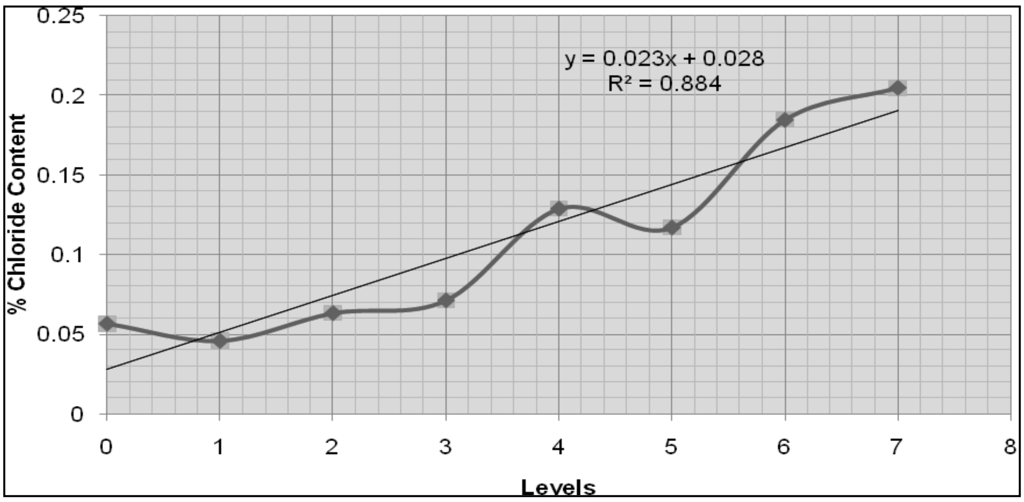
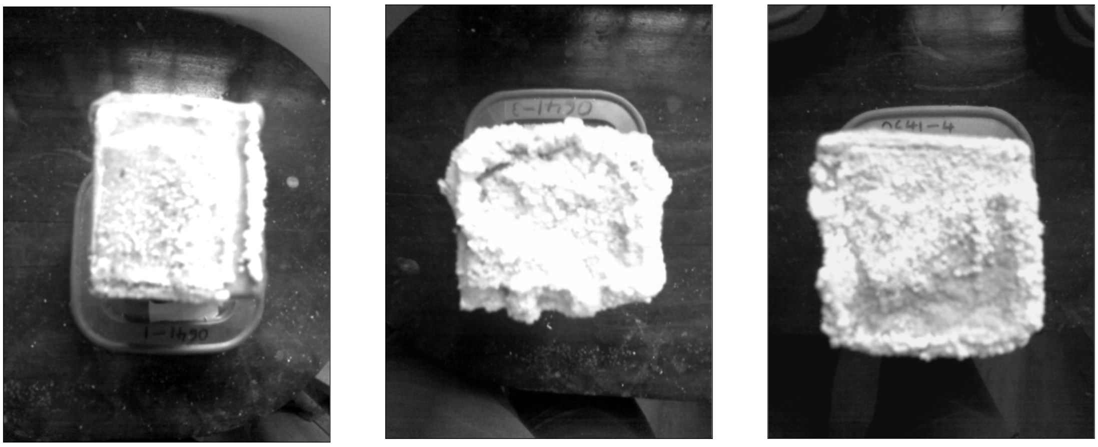


Figure-5: % Chloride content VS Different Levels for Brick Set to Absorb Normal Tap water in One Directional Flow

In case of Normal Tap water; percent chloride contents increase with respect to levels as shown in Figure-5. It shows a totally different and reverse trend than both 35000 mg/l and 1000 mg/l NaCl solution. The concentration of saline water is lesser which results in a lesser unit mass. The surface tension is lesser in case of normal tap water. The lesser the concentration is the faster the movement is. It will also be expressed in the later parts of results and discussion section. Water in this case moves faster and passes away the bottom layers but gets deposited on the top layers as there are no more layers available to propagate further. In case of higher concentrations there occurred some residue which stands as the testimony of the lower sucking rate for higher concentration of water.

4.1.4. The occurrence of efflorescence

Upon suction of salt water and subsequent evaporation; the occurrence of efflorescence was observed as follows (Figure-6). Brick exhibits various kinds of efflorescence phenomenon. The variable concentrations of NaCl is absorbed in different manner which is evident from the photographs shown.



Sunken in 35000 mg/l　　Sunken in 1000 mg/l　　Sunken in Normal tap-water

Figure-6: Salt Deposition as a Consequence of Capillary Action, Evaporation & Efflorescence

4.2 One of Brick Samples Set to Absorb Salt Water (Multi Directional Flow)

Table-5: Test Results for Multi Directional Flow/ Plastered Brick's Brick Portion (35000 mg/l NaCl)

Level	Dry sample (g)	Initial Reading (ml)	Final Reading (ml)	DF (Dilution Factor)	Cl^- as mg/L	Water Added (ml)	% Cl^- content
0	170.10	10.3	18.2	25	4812.5	180	0.509
1	158.73	6.7	10.3	25	2125	180	0.240
2	168.50	1	6.7	25	3437.5	189	0.385
3	175.56	18	27.3	25	5687.5	178	0.576
4	152.91	11.5	18	25	3937.5	200	0.515
5	175.03	0	11.5	25	7062.5	195	0.786

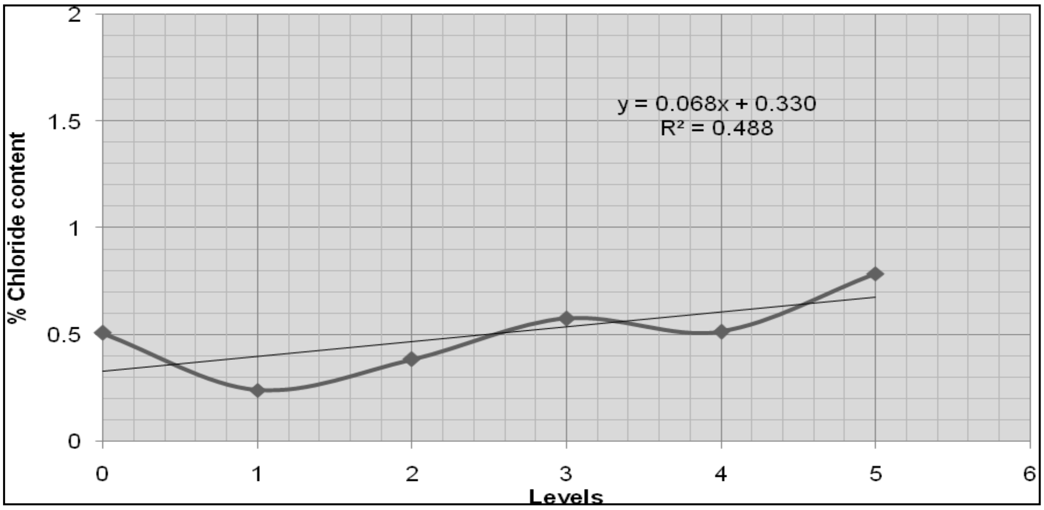


Figure-7: % Chloride content VS Different Levels for Plastered Brick's Brick Portion Set to absorb 35000 mg/l saline water in Multi Directional Flow

Figure 7 & 8 depict completely different trends. Despite representing the consecutive levels of the same brick these two are showing two different patterns. Brick had absorbed saline water in such a pattern that the layers near to saline water front; contained lesser amount of Chloride ion i.e. NaCl deposition had occurred in the minimal percentage. The curve is not in consistently uprising pattern as because the pore spaces in brick is not distributed evenly. The pore spaces form a network of flow path for the salt laden water. In the next figure, (Figure-8) it is evident that the percentage of saline content is in a decreasing trend. The concept of uneven distribution of brick pore spaces is also applicable here. The reason behind showing distinct pattern is that brick and plaster both were exposed to saline water and sucked water simultaneously from the saline water container. Plaster has got a larger volume of pore spaces in comparison to bricks. The highly concentrated saline water moves at a very slower rate. The intruded salt starts moving laterally towards the location o higher porosity to make a place

Table-6:Test Results for Multi Directional Flow/ Plastered Brick's Plaster Portion (35000 mg/l NaCl)

Level	Dry sample (g)	Initial Reading (ml)	Final Reading (ml)	DF (Dilution Factor)	Cl^- as mg/L	Water Added (ml)	% Cl^- content
0	127.61	16.5	52.5	20	17900	100	1.403
1	102.07	0	16	20	7900	100	0.774
2	88.05	30.5	44.6	20	6950	100	0.789
3	106.74	12.5	30.5	20	8900	100	0.834
4	132.20	0.9	12.5	30	8550	100	0.647
5	161.37	0	0.9	20	350	190	0.041

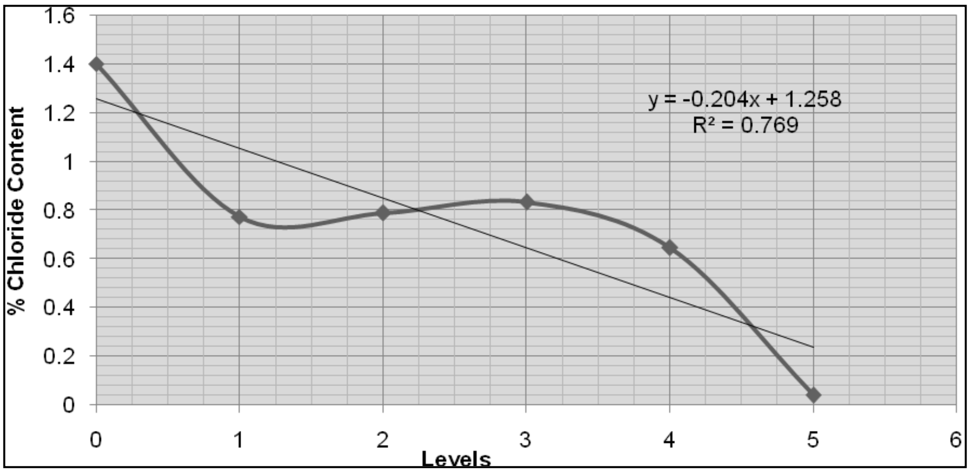


Figure-8: % Chloride content VS Different Levels For Plastered Brick's Plaster Portion Set to Absorb 35000 mg/l NaCl Water in Multi Directional Flow.

of rest which in turn causes the occurrence of a little amount of salt deposition inside brick. The bricks collected from locally available sources were not free from salt also. This will be exemplified in later discussions. The saline water intrusion causes a rush of the previously present salts as well as intruded salts to deposit at upper layers of brick samples which in turns causes blockage of the pore spaces in bricks and retards the salts to move further and deposit subsequently in the plasters in upper levels. It was experimentally testified that highly concentrated saline water moves at a considerably slower speed through brick pores. That is why the salt accumulation or stratification pattern got inverted in bricks and plaster overlaid on to it. The phenomenon can be conceivably explained that salts accumulate to form a pyramid in brick pore spaces and an inverted pyramid in plaster surrounding it.

4.2 Multi Directional Flow Only Brick (1000 mg/l NaCl)

Under multi directional flow; a set up was also run to suck 1000 mg/l saline water to watch how salt laden water moves through the brick-pore spaces. pattern of saline water depicted in Figure-9 is also identical in term of trend as shown in Figure-7. Brick is the only thing in this set up which was in exposure to sunlight & air whereas in case of plastered brick there was an option for brick to push the saline water to larger pore spaces of plaster but in case of brick, brick is the only option for salts to accumulate. It was evident that accumulation had occurred in all five surfaces (bottom surface was exposed to saline water) those were exposed to weather. In Figure-9 it is seen that salt accumulation at bottom layers is very low in comparison to the upper layers. Salt laden water pushes both the salt present in brick previously and the deposited salt. In one directional flow, it was found that the salt particles got the weather exposure at the topmost surface only in one directional flow. So the rush of the saline water movement was towards the topmost surface but in case of multi directional flow (MDF), the samples got their exposure condition from all sides. Therefore, saline water caused the deposition of salt wherever it got the exposure of sun and air and got the opportunity to take much more saline water to the topmost layers. The chloride distribution in one directional flow (ODF) water lost salt particles at bottom layers as the exposed surface is quite further than that of MDF's.

Table-7: Test Results for Multi Directional Flow/ Only Brick (1000 mg/l NaCl)

Level	Dry sample (g)	Initial Reading (ml)	Final Reading (ml)	DF Dilution Factor	Cl^- as mg/L	Water Added (ml)	% Cl^- content
0	172.83	27.4	27.9	20	150	200	0.017
1	155.10	1.3	2.1	20	300	126	0.024
2	190.00	0	1	20	400	200	0.042
3	173.31	31.3	32.1	20	300	200	0.035
4	164.72	2.2	4.5	20	1050	200	0.127
5	149.67	28	36.9	20	4350	200	0.581

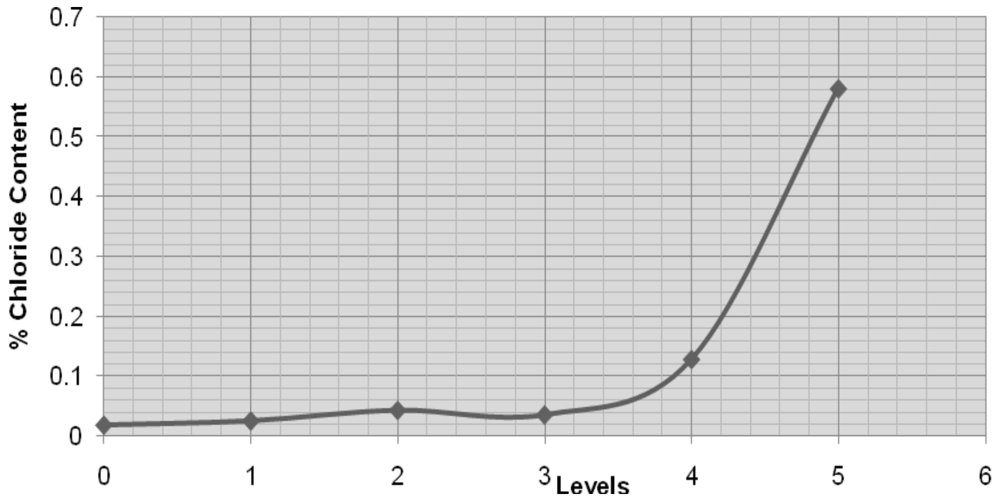


Figure-9: % Chloride Content VS Different Levels for Only Brick Set to Absorb 1000 mg/l NaCl Water in Multi Directional Flow

4.5 One Directional Flow Brick (Distilled Water)

Figure-10: Distressed brick top due to One directional Flow of Distilled Water

In case of Distilled water intrusion; lesser amount of salt accumulated at the lower levels but most of the salts i.e. chloride started rushing toward the top level and got accumulated there. In Figure-10; it is illustrated that how salt has disfigured the brick top. The photograph is from the distilled water with one directional flow. In this brick sample it was evident that the lower layer was less affected by salt where the upper layer was affected a lot. Distilled water contains no chloride. It means the unit mass, surface tension and susceptibility to capillary action is the least among all other water samples used. Therefore the water moved in a faster speed and pushed the salt incorporated in the brick sample further. They dragged the salts from bottom to the top and subsequent efflorescence occurred there. As a consequence the upper layer shows a very higher degree of chloride accumulation and bottom layers shows chloride holding in a lesser quantity. As the water moves faster; the distressing mechanism appears earlier than that of other higher concentrations.

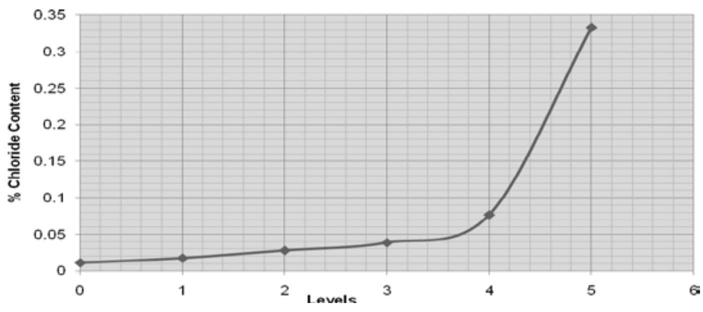


Figure-11: % Chloride Content VS Different Levels for Brick Set to Absorb Distilled Water in One Directional Flow.

5. CONCLUSIONS

Salinity is one of the striking problems in southern part of Bangladesh. Salt intrusion is a detrimental thing to brick made portion of a structure. Salt intrusion from different potentially hazardous saline solution causes distressed bricks from within and efflorescence at the exposed end of brick. Plaster overlaid onto the brick is also highly vulnerable to saline intrusion. Both of bricks and plasters get turned into dusts. This research is a little contribution in context of salinity damage in buildings or industries. Several important findings have been revealed after analyzing the data observed. The flow pattern of saline water varied with the variation of concentrations of saline water as observed in experiments.

An experiment was conducted to determine the exactitude of the performed experiment. Samples from a super-structure previously exposed to salinity were collected and the obtained result showed that Plaster overlaid on the brick surfaces contain higher salt in comparison to brick at that position. The pore spaces in the plaster are higher than that of brick as sand constitutes the main part of plaster.

The porosity of brick samples were found identical with a lesser variation. Porosity of brick samples used varied between 0.344 and 0.315. The void volume of the samples varied between 17 and 14 in^3.

In case of One Directional Flow; three samples were run and it was found that in case of 1000 mg/l NaCl solution, salt accumulation happened in a decreasing pattern from bottom to top. Saline solution losses most of its particles at lower levels leaving a smaller share for the upper layers. Another artificially made solution i.e. 35000 mg/l representing the sea water also illustrated the decreasing pattern. The pattern has been described by a trend line. The curved does not show a definite pattern as the pore space distribution in brick is not uniform. Normal tap-water is low in saline content and moves faster than other highly concentrated solutions. Thus it causes higher accumulation in upper layers. The surface tension and unit mass of saline solution that effect the flow speed in brick pore spaces is also a matter to think about.

In case of Multi Directional Flow samples plaster and bricks of each level were tested separately to watch the flow pattern in plaster and bricks. In case of 35000 mg/l NaCl solution; saline water causes the occurrence of a little amount of salt deposition inside brick sucked water simultaneously from the saline water container. Plaster has got a larger volume of pore spaces in comparison to bricks. The highly concentrated saline water moves at a very slower rate. The intruded salt starts moving laterally towards the location of higher porosity to make a place of rest which in turn causes the occurrence of a little amount of salt deposition inside brick whereas plaster at that level holds a higher amount of salt inside.

The set up for 1000 mg/l NaCl in Multi Directional Flow illustrates a different pattern. It is still to be analyzed for clear understanding but a solution can be set after examining the pattern. The pore spaces in plasters at that level are higher than that of brick and that is why salt started getting deposited inside the plaster at that level. After some days the plaster pores gets saturated with salts and no spaces are left empty. Then saline water starts depositing the salts in the brick pore spaces. As the speed of lighter solution is higher so solution depletes fast. Thus there remains no water left for being sucked. Eventually a pyramid of deposited salt is formed inside the brick pore spaces and an inverted pyramid is formed in the pore spaces of plaster.

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DOUBLE FILTRATION FOR ARSENIC REMOVAL FROM HIGHLY CONTAMINATED GROUNDWATER

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ABSTRACT

Simple iron oxidation filtration unit made of locally available materials was developed and run in highly arsenic contaminated 2 nos. household tubewells in Rupsha upazilla of Khulna district. The raw groundwater contained very high amount of arsenic (average around 400 µg/L). Previous field study of the same filter unit showed that single unit filtration was not appropriate to reduce down to the allowable limit of arsenic for drinking purpose (50 µg/L). Then double filter unit was developed and run in field level for more than 6 months. The influent and effluent samples were collected in every 2 weeks. Not only arsenic, also iron, color, turbidity etc. water quality parameters were then analyzed following standard methods in the Environmental Engineering laboratory of Khulna University of Engineering and Technology (KUET), Khulna, Bangladesh. The removal efficiencies of arsenic, iron, color and turbidity were found around 88%, 100%, 98% and 97% respectively. The flow rate decreased gradually and filter was required to be changed in 6 months due to the clogging on the filter surface. The approximate cost of this filter unit was around 2 to 3 USD only. Therefore, single and double unit filtration can be used according to the level or degree of contamination of arsenic in the groundwater.

Keywords: *Arsenic, groundwater, ceramic filter, oxidation, double filtration.*

1. INTRODUCTION

The presence of arsenic at concentrations above acceptable standards in drinking water is a significant health concern, because prolonged exposure to elevated arsenic concentrations (even at quite low concentrations) has been linked to several types of cancer and non-cancer health hazard. Elevated arsenic concentrations have been detected worldwide in groundwater, with the greatest problems being associated with the high arsenic loads found in large areas of Bangladesh and West Bengal, India. A large amount of affected people has been identified in rural area of Bangladesh which is arsenic related disease ranging form melanosis to skin cancer and gangrene. A recent report maintain that arsenic contaminated tubewells water is contributing to nearly 125,000 cases of skin cancer and killing about 3000 people in Bangladesh each year (Clark, 2003). The mortality rate from arsenic poisoning is expected to rise substantially in the near future as it has a possibility of arsenic contamination in food chain through irrigation water too.

Due to the carcinogenic nature of arsenic, recently EPA as well as WHO revised the maximum concentration limit (MCL) for arsenic in drinking water by decreasing it from 50 to 10 µg l^{-1} (WHO, 1996; EPA, 2002). As a result of this revision, many areas in the world exceeded the new limit of arsenic in drinking water. Moreover, all developing countries affected with contaminated groundwater are still struggling to keep up with the previous WHO guideline value of 50 µg l^{-1}. Chronic exposure to arsenic >50 µg l^{-1} in drinking water can result in serious health problems. Symptoms of chronic exposure include skin, cardiovascular, renal, hematological and respiratory disorders (Marshall et al., 2007; Smith et al., 1998; Mazumdar et al., 1998).

Arsenic (As) contamination of groundwater is major concern on a global scale. Arsenic contaminated groundwater has been found in Argentina, Chili, Mexico, China, Hungary, West Bengal, Bangladesh and Vietnam. Of these regions, West Bengal and Bangladesh are most seriously affected in terms of the size of the population at risk and the magnitude of the health problems. A recent survey of shallow groundwater aquifers in Bangladesh showed that 27% of the aquifers have arsenic concentrations >50 µg l^{-1} (BGS, 1999) and more than 90% of the rural population in Bangladesh gets drinking from 4-5 million tubewells.

One important mechanism through which the groundwater is polluted with arsenic is the oxyhydroxide (FeOOH) reduction of iron by microorganism or in reducing environment and subsequent de-sorption of arsenic from the iron surfaces. In the Bengal Basin (part of Bangladesh and West Bengal), it is the main mechanism by which groundwater become contaminated with arsenic (BGS, 1999; Fazal, 2001; Smedley and Kinniburgh, 2002).

Among the arsenic removal technologies, adsorption and subsequent co-precipitation with iron salts is the simplest and convincible arsenic removal technique. Iron salts occur in two forms, Fe(II) and Fe(III), while removal by Fe(III) salts are more commonly used technology (Katsoyiannis and Zouboulis, 2002; Thirunavukkarasu et al., 2003; Zeng, 2003) . Arsenic removal by Fe(III) salts need pre-oxidation of As(III) to As(V) because As(III) is the most common species in anaerobic ground waters (Harvey et al., 2002) and generally is removed less efficiently than the oxidized As(V) (Dixit and Hering, 2003).

Fe (II) can be oxidized by both physicochemically and biologically but the dominant one is depend on the physical and chemical characteristics of the raw water and process conditions. The biological iron oxidation is caused by the presence of several iron oxidizing microorganisms in water. *Gallionella* sp and *Leptrothrix ochracea* cause primary intercellular oxidation by enzymatic action, while secondary extracellular oxidation is caused by the catalytic action of polymer excreted filaments (Czekalla et al., 1985). A biological process of iron removal has advantages than that of physicochemical process. Mounchet (1992) reported that a biological process could have high filtration rate, high retention capacity, flexibility of operation and reduced the capital cost. On the other hand, the rate of iron oxidation can be increased in the presence of iron oxidizer (Michalakos et al., 1997). Thus the arsenic removal method based on biological iron oxidation would be an ideal option in developing countries such as Bangladesh and India.

With the collaboration program between Ritsumeikan University, Japan and Khulna University of Engineering and Technology, Khulna, Bangladesh, a filter unit is invented. The aim of this filter is to be used in rural areas of Bangladesh to remove or to reduce Arsenic, as well as Iron below allowable value (Arsenic 50 μg/L for Bangladesh). This filter unit was able to reduce 150 to 200 μg/L of influent arsenic concentration to the allowable limit. But, this Single unit system was not appropriate for highly arsenic contaminated (up to 400 to 500 μg/L) groundwater. The main objective of this study was to evaluate the effectiveness of the Double unit filter system in case of this highly arsenic contaminated groundwater treatment.

2. METHODOLOGY

The main components of the filter unit are: Ceramic filter, Iron net/ Scrap iron/ Iron rod, Iron bacteria sludge, Reactor (14 16 L Clay pot was used), Effluent storage bucket, Wooden stand etc.

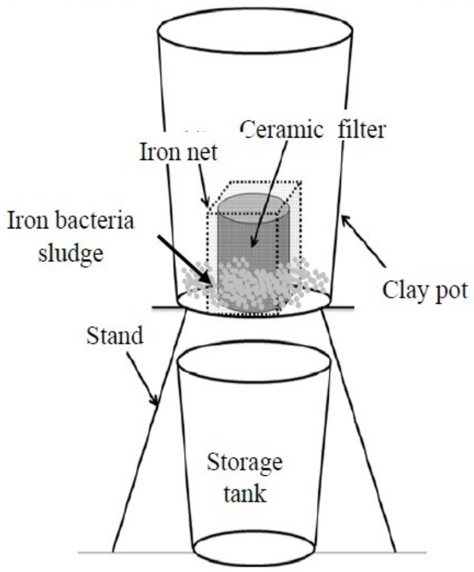

Figure 1: Components of Filter Unit with iron net

2.1 Manufacturing of Ceramic Filter

The filter was made with locally available and cheap materials as rice bran, clay soil and water.

- Oven-dry soil was grind with hammer. Then soil and rice bran was screened through 0.5 mm and 1 mm sieve respectively. Soil (640g for 1 filter) and rice bran (160g for 1 filter) was taken in ratio of 80:20.

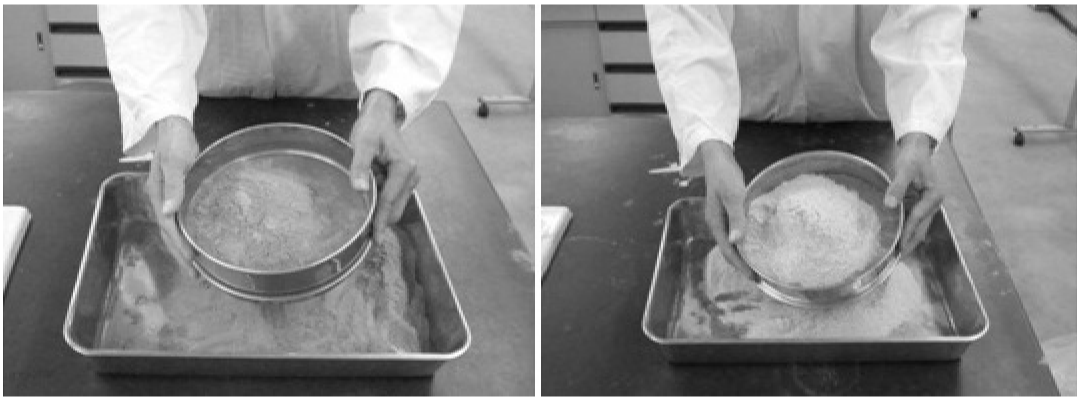

Figure 2: Sieve analysis of soil and rice bran

- Soil and rice bran was mixed homogeneously with water to make dough. Then dough was placed around the bar of the dice and two pieces of PVC pipe were pushed by hand from both sides to make cylindrical shape.

Figure 3: Mixing of soil and rice bran and making of dough with water

Figure 4: Shaping of filter from dough using dice

- Next the pipes were taken off and the surface of the filter was polished with water. The total frame was then toppled down to remove the dice.

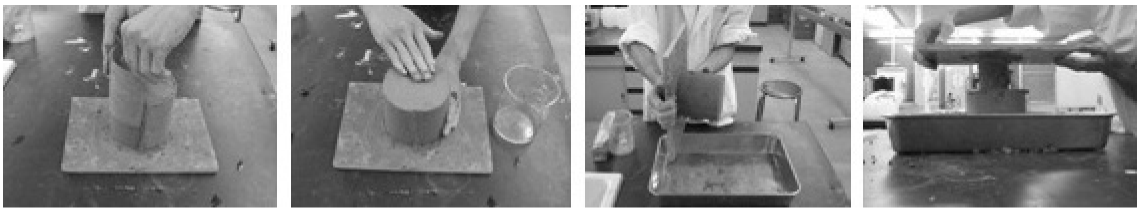

Figure 5: Final step to get raw ceramic filter

- The resulting cylindrical ceramic filters were hollow with one side open. This soft filter was then dried in the sun for at least 3 days.

- The air dried filters were burnt in potter kiln at 900 to 1000℃. After continuous burning for 6 to 8 hours, the kiln was kept to cool down. After some hours the filters were taken out from the kiln. The final ceramic filters had a height of 10 cm and a thickness of 2 cm.

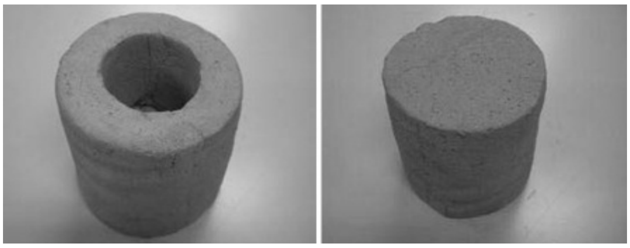

Figure 6: Final ceramic filter before burning

Figure 7: Filter burning in Potter kiln

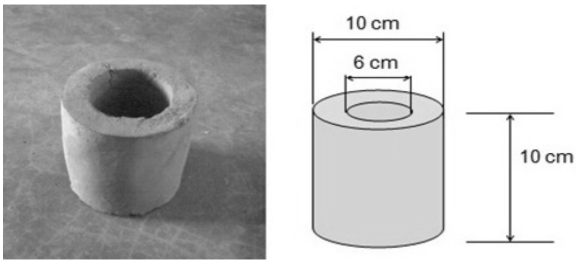

Figure 8: Final ceramic filter after burning

2.2 Preparation of Iron Net

600 gm commercially available iron net without coating was taken and 11cm×11cm×11cm cube with one side open by the iron net was made.

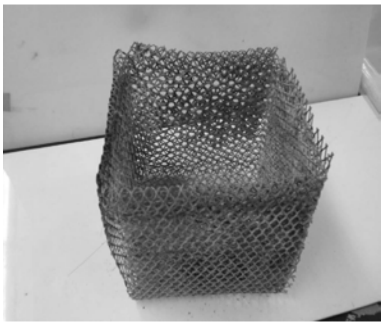

Figure 9: Iron net for the filter unit

2.3 Preparation of Iron Sludge

Water was filled in a big drum of capacity 100 L. Some iron net, iron bar and other iron materials was added in the drum. This was aerated with a stick for 5 minutes daily. Iron bacteria layer will be deposited at the bottom of the drum after 10-15 days.

Figure 10: Iron bacteria culturing and collected iron bacteria sludge

2.4 Double Unit Filter System Set Up

Raw influent was poured into the first reactor and the filtrated effluent was automatically poured into the second reactor and final effluent was found in the storage bucket.

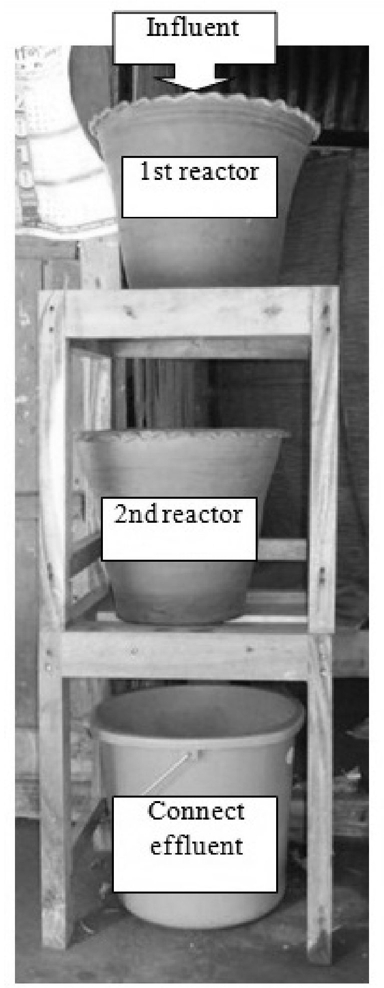

Figure 11: Double unit filter system

2.5 Arsenic Removal Mechanism

This removal of arsenic occurred due to the oxidation of iron and arsenic followed by their subsequent adsorption and precipitation on and with biologically produced iron hydroxides. Biological oxidation of iron by iron bacteria is the main mechanism in respect to the removal of arsenic in this study.

Both forms of inorganic arsenic (AS (III) and As (V)) could be removed more efficiently during iron oxidation than formed iron precipitation. This might be because a very fine iron hydroxide floc is produced which had the high adsorptive surface area and high binding energy resulting in the effective removal of both forms of arsenic at the beginning of biological iron oxidation.

Firstly Fe (II) oxidation is catalyzed by the iron bacteria and transformed to Fe (III). Secondly a part of As (III) is oxidized to As (V) in the presence of Fe (II) and the iron bacteria. Finally adsorption of As (V) on iron hydroxides occurred. These processes are schematically shown in Figure 12.

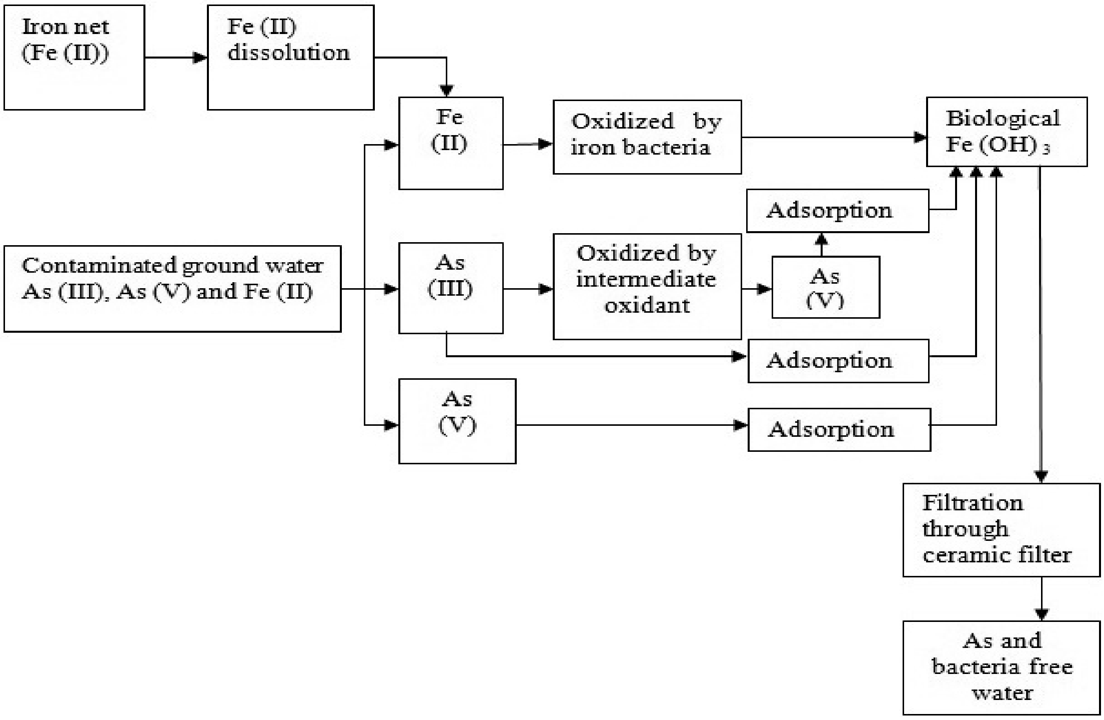

Figure 12: Mechanism of arsenic removal by iron oxidation

3. RESULTS ANG DISCUSSION

2 nos. Double unit filter systems (‘R1double’ and ‘R2double’)were installed and run in 2 different households in Rupsha, Khulna. The influent and effluent samples were collected in every 2 weeks for over 6 months. Not only arsenic, also iron, color, turbidity etc. water quality parameters were analyzed following standard methods in the Environmental Engineering laboratory of Khulna University of Engineering and Technology (KUET), Khulna, Bangladesh.

3.1 Removal Performance of Arsenic

The influent raw water samples contained very high amount of arsenic concentration. The proposed double unit systems were able to reduce the arsenic level to the allowable limit for drinking purpose. The influent and effluent sample characteristics are presented in Table 1.

Table 1: Variation of Arsenic concentration of Influent and Effluent

	‘R1double’		‘R2double’	
	Influent	Final Effluent	Influent	Final Effluent
Average As (µg/L)	418.86	50.10	416.19	52.86
Highest As (µg/L)	465	71	450	75
Lowest As (µg/L)	387	38	355	38

Average arsenic removal efficiency for ‘R1double’ and ‘R2double’ were 88.19% and 87.33% respectively.

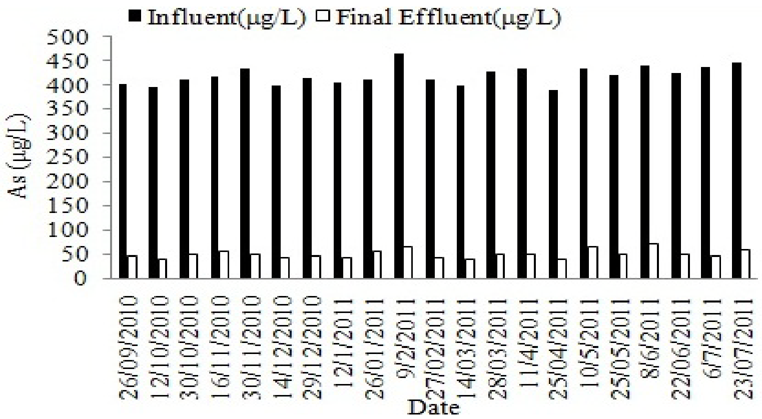

Figure 13: Influent and effluent characteristics of arsenic for 'R1double'

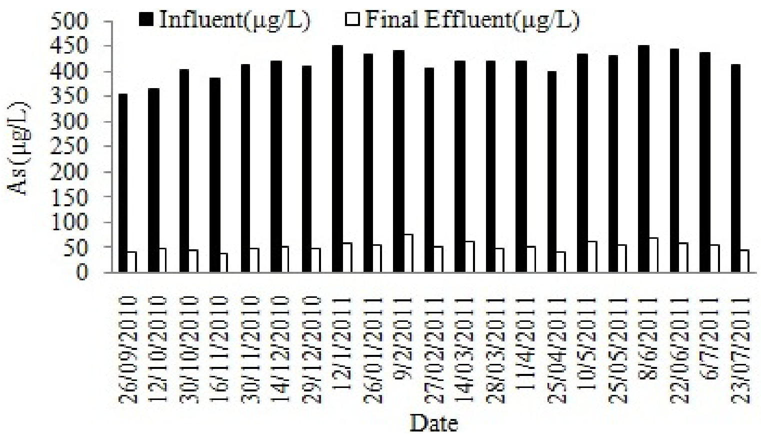

Figure 14: Influent and effluent characteristics of arsenic for 'R2double'

3.2 Removal Performances of Other Parameters

3.2.1 Iron Removal

The influent water contained very high level of iron concentration, Fe (Fe2+). The average values of influent Fe (2+) concentrations were 8.71 mg/L and 8.10 mg/L for 'R1double' and 'R2double' respectively. Fe (2+) was completely removed by the double unit filter system.

3.2.2 Color Removal

Double unit filtration was able to reduce the color level of highly colored influent samples of the household tubewells satisfactorily. Average color removal efficiency for 'R1double' and 'R2double' were 97.40% and 97.77% respectively.

Table 2 Color concentration of influent and effluent samples

	'R1double'		'R2double'	
	Influent	Final Effluent	Influent	Final Effluent
Average Color (Pt-Co)	428.67	11.43	383.38	8.95
Highest Color (Pt-Co)	494	24	453	25
Lowest Color (Pt-Co)	388	3	309	2

3.2.3 Turbidity Removal

Double unit filter units were very much effective for the turbidity reduction in this study. The influent raw water was highly turbid and after double filtration the samples contained very low level of turbidity. Average turbidity removal efficiency for 'R1double' and 'R2double' were 97.30% and 97.50% respectively.

Table 2 Color concentration of influent and effluent samples

	'R1double'		'R2double'	
	Influent	Final Effluent	Influent	Final Effluent
Average Turbidity (NTU)	428.67	11.43	383.38	8.95
Highest Turbidity (NTU)	494	24	453	25
Lowest Turbidity (NTU)	388	3	309	2

3.3 Maintenance of Filter Unit

The maintenance and operation was very simple and easy. To avoid the clogging on the filter surface and to get the filtrated water in desirable flow rate, cleaning of the surface was required in every month. After six months of regular using in this way, the filter core can be replaced by new one and the total filter unit can be started using again.

4. CONCLUSIONS

Basic findings of the study on the double filtration may be pointed out as:

- The treatment unit was cheap and made of locally available materials without using chemicals.
- For the treatment of very highly arsenic contaminated groundwater Double unit filtration may be a good solution as most of the cases it reduced the arsenic concentration to the allowable limit for Bangladesh.
- Iron, color and turbidity removal efficiency was very much satisfactory.
- The operation and maintenance procedure of the filter unit was so much easy and simple.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF SEDIMENT QUALITY BASED ON INPUT MATERIALS INTO SHRIMP POND

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ABSTRACT

In this study, we investigated the total inputs into shrimp pond to understand the effects of these different products (input materials) in pond water and sediment. To accomplish the objectives, a comprehensive field survey has been conducted in 30 major shrimp producing farms in the south-west coastal region of Bangladesh. Compilation of the substances used for different purposes in shrimp farming, and the possible chemical composition of these products were documented. Finally, based on available literatures, the sediment quality in the pond bed was assessed for potential environmental degradation. In this study, we accounted the composition of typical brackish water, soil and water treating disinfectants, antibiotics, pesticides, fertilizers, vitamins and feed additives as input sources. The application rate of agricultural lime (CaO), organic fertilizer, inorganic fertilizers of urea and TSP was found varied in farmer to farmer and the rate of feed additives around 2~5% of fish weights applying 2 times a day. The fertilizers and readymade feed additives are the main source of available nutrients in shrimp pond. The feed industry produces feed additives that contain N (6%), P (1.5%), Protein (26-27) %, Fat (6.5-7.5) % and Calcium (3-3.25) %. The liming compounds, zeolite contains SiO_2 (70%), Al_2O_3 (12.6%), Fe_2O_3 (3.4%), CaO (5.8%), MgO (3.8%), NaO (4.08%), K_2O (0.45%), Mn (0.03%) and P (0.015%). It will provide technical information on adverse environmental impacts of shrimp pond soils. This study deserves an important role towards the sustainability of coastal aquaculture and agricultural sectors in Bangladesh.

Keyword: *Environmental degradation, input materials, sediment quality, shrimp firming*

1. INTRODUCTION

Shrimp, the white gold bar of Bangladesh is the second highest foreign exchange earner which accounts for 5.2% of the total export following the readymade garments (RMG) sector. At present, Bangladesh is the 8th largest shrimp producer of the world and produces 2.5% of the global shrimp output (DoF, 2006; BCAS, 2001). There are two types of shrimp normally cultivating in coastal belt, in which one is Bagda (Paeneus monodon) cultivated mainly near the coast where water salinity is very high and second Golda (Macrobrachium rosenbergi) is cultivated in the interior areas with low water salinity. They used to trap tidal water within the paddy fields during January/February to June/July for aquaculture followed by T (traditional) Aman plantation during monsoon. It seemed a sustainable and environment friendly aquaculture practice which rotated with agriculture. Recently, the coastal shrimp farming industry is being vulnerable situation due to unbalanced feeding, slower growth, mortality and possibly higher sensitivity to disease which have been reported. The environmental scientists have focused its prime cause of water and sediment quality in the pond. Pond sediment is more critical for shrimp than for most other aquaculture species because shrimp spend most of their time near the sediments (Boyd, 1990). The added the application of various chemicals and biological compounds are being used into shrimp pond which directly leads to the development of anoxic conditions at the sediment–water interface. They have been studied on composition of input materials into pond and its detrimental effect on the aquatic environment, adjacent terrestrial ecosystems and health of human. Their findings are: the use of oxidant agent that pose a health hazard for human (Boyd and Massaut, 1999); regarding the health of cultured shrimp, the use of chemicals can increase their stress levels, and thereby decrease growth and/or increase their susceptibility to infections (Reyes et al., 1999); Residues of antibiotics in the environment may affect coastal or marine ecosystems and human health that is one of the main problems of shrimp pond management (Le and Munekage, 2004); antibacterial residues in the sediment can negatively affect the water and soil quality of the shrimp ponds (GESAMP, 1997); many aquaculture chemicals persist for very short time periods for example, formalin has a half-life in water of 36 hrs (GESAMP, 1997) and the antibacterial furazolidone has a half-life of 18 hrs in marine sediment at 4^0C (Samuelsen et al., 1991); several antibiotics (oxytetracycline, oxolinic acid and flumequine) have been found in sediments 6 months after treatment (GESAMP, 1997); fertilizer and feed additives are the main sources of nutrient in shrimp ponds as results in the eutrophication problem (Sansanayuth et al., 1996) that is often mentioned as an important negative impact of shrimp farming on the environment (Hopkins et al., 1995); Pesticides and other products used in shrimp farming for their toxic properties can pose a risk to wild flora and fauna, but also to the health of the shrimp cultures (Baticados et al., 1986); the use of synthetic organic pesticides (formalin, malachite green, and the copper compounds etc) that may persist or leave toxic residues in the pond environment, and be released with pond water or sediment to nearby ecosystems (Graslund and Bengtsson, 2001).

Shrimp farming in Bangladesh has been expanding since the early eighteens and reached an industrial scale followed by increasing demand for shrimp in the export market. Despite rapid explosion of shrimp farming in Bangladesh for the last two decades, water and sediment qualities and their implications in shrimp production as well as the impacts of farm effluent on the receiving ecosystem remains poorly understood. It is well known that our farmers always like extensive than intensive in production because the shortage of their knowledge on farming method. In this transition period of Bangladesh shrimp farming industry, we should have renewed of documentation of input source from commercially available products that affects the sediment quality and consequently aquaculture environment.
The specific objectives of this study is prepare a documents of the substances used for different purposes in shrimp farming, and the proportion of chemical composition of these products. Secondly, assess the potential environmental degradation of sediment quality in the pond.

2. MATERIALS AND METHODS

A questionnaire survey has been carried out recently in 30 shrimp producing farms in the south-west coastal district such as Khulna, Satkhira and Bagerhat of Bangladesh (figure 1).

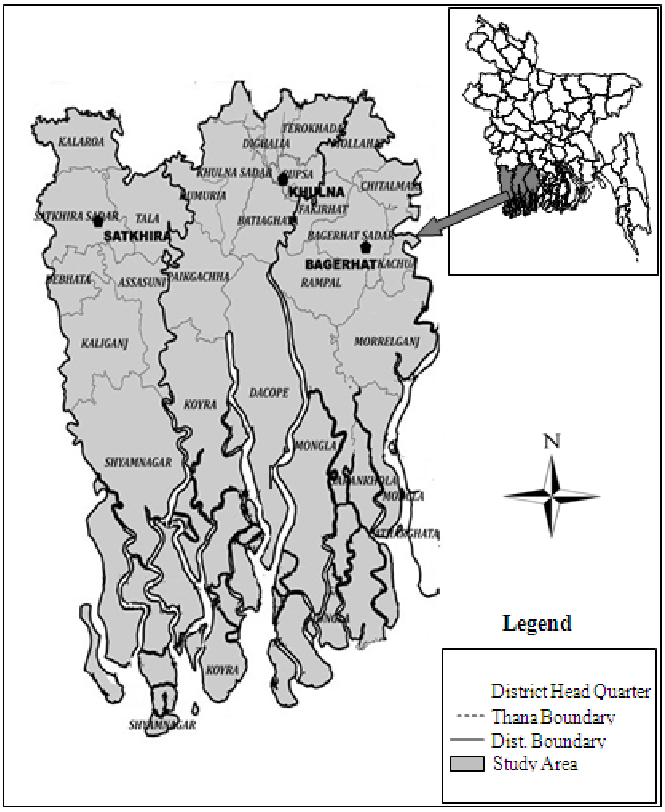

Figure 1: Location of study area

Farms were selected as randomly as possible within each area. The survey was accomplished based on interview method, asking questions to owner of farms or responsible manager of the farms. The questions were open-ended and followed up when necessary. The survey has covered with general information of farms; on the method of pond management practice; at various stages, the use of multifarious materials or compounds into their pond with a focus of chemicals and biological products. The chemicals and biological products documented in the survey were arranged into the following major groups: fertilizers, pesticides and disinfectants, antibiotics, microorganisms, vitamins, soil and water treatment compounds, and feed additives. After compilation of various compounds, an attempt was taken to assess their impacts on sediment quality in pond environment based on available scientific technical reports.

3. RESULT AND DISCUSSION

3.1 Proportion Use of Chemicals and Biological Products

More than 43 different substances or types of products were identified in the present study (Table 1, 3 & 4). Farmers are using these products in different purpose and phases during a cycle of shrimp production. The most frequently used groups of products were soil and water treatment compounds, fertilizers, pesticides and disinfectants, antibiotics, feed additives and vitamins (which is normally included in feed additives in commercially products). The percentages of farmers that make use of the different products are showed in Fig.1

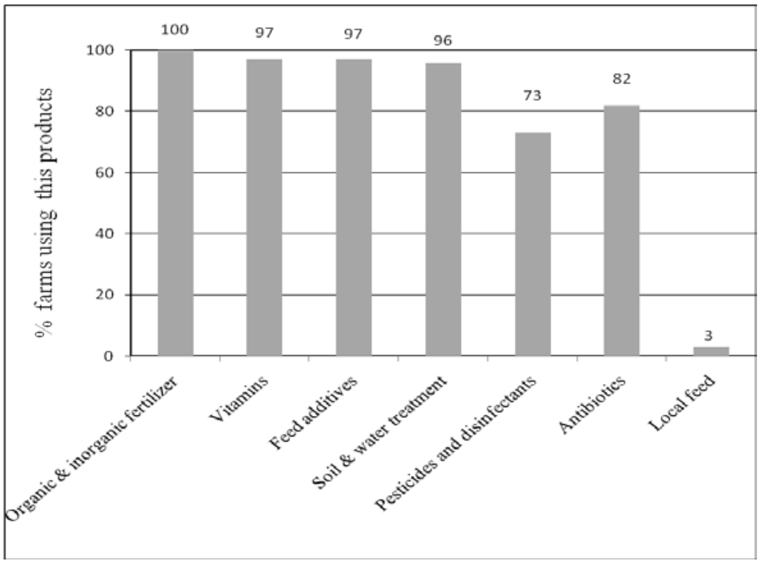

Figure 2 Proportion of farms in study area those using one or several chemical/biological products

3.2 Soil and water treatment compounds

The most commonly used substances were liming compounds, which were used by all without 4% of the farmers (fig.2). A majority of these farmers used them to adjust pH, but at least one third additionally used them as disinfectants to kill pathogens or plankton. Shrimp farms are often located in acid sulfate soil areas, e.g. former mangrove areas, where liming may be of particular importance (GESAMP, 1997). Zeolite products, i.e. hydrated alkali–aluminum silicates, were also used by a majority (70%) of the farmers. Most of the farmers used Zeolite during pond preparation to purifying water, improved water quality, increase dissolved oxygen, helps in the production of algae, reduces shrimp death rate and removes offensive odor from water, whereas only a few used them to remove ammonia. Removal of ammonia is previously known as one of the main purposes of Zeolite use in shrimp ponds, a practice that has been questioned (Boyd, 1995). In this study, there are three types of liming compounds has documented in (table- 1) with molecular formula and application rate during pond preparation.

Table 1: List of liming and fertilizer products used in shrimp farms

Use	Group	Product	Molecular formula	Dosage (kg/ha)
Soil and Water Treatment	Liming compounds	Calcium Carbonate	$CaCO_3$	75-100
		Dolomite	$CaMg(CO_3)_2$	30-35
		Zeolite	$Na_2Al_2Si_3O_{10}\cdot 2H_2O$	70-80
Fertilizers	Inorganic fertilizers	Urea	NH_2-CO-NH_2	50-100
		Triple superphosphate (TSP)	$Ca(H_2PO_4)_2$	20-50
		Oil cake	-	4-4.5
		Di-ammonium Phosphate	$(NH_4)_2HPO_4$	25-50
	Organic fertilizers	Cow dung	-	1000-1200
	Mixed fertilizer	Compost	-	750-1000

Table 2: Chemical Composition of Liming compounds (Zeolite) available in commercially

Product Name	**Composition**	**Fraction in (%)**
Zeolite	SiO_2	70
	Al_2O_3	12.6
	Fe_2O_3	3.4
	CaO	5.8
	MgO	3.8
	NaO	4.08
	K_2O	0.45
	Mn	0.03
	P	0.015

3.3 Fertilizers

Fertilizers are added to ponds to increase nutrient concentrations, stimulate phytoplankton growth and to increase food supply to the cultured shrimp (Boyd and Tucker, 1998). Thirty of the interviewed farmers (100%) used organic or inorganic fertilizer in the pond management (fig.2). In this survey, the most commonly used fertilizing products were NPK-fertilizer (inorganic group) and cow dung, chicken manure/ Poultry litter in organic group (table-1). At least 11 farmers out of 30 surveyed farms used both inorganic and organic fertilizers. Apparently, even though most of the farmers in the survey used pellet feed in the farm management, many farmers additionally chose to use fertilizers to provide natural feed as a complement.

3.4 Pesticides and disinfectants

During grow-out operation, disease is an important problem in shrimp farms of Bangladesh. During this operation period, the most farmers (73%) used at least one kind of pesticide or disinfectant to get protection from disease outbreak in the shrimp pond management (fig.2). As pretreatment the chlorine compound was used by 80% of the farmers and the other infrequently used products were formalin group, copper compounds and potassium permanganate (table-3). These compounds were often used for several purposes, for example, to kill algae, unwanted bacteria, fungi and protozoa and also to clear water. As mentioned in table, liming compounds were also frequently used with the purpose of disinfection and Tea seed was the most commonly used organic pesticide.

Table 3: Different chemicals, antibiotics and drugs used in shrimp farms

Disinfectant/Antibiotics/Chemical Substance	Molecular Formula	Dosage
Lime	$CaCO_3$	80-100kg/ha
Dolomite	$CaMg(CO_3)_2$	30-35 kg/ha
Sodium Hypochlorite	NaClO	5-10 ppm
Tea seed cake	-	4-4.5 kg/ha

Formaldehyde	CH_2O	15-25 ppm
Malachite green	$C_6H_5C(C_6H_4N(CH_3)_2)_2]Cl$	0.75-3 ppm
Copper sulphate	$CuSO_4$	2-3 ppm
Methylene blue	$C_{16}H_{18}N_3SCl$	8-10 ppm
Potassium Permanganate	$KMnO_4$	2.5-5 ppm
Mahua oil and tea-seed cake	-	2-10 kg/ha
Malathion	$C_{10}H_{19}O_6PS_2$	0.5-2 ppm
Ethylenediamine tetra acidic acid	$C_{10}H_{16}N_2O_8$	5-10 ppm
Antibiotics	Molecular Formula	Dosage
Oxytetracyline	$C_{22}H_{24}N_2O_9$	8-10 ppm
Erythromycin	$C_{37}H_{67}NO_{13}$	2-3 gm/kg feed
Furazolidone	$C_8H_7N_3O_5$	10-15 ppm
Sulphamethaxazole (Antibacterials)	$C_{10}H_{11}N_3O_3S$	0.5-2 ppm

3.5 Antibiotics & Antibacterial

Antibiotics were used to prevent and to treat Vibrio infections, but also in the belief that they could prevent and treat viral infections such as white spot disease (Holmstrom et al., 2003). The antibiotics were generally distributed to the shrimps mixed with the feed. According survey outline there is around (82%) of farms used one or several types of antibiotics (fig.1). In case of both pre and post treatment, they use antibiotics like oxytetracycline, erythromycin, furazolidone, and antibacterial like sulphamethaoxazole with chemicals be fond of malachite green, copper sulphate, formalin, to control common shrimp diseases (table 3).

3.6 Vitamin and feed additives

A majority of the interviewed farmers (97%) used vitamins (fig.2) to mix with the shrimp feed to increase the nutrient content for enhance growth, and prevent disease outbreaks. In this study, we documented a total of 14 commercial products of different fish feed company with various ingredients shown in table 4.

Table 4: List of Feed additives used in shrimp farms

Product	Compositions	Fraction (%)	Dosages
Commercially manufactured Feed	Humidity	11 - 11.5	2~5% of fish weights applying 2 times a day
	Protein	26 - 27	
	Fat	6.5 - 7.5	
	Calcium	3.0 - 3.25	
	Phosphorus	1.3 - 1.5	
	Nitrogen	5.0 – 6.0	
	Vitamin	optimum	
	Mineral	optimum	
Other feed additives	Soybean meal		Vary from farmer to farmer
	Fish meal		
	Seed meal		
	Di-calcium phosphate		
	Lime stone		
	Rice bran		
	Oil cake		
	Snail meat		
	Corn cake		
	Cooked rice		
	Poultry feed		

3.7 Chemical Composition of Sea Water

Sea water at 3.5% salinity (ppm)

Table 5: Typical composition of sea water (Source:http://mistupid.com/chemistry/seawatercomp.htm)

Element	Percentage
Oxygen	86.0341%
Hydrogen	10.7177%
Chlorine	1.8902%
Sodium	1.0523%
Magnesium	0.1257%
Sulfur	0.0881%
Calcium	0.0400%
Potassium	0.0382%

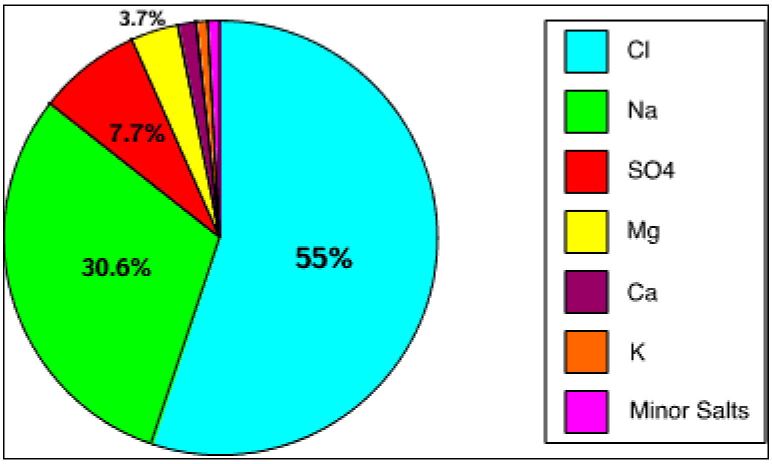

Figure 3: Relative proportions of dissolved salts in seawater
(source:http://www.physicalgeography.net/fundamentals/8p.html)

3.8 Assessment of sediment quality

The absence of exact quantitative information, it is very difficult to assess the sediment quality of the shrimp pond. However, with the surveyed information on different substances using into shrimp pond and previous technical research paper, we can discuss the fate of sediment.

The sources of Sulfur (S) in the pond soils derived from the organic matter (shrimp food and excreta) and seawater (Prabnarong et al., 1994). It is generally known that S in soils is taken by shrimp and released as SO_2 under aerobic conditions during the summer season, and released as H_2S under anaerobic conditions during the rainy season (Tisdale et al., 1985). Phosphorus is one of the main components of shrimp food as milled fish. The amount of nutrient (N, P) depends on the application of fertilizer and feed additives. The pond soil pH is governed by 3 factors (1) soil parent materials (2) the amounts of major elements and (3) organic matter present in the soils. The pH is rising with increasing in basic elements (Ca, Mg, K and Na) and a decline in soil pH would relate to the amount of organic matter and soil microorganisms' processes. This nitrification process was expected to have taken place widely in the pond soils under aerobic conditions, since the pH range of the pond soils, approximately 7.6–8.4 was optimum for nitrifying bacteria (Tisdale et al., 1985). The acidification can also be observed in the soils with frequent application of ammoniacal and most organic nitrogen fertilizers (Tisdale et al., 1985). Furthermore the end products of organic matter decomposition are organic acids (fulvic and humic acids) resulting in lowering soil pH as well. According to a comparison study of chemical characteristics between soils collected from groups of active and abandoned shrimp ponds in Samutprakarn and Samutsakorn Provinces (Central Thailand) conducted by (Chainark, 1997), Ca, K and Na accumulated while organic matter depleted. Seawater elements or salts (table 5) accumulated in the pond soils due to introduced seawater into the ponds.

4 CONCLUSION

Shrimp pond soil always exhibits high salt content, organic and inorganic compounds, chemicals and various humic substances; because the saline soils in the ponds have no potential for purifying or reclamation themselves by leaching salt away with rain water since most of the rain water is trapped within the ponds due to the slow leakage and seepage nature of the clayey texture of pond soils as well as the shallow water table (1.50 m below ground level). Some of these compounds accumulated in the sediment and sometimes it can harmful to shrimp culture productivity and sustainability.

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GREYWATER AS A RESOURCE AND ITS RECYCLING: BANGLADESH PERSPECTIVE

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ABSTRACT

Greywater is a household wastewater which has not been contaminated by toilet wastes. It includes wastewater from bathtubs, sinks, laundry tubs, showers, bathroom washbasins, clothes washing machines. The principal focus of this study is to investigate the characteristics of greywater as a resource and to develop a simple low-cost greywater treatment system which satisfies the normal water standards for flushing toilet, gardening, cultivation, car washing and laundry. To achieve these aims a simple treatment unit consisting of roughing filtration using brick chips followed by sand filtration was developed. The influent and effluent water quality parameters were monitored to determine the performance of the developed greywater treatment system for the potential recycling of water for selected reuse purposes. Using this simple greywater treatment process, the removal efficiency of organic matters, color, sulphate and nitrate were found to be around 60%, 83%, 40% and 96%, respectively. The microbiological water quality in terms of total coliform (TC) and E. coli was also improved satisfactorily with average removal efficiency about 60%. The treated water was very clear with turbidity value roughly 3 NTU. The hardness and total solid removal efficiency were noticed to be 41% and 82%, respectively. Based on the above findings, it can be concluded that the recycling and reuse of treated greywater would have promising application in selected household and agricultural uses after simple treatment without any hazard to human health and environment.

Keywords: *Greywater, low-cost treatment, recycling and reuse.*

1. INTRODUCTION

The world's population is projected to increase dramatically between now and the near future and with this growth will come an amplified need for water to meet various needs, as well as an increased production of wastewater. Many communities throughout the world are impending, or have already reached, the limits of their available water supplies. Bangladesh is a south Asian country having more than 150 million people. During wet season water is available in almost everywhere. But during the dry season water is scarce in most of the areas in Bangladesh. Sometimes cultivation is hampered due to scarcity of water. Moreover the groundwater table is undergoing day by day due to very frequent use of groundwater. That is why water reclamation and reuse have almost become necessary for conserving and extending available water supplies. Water demand management is an approach to get the most from limited supplies by using water efficiently and promoting conservation as a response to scarcity as opposed to looking for new sources of supply from already overtaxed water resources. Among the avenues being pursued is the use of wastewater to counterbalance the demand for fresh supplies. Reusing wastewater brings with it risks largely related with the high levels of nutrients and the existence of pathogens (Ghawi & Kris, 2009). These risks can be mitigated by sorting out waste water into its black water and greywater components. There are significant distinctions between greywater and toilet water which is also called "blackwater". This distinctions give us knowledge of how and why these water should be treated or managed, for the purpose of reuse, public health and environment protection, these water should not be mixed together. It has been estimated that water savings in the range of 18%-29% for an average household could be achieved by reusing greywater (Lechte, 1992). Greywater represents a precious renewable resource that can permit significant household water reserves and reduces demand for potable water. Reusing greywater also reduces discharge to the sewerage system, which can direct to community cost savings through reduced stress on sewerage treatment systems and infrastructure. Grey water consists of wastewater from showers, bathtubs, spas, hand basins, washing machines, laundry troughs, Washing nappies or kitchen sinks wastewater is also a possible source of greywater but because it can be highly contaminated with food particles, oils, and grease and since it only accounts 5% of the average household consumption, it's participation in greywater quantity is almost negligible and not recommended. Toilet wastewater, termed "blackwater" must be chemically treated before reuse (Stanko, 2008 & Stanko, 2009). However, there are important distinctions between grey water and black water. Grey water contains about 70% of COD and 10% of the nitrogen contained in black water which, as nitrate and nitrite, is the most serious pollutant affecting potable water, and the most difficult to remove.

Similarly, black water is the most significant source of human pathogens-those organisms that threaten human health but which do not grow outside the body unless incubated or hosted in human excreta (Ghawi & Kris, 2009). Bacteria, foam, food particles, high pH, hot water, odor, oil and grease, organic matter, oxygen demand, salinity, soaps, suspended solids, turbidity, bleach, phosphate, nitrate and nitrite, chlorine etc. may present in greywater. The common characteristics of some selected greywater are shown in table 1.

Table1: Water-quality characteristics of selected domestic wastewater.

Water Source	Characteristics
Automatic Clothes Washer	Bleach, Foam, High pH, Hot water, Nitrate, Oil and Grease, Oxygen demand, Phosphate, Salinity, Soaps, Sodium, Suspended solids, and Turbidity
Automatic Dish Washer	Bacteria, Foam, Food particles, High pH, Hot water, Odor, Oil and grease, Organic matter, Oxygen demand, Salinity, Soaps, Suspended solids, and Turbidity
Bath tub and shower	Bacteria, Hair, Hot water, Odor, Oil and grease, Oxygen demand, Soaps, Suspended solids, and Turbidity
Evaporative Cooler	Salinity
Sinks, including kitchen	Bacteria, Food particles, Hot water, Odor, Oil and grease, Organic matter, Oxygen demand, Soaps, Suspended solids, and Turbidity
Swimming Pool	Chlorine, and Salinity

The characteristics of greywater differ from source to source. The factors affecting greywater quality are greywater source, type of product use, composition of family (i.e. age distribution of members), individual lifestyle, specific house characteristics such as type of water appliance used. Greywater systems can be classified in two broad groups- (I) primary greywater systems and (II) secondary greywater systems. Primary greywater systems are systems directly reusing virtually untreated domestic greywater from a single family residence for sub-surface territory and/or garden watering. These systems do not permit storage or treatment, apart from some surge storage and coarse screening/filtration which removes hair, lint and coarse particles. Secondary greywater systems permit greywater to be treated and stored for toilet/urinal flushing and/or lawn and garden watering (including surface watering methods). Secondary greywater systems may be used for multiple residence buildings. Greywater systems for irrigation, even with a higher automation level, can fit in to the primary greywater systems group if there is no treatment and storage allowed. In distinction, even the simplest greywater systems for toilet flushing have to let for some treatment and storage of greywater; therefore they belong to the secondary greywater systems group (Christov *et al*., 1995). The main objectives of this study are to investigate the characteristics of greywater as a resource and to develop a simple low-cost greywater treatment system which satisfies the normal water standards for flushing toilet, gardening, cultivation and laundry.

2. METHODOLOGY

In this study, a simple treatment unit was developed for the reclamation of greywater for its potential reuse in selected household purposes. In the treatment process, the water samples from different stages were collected and tested to judge whether the reclaimed water is suitable for selected reuse purposes or not. The major treatment operation was consisted of roughing filtration and sand filtration. Finally, based on this study, a household greywater reclamation and reuse model was proposed.

2.1 Development of Simple Treatment Unit:

For the purpose of the development of a low cost greywater treatment unit a simple filtration unit was developed. The schematic diagram of the filtration unit is shown in figure 1. The filtration unit consists of three major components. These components are storage tank, roughing filter, and sand filter.

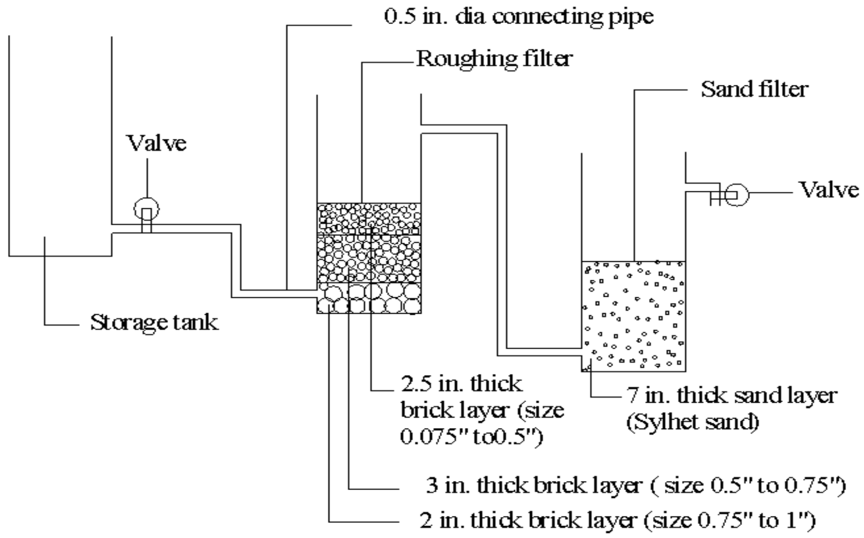


Figure 1: Schematic diagram of developed greywater treatment unit

Roughing filter consists of brick khoa in three different layers of three different sizes. At the bottom of the roughing filter 2 in. thick brick khoa layer of size 0.75 in. to 1.0 in. was placed. At the middle of the roughing filter 3 in. thick brick khoa of size 0.5 in. to 0.75 in. was placed, and at the top of the roughing filter 2.5 in. thick brick khoa layer of size 0.075 in. to 0.5 in. was placed. So the total thickness of the filter material in the roughing filter was become 7.5 inches. The sand filter consists of sylhet sand as a filter material. The thickness of the sand layer was 7 inches. The thickness of the filter material in the filter is shown in the table 2. The storage tank, roughing filter, and sand filter were connected by 0.5 in. dia connecting pipe. At the outlet of the storage tank flow regulator was set to control the flow of the water.

Table 2: Thickness of aggregate layers

Aggregates	Aggregate size	Thickness of layers
Brick (roughing filter)	1 in. to 0.75 in.	2 in.
	0.75 in. to 0.5 in.	3 in.
	0.50 in. to 0.075 in.	2.5 in.
Sand (sand filter)	-	7 in.

The flow rate was 8.33×10^{-5} m^3/sec and the flow velocity was 0.2 cm/sec. Up flow water collection system was applied. No power is used for the flow generation. Gravity flow pattern was used in this system.

3. RESULTS AND DISCUSSION:

The performance of the developed greywater treatment unit was studied in the laboratory with regards to different water quality parameters (e.g., pH, Color, Turbidity, Hardness, Total Solids, BOD_5, COD, Cl^-, NO_3, SO_4, Total coliform, E. coli, etc.). Influent and effluent water (roughing filtered and sand filter followed by roughing filter) quality were tested. Than the obtained water quality was compared with the WHO wastewater irrigation standard (Table 3) and feasibility study of toilet flushing, gardening, agricultural use, car wash and laundry use were conducted. The chemical constituents in reclaimed water of concern for agricultural irrigation are salinity, sodium, trace elements, excessive chlorine residual, and nutrients. Sensitivity is generally a function of a given plant's tolerance to constituents encountered in the root zone or deposited on the plants. Reclaimed water tends to have higher concentrations of these constituents than the groundwater or surface water sources from which the water supply is drawn.

Table 3: Performance of the developed greywater treatment unit

Parameters	Units	Influent water quality	Effluent water quality		Waste water irrigation standard *
			Roughing filtration	Sand filtration followed by roughing filter	
pH	-	8.60	8.69	8.20	8.4
Total coliform	No./100ml	24	22	10	1000
E. coli	No./100ml	16	15	7	200
BOD_5	mg/l	62	56	30	110
COD	mg/l	112	96	48	150
Color	Pt. Co. unit	497	91	87	-
Turbidity	NTU	9.99	5.49	3.25	-
Conductivity	µS/cm	2.92	4.08	3.14	6350
TDS	mg/l	1150	956	385	500
TSS	mg/l	1967	1356	198	100
chloride	mg/l	247	237	219	2000
Alkalinity	mg/l	260	255	165	200
Hardness	mg/l	315	425	185	200
Nitrate	mg/l	4.4	0.3	0.2	-
Sulphate	mg/l	196	193	120	250

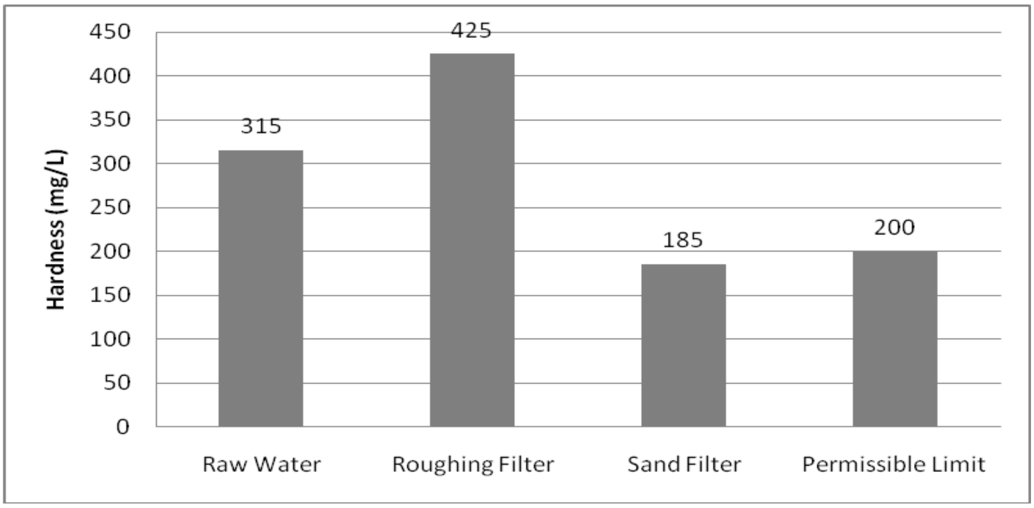
* WHO standard for wastewater irrigation.

3.1 Salinity:

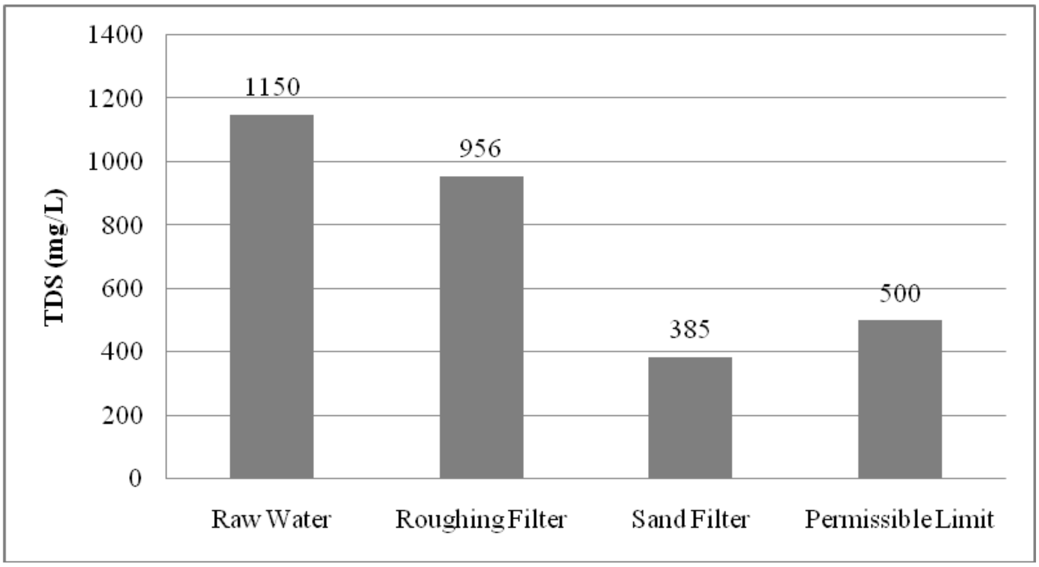
Salinity is the single most important parameter in determining the suitability of the water to be used for irrigation. Salinity is determined by measuring the electrical conductivity (EC) and / or total dissolve solid (TDS) in the water. From this study the obtained value of EC and TDS after reclamation was found 3.14 µS/cm and 385 mg/l, respectively. These two values satisfied the permissible limit for wastewater irrigation. The salinity tolerance of plants varies widely. Crops must be chosen carefully to ensure that they can tolerate the salinity of the irrigation water, and even then the soil must be properly drained and adequately leached to prevent salt increase. Leaching is the purposeful over application of irrigation water in excess of crop needs to create a downward movement of water and salt away from the root zone. The amount of salt accumulation in the soil depends on the concentration of salts in the irrigation water and the rate at which salts are removed by leaching. Salt accumulation can be especially harmful during germination and when plants are young (seedlings). At this stage, damage can happen even with relatively low salt concentrations. Concerns with salinity relate to possible impacts to the following: the soil's osmotic potential, specific ion toxicity, and degradation of soil physical conditions. These conditions may result in reduced plant growth rates, reduced yields, and, in severe cases, total crop failure (EPA, 2004).

3.2 Chloride:

From the experimental data it was found that the amount of chloride decreased to 219 mg/l which is within the permissible limit for wastewater irrigation standard. Some sensitive plants are damaged at low chlorine concentration. Some woody crops, however, may gather chlorine in the tissue to toxic levels. Excessive chlorine has a similar leaf burning outcome as sodium and chloride when sprayed directly on foliage. So the concentration of chlorine should less.



(a)



(b)

Figure 2: Performance study of the developed treatment unit (a) Hardness (b) TDS.

3.3 Nutrients:

The nutrients most important to a crop's needs are nitrogen, phosphorus, potassium, zinc, boron, and sulfur. Reclaimed water usually contains enough of these nutrients to supply a large portion of a crop's needs. The amount of sulphate was reduced 196 mg/l to 120 mg/l and the amount of nitrate was reduced 4.4 mg/l to 0.2 mg/l. The most beneficial nutrient is nitrogen. Both the concentration and form of nitrogen need to be considered in irrigation water. While excessive amounts of nitrogen stimulate vegetative growth in most crops, it may also delay maturity and reduce crop quality and quantity. The nitrogen in reclaimed water may not be present in concentrations great enough to produce satisfactory crop yields, and some supplemental fertilizer may be necessary. In addition, excessive nitrate in forages can cause an imbalance of nitrogen, potassium, and magnesium in grazing animals.

3.4 Organic Matter and Bacteria:

BOD_5 and COD reduced 62 mg/l to 30 mg/l and 112 mg/l to 48 mg/l, respectively after filtration. So the removal efficiency of the of the greywater treatment unit in organic matter removal is about 60%. The color of the raw waste water was 497 Pt. Co. unit where after filtration it became 87 Pt. Co. unit. So the color removal efficiency is 83%. The microbiological water quality in terms of total coliform (TC) and E. coli was also improved satisfactorily with average removal efficiency about 60%. The treated water was very clear with turbidity value roughly 3 NTU. The hardness and total solid removal efficiency were noticed to be 41% and 82%, respectively. The water quality of raw wastewater and treated water at different stages and comparison with WHO standard for wastewater irrigation is depicted in figure 2.

4. RECOMMENDED LAYOUT IN HOUSE FOR GREYWATER REUSE:

The developed greywater treatment unit would be located at any suitable subsurface level around the house premises. The treatment unit would have two chambers containing brick khoa and sand. A storage tank following the treatment operation could provide storage of the treated water for its potential reuse purposes.

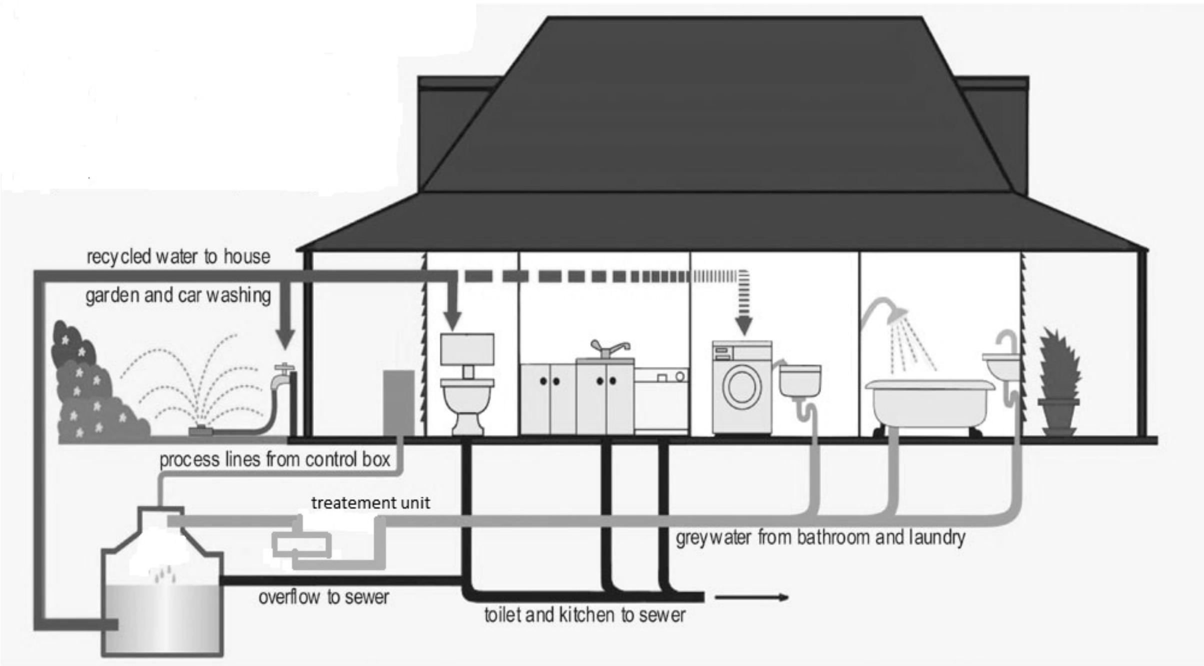


Figure 3: Recommended layout in house for greywater reuse.

Greywater is collected from bathroom, basin, and laundry by connecting them through pipe. This water will pass through filter and the reclaimed water will be stored in the storage tank as shown in figure 3. The reclaimed water will be pumped from storage tank. There must have provision for excess water to overflow to sewer. Then the water will be used for different purposes like gardening, car washing, toilet flushing. If the treated water is clear and microbiologically save then it may be used for laundry.

5. CONCLUSIONS:

The treated water quality was found to be satisfying the permissible limit for wastewater irrigation. Thus, the reclaimed greywater could be efficiently used for household gardening and agricultural purposes. Toilet flushing might be another option for this water reuse. The microbiological water quality in terms of total coliform (TC) and E. coli was also improved satisfactorily with average removal efficiency about 60% and reduced to 10 and 7 cells, respectively. The treated water was very clear with turbidity value roughly 3 NTU. The hardness and total solid removal were also satisfactory. Thus, the reclaimed greywater can be used for car washing and also may be a potential source of laundry use. Based on the above findings, it can be concluded that the recycling and reuse of treated greywater would have promising application in selected household and agricultural uses after simple treatment without any hazard to human health and environment.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ARSENIC REMOVAL FROM HIGHLY CONTAMINATED GROUNDWATER BY IRON OXIDATION CERAMIC FILTER UNITS WITH DIFFERENT IRON SOURCES

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ABSTRACT

The goal of this study was to induce a low-cost ceramic filter unit for arsenic removal from the groundwater of the rural Bangladesh. The proposed treatment unit was composed of a ceramic filter, iron-mesh and clay pot. The filter unit removes arsenic through adsorption and co-precipitation onto iron hydroxide solid phase. Field studies were conducted to investigate the performances of 3 different iron sources as iron net, scrap iron and iron rod with the ceramic filter units with iron oxidation process to treat very highly arsenic contaminated groundwater. 3 nos. of filter units were provided in a household having the tubewell of highly arsenic contaminated water in Dighalia upazilla of Khulna district. The raw water and treated water samples were collected and selected water quality parameters including arsenic, iron, phosphorus, color, turbidity etc. were tested in Environmental Engineering laboratory of Khulna University of Engineering and Technology (KUET), Khulna, Bangladesh. The removal efficiency of arsenic was found 73.63%, 74.42% and 73.73% respectfully for filter units with net, scrap and iron rod. Iron removal efficiency was found around 100% for every filter unit. Color removal efficiency was 97.15%, 94.70% and 97.11% respectively. Also, turbidity removal was satisfactory and removal efficiencies for net, scrap and rod were nearly same.

Keywords: *Arsenic, iron oxidation, ceramic filter, oxidation, filtration.*

1. INTRODUCTION

Arsenic (As) contamination of groundwater is major health concern on a global scale. Arsenic contaminated groundwater has been found in Argentina, Chili, Mexico, China, Hungary, West Bengal of India, Bangladesh and Vietnam. Of these regions, West Bengal and Bangladesh are most seriously affected in terms of the size of the population at risk and the magnitude of the health problems.

Arsenic (As) is a well-known toxic metal and is present mainly as oxyanion compounds in groundwater. Two forms of arsenic are found chemically in ground water: inorganic and organic. Inorganic Arsenic has four main chemical forms having oxidation states, -3, 0, +3, and +5, but in natural water its predominant forms are inorganic oxyanions of trivalent arsenite (As (III)) or pentavalent arsenate (As (V)). As (V) is the predominant species under atmospheric or more oxidizing environment, which exits predominantly as oxyanions, namely, $H_2AsO_4^-$ or $HAsO_4^{2-}$ in the pH range of 6-9. As (III) is thermodynamically stable and exits predominantly as H_3AsO_3 or $HAsO_2$ under mildly reducing conditions (Smedley and Kinniburgh, 2002). The toxicity of different arsenic species varies in the order arsenite> arsenate> monomethylarsonate> dimethylarsinate. As (III) is about 60 times more toxic than arsenic in the oxidized As (V), while inorganic arsenic compounds are about 100 times more toxic than organic arsenic compounds (Jain and Ali, 2000).

A large amount of affected people has been identified in Bangladesh rural area which is arsenic related disease ranging form melanosis to skin cancer and gangrene. A recent report maintain that arsenic contaminated tubewells water is contributing to nearly 125,000 cases of skin cancer and killing about 3000 people in Bangladesh each year (Clark, 2003).

Developing countries like Bangladesh, India and Vietnam cannot afford expensive and/or large scale treatments. Low-cost, effective technologies that are readily available at the household or community level are needed to solve the present crisis. Large-scale treatments are not appropriate because many people in third-world countries obtain water from wells rather than from large municipal water plants. Appropriate in-home technologies to be implemented in third-world countries should meet certain criteria to be effective. The treatment must be applicable over a wide range of arsenic concentrations and easy to use without running water or electricity, and the materials for the treatment must be cheap, readily available, and/or reusable to reduce costs. Finally, such technology should not introduce any harmful chemicals into drinking water.

Among the arsenic removal technologies, adsorption and subsequent co-precipitation with iron salts is the simplest and convincible arsenic removal technique. Iron salts occur in two forms, Fe(II) and Fe(III), whilst removal by Fe(III) salts are more commonly used technology (Katsoyiannis and Zouboulis, 2002; Thirunavukkarasu et al., 2003; Zeng, 2003) . Arsenic removal by Fe(III) salts need pre-oxidation of As(III) to As(V) because As(III) is the most common species in anaerobic ground waters (Harvey et al., 2002) and generally is removed less efficiently than the oxidized As(V) (Dixit and Hering, 2003).

Recent studies have been found that As(III) is partially oxidized by reactive intermediates (possibly Fe(IV) species) form during the physicochemical oxidation of Fe(II), results the high efficiency of arsenic removal than those of Fe(III) (Berg et al., 2006; Hug and Leupin, 2003). The removal of As(III) with Fe(II) is thus expected to have advantages over Fe(III).

Fe (II) can be oxidized by both physicochemically and biologically but the dominant one is depend on the physical and chemical characteristics of the raw water and process conditions. The biological iron oxidation is caused by the presence of several iron oxidizing microorganisms in water. *Gallionella* sp and *Leptrothrix ochracea* cause primary intercellular oxidation by enzymatic action, while secondary extracellular oxidation is caused by the catalytic action of polymer excreted filaments (Czekalla et al., 1985). A biological process of iron removal has advantages than that of physicochemical process. Mounchet (1992) reported that a biological process could have high filtration rate, high retention capacity, flexibility of operation and reduced the capital cost. On the other hand, the rate of iron oxidation can be increased in the presence of iron oxidizer (Michalakos et al., 1997). Thus the arsenic removal method based on biological iron oxidation would be an ideal option in developing countries such as Bangladesh and India.

In this study, 3 nos. of treatment units having ceramic filter with three types of iron sources as iron net, scrap iron and iron rod were installed in a household having tubewell of of very highly arsenic contaminated tubewell water. The main objective of this study was to evaluate and compare the performances of these three different filter units.

2. METHODOLOGY

The main components of the filter unit are: Ceramic filter, Iron net/ Scrap iron/ Iron rod, Iron bacteria sludge, Reactor (14 16 L Clay pot was used), Effluent storage bucket, Wooden stand etc.

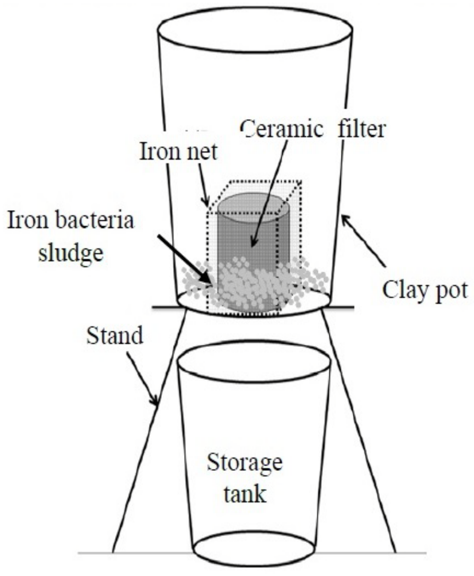

Figure 1: Components of Filter Unit with iron net

2.1 Manufacturing of Ceramic Filter

The filter was made with locally available and cheap materials as rice bran, clay soil and water.

- Oven-dry soil was grind with hammer. Then soil and rice bran was screened through 0.5 mm and 1 mm sieve respectively. Soil (640g for 1 filter) and rice bran (160g for 1 filter) was taken in ratio of 80:20.

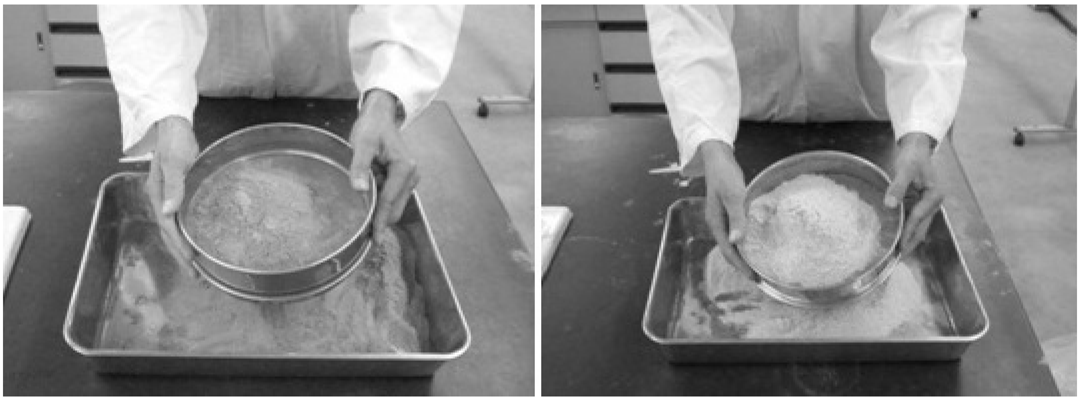

Figure 2: Sieve analysis of soil and rice bran

- Soil and rice bran was mixed homogeneously with water to make dough. Then dough was placed around the bar of the dice and two pieces of PVC pipe were pushed by hand from both sides to make cylindrical shape.

Figure 3: Mixing of soil and rice bran and making of dough with water

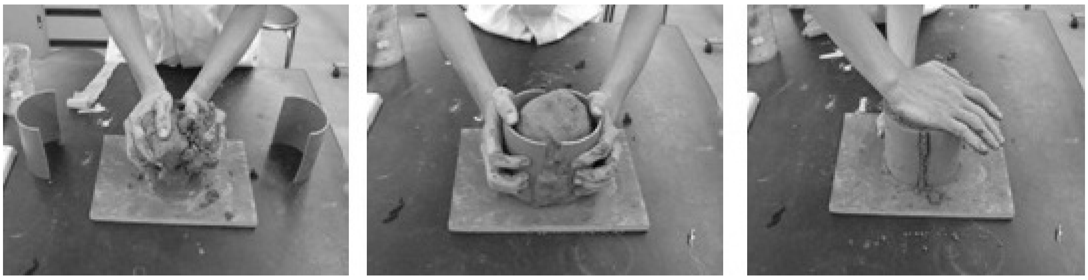

Figure 4: Shaping of filter from dough using dice

- Next the pipes were taken off and the surface of the filter was polished with water. The total frame was then toppled down to remove the dice.

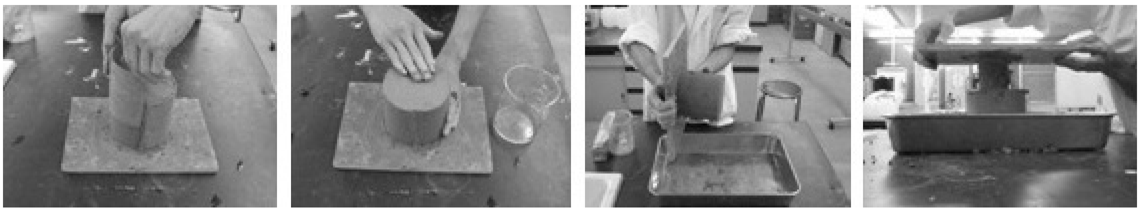

Figure 5: Final step to get raw ceramic filter

- The resulting cylindrical ceramic filters were hollow with one side open. This soft filter was then dried in the sun for at least 3 days.
- The air dried filters were burnt in potter kiln at 900 to 1000℃. After continuous burning for 6 to 8 hours, the kiln was kept to cool down. After some hours the filters were taken out from the kiln. The final ceramic filters had a height of 10 cm and a thickness of 2 cm.

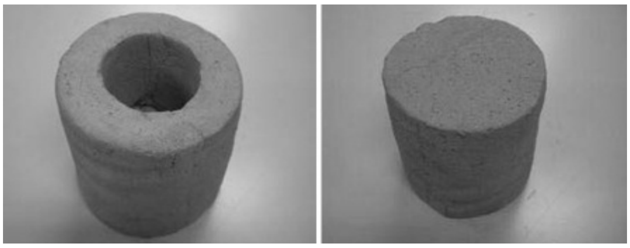

Figure 6: Final ceramic filter before burning

Figure 7: Filter burning in Potter kiln

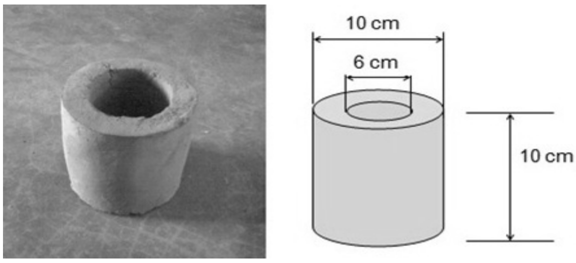

Figure 8: Final ceramic filter after burning

2.3 Preparation of Iron Net, Scrap Iron and Iron Rod

600 gm commercially available iron net without coating was taken and 11cm×11cm×11cm cube with one side open by the iron net was made.
600 gm of scrap iron was collected from the local lathe. Iron rod was collected and cut in small shape for the convenience in using in the filter system.

Figure 9: Iron net, scrap iron and iron rod for the filter unit

2.4 Preparation of Iron Sludge

Water was filled in a big drum of capacity 100 L. Some iron net, iron bar and other iron materials was added in the drum. This was aerated with a stick for 5 minutes daily. Iron bacteria layer will be deposited at the bottom of the drum after 10-15 days.

Figure 10: Iron bacteria culturing and collected iron bacteria sludge

2.5 Conceptual Arsenic Removal Mechanism

This removal of arsenic occurred due to the oxidation of iron and arsenic followed by their subsequent adsorption and precipitation on and with biologically produced iron hydroxides. Biological oxidation of iron by iron bacteria is the main mechanism in respect to the removal of arsenic in this study.
Both forms of inorganic arsenic (AS (III) and As (V)) could be removed more efficiently during iron oxidation than formed iron precipitation. This might be because a very fine iron hydroxide floc is produced which had the high adsorptive surface area and high binding energy resulting in the effective removal of both forms of arsenic at the beginning of biological iron oxidation.
Firstly Fe (II) oxidation is catalyzed by the iron bacteria and transformed to Fe (III). Secondly a part of As (III) is oxidized to As (V) in the presence of Fe (II) and the iron bacteria. Finally adsorption of As (V) on iron hydroxides occurred. These processes are schematically shown in Figure 11.

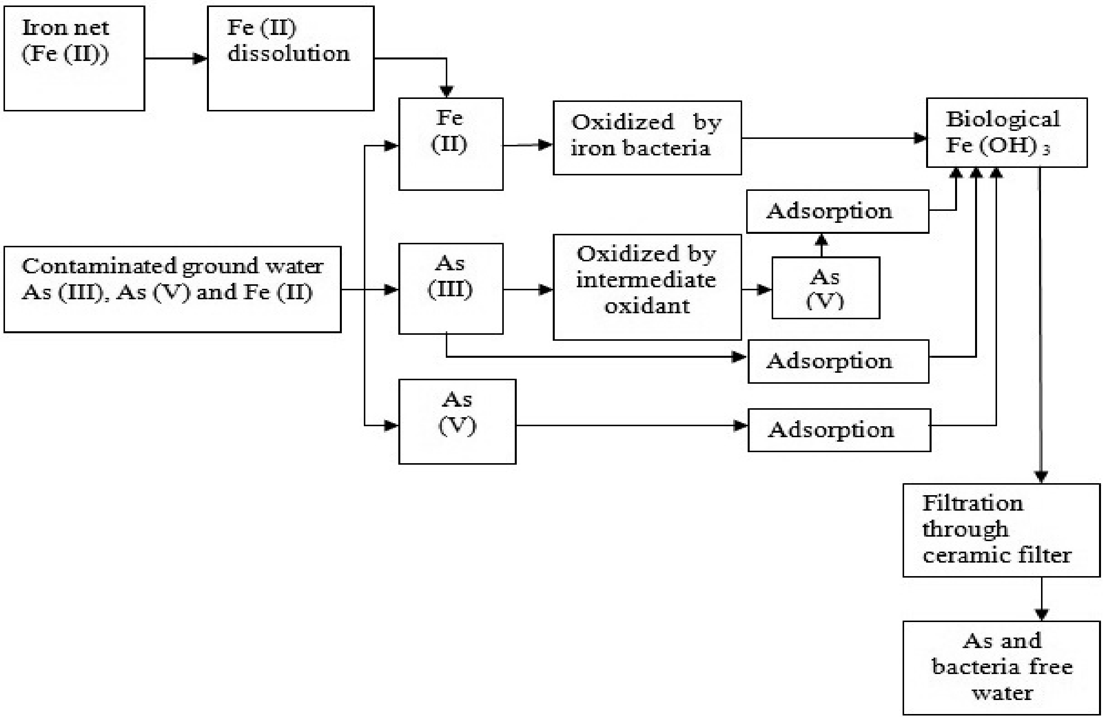

Figure 11: Conceptual Mechanism of arsenic removal by iron oxidation

3. RESULTS ANG DISCUSSION

The filter units were installed in a household having highly arsenic contaminated tubewell water in Dighalia upazilla of Khulna district in Bangladesh. The treatment units had been used regularly for more than 6 months. The influent and effluent samples were collected once a week. In addition to arsenic, other water quality parameters (i.e. iron, color, turbidity etc.) were analyzed following Standard Methods in the Environmental Engineering laboratory of Department of Civil Engineering of Khulna University of Engineering and Technology.

3.1 Removal Performance of Arsenic

The raw water of the household tubewell contained very high level of arsenic concentration. Average, highest and lowest arsenic concentrations for influent and effluent options are shown in Table 1.

Table 1: Variation of Arsenic concentration of Influent and Effluent

	Average As (µg/L)	Highest As (µg/L)	Lowest As (µg/L)	Average Removal Efficiency (%)
Influent	489.7	582.7	399.2	-
Effluent (Net)	125.6	211.4	100.3	73.63
Effluent (Scrap)	125.5	235.3	94.6	74.41
Effluent (Rod)	127.5	254.9	95.2	73.73

It was found that the performance with respect to the arsenic removal efficiency for different filter options were almost same. Whichever the source of iron in the unit was the removal capacity did not change. The average values are presented graphically in Figure 12.

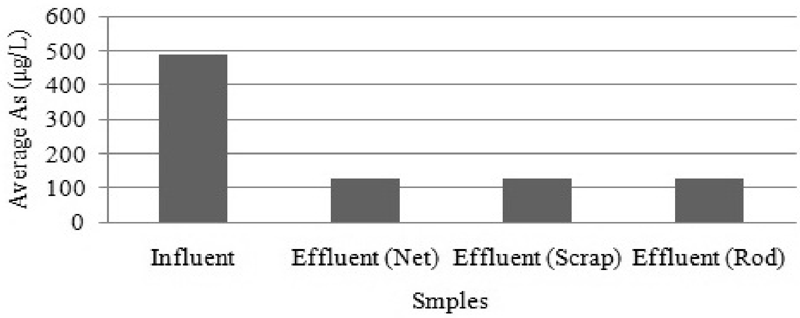

Figure 12: Average values of Arsenic of Influent and Effluent options

3.2 Removal Performances of Other Parameters

3.2.1 Iron Removal

The influent water contained very high amount of iron. The average Fe (2+) concentration was 7.90 mg/L with highest and lowest value 10 mg/L and 5 mg/L respectively. Our proposed filter unit system was very much effective in reducing Fe (2+) concentration level. Effluent from the filter units with all the options as iron net, scrap iron and iron rod were able to remove Fe (2+) almost fully in all cases.

3.2.2 Color Removal

In this study, it was found that the influent raw water contains high level of color. Also our filter unit reduced the color level significantly.
In Table 2, the removal characteristics of the filter options are presented. It is shown that the removal capacity is satisfactory and almost same for different options. It is also shown graphically in Figure 13.

Table 2: Variation of Color of Influent and Effluent

	Average Color (Pt-Co)	Highest Color (Pt-Co)	Lowest Color (Pt-Co)	Average Removal Efficiency (%)
Influent	473.42	620	359	-
Effluent (Net)	14.26	49	5	97.15
Effluent (Scrap)	25.90	93	12	94.70
Effluent (Rod)	14.16	41	5	97.11

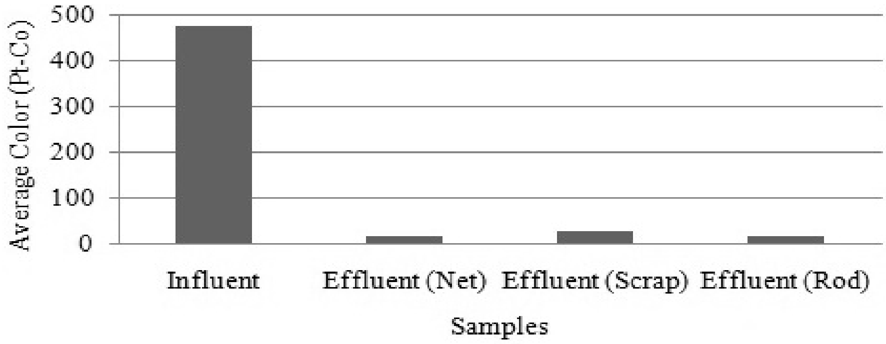

Figure 13: Average values of Color of Influent and Effluent options

3.2.3 Turbidity Removal

The influent raw water of this field study contains about 99 NTU of turbidity. This proposed filter unit reduced the turbidity level of raw water significantly. Average, highest and lowest Turbidity for influent and effluent options are shown in Table 3. The average removal efficiency is also shown in the table.

Table 3: Variation of Turbidity of Influent and Effluent

	Average Turbidity (NTU)	Highest Turbidity (NTU)	Lowest Turbidity (NTU)	Average Removal Efficiency (%)
Influent	99	211.2	61.1	-
Effluent (Net)	10.25	25.2	5.8	89.69
Effluent (Scrap)	10.65	20.5	5.8	89.20
Effluent (Rod)	10.79	27.6	4.5	89.23

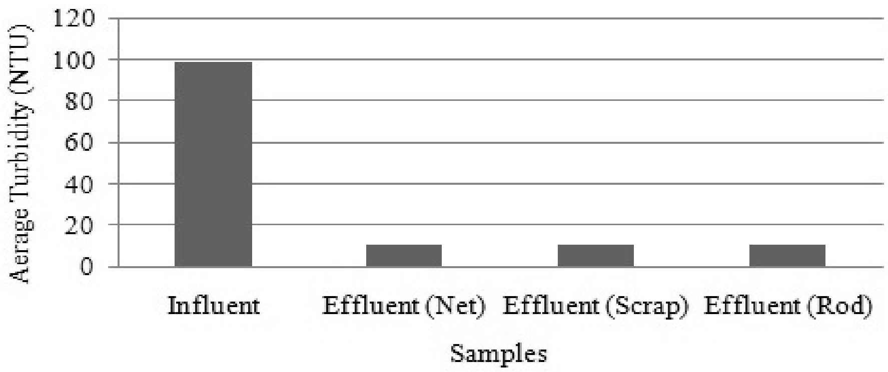

Figure 14: Average values of Turbidity of Influent and Effluent options

3.3 Economic Aspects:

The approximate cost of the filter unit was around 200-250 BDT. Other important point was that the unit was reusable. After six months of regular using with cleaning in every month, the filter core can be replaced by new one and the total filter unit can be started using again.

4. CONCLUSIONS

The main findings of the study may be drawn as follows:

- High arsenic removal efficiency was found without using any additional chemicals. As influent water contained extraordinarily high arsenic concentration, treated water couldn't maintain standard value for drinking purpose. It may be fully satisfactory for less contaminated groundwater samples.

- There was no significant difference in the removal efficiency for different options as filter unit with iron net, scrap iron or iron rod.

- Iron, color, turbidity removal was fully satisfactory (satisfying the guideline values) for every option.

- The treatment unit was low-cost and easily affordable for the rural people. Also the maintenance and operation was very easy and simple**.**

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ESTABLISHMENT OF BOD AND COD RELATIONSHIP OF DIFFERENT WASTEWATER

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ABSTRACT

BOD is the most widely used parameter to measure the discharge quality of waste water, and COD is a measure of total organic molecules dissolved in waste water. These concepts were introduced while people starts throw the waste water into the water bodies. These tests are also used to measure the organic matter in industrial and municipal wastes that contain compounds that are toxic to biological life. For many type of waste it is possible to correlate COD with BOD_5. This can be very useful because the COD can be determined in 3 hr, compared with 5 days for the BOD5. Once the correlation has been established, COD measurements can be used to good advantage for treatment plant control & operation (Metcalf & eddy, 1996). The ratio of BOD_5 and COD ranges from 0 to 1. The higher value indicates easily biodegradable waste water and the lower value indicates slowly biodegradable waste water. There is no generalized correlation between the 5-day BOD and the ultimate BOD. Similarly there is no generalized correlation between BOD_5 and COD. It is possible to develop such correlations for specific waste contaminants in a specific waste water stream but such correlations cannot be generalized for use with any other waste water streams. This is because the composition of any waste water stream is different. In this study several BOD_5 and COD tests are done with different wastewaters by standard method. Ultimate BOD and BOD rate constants are determined by Least Square and Fujimoto method. 0.61, 0.37 and 0.14 are the BOD_5 /COD ratio values of Domestic, Municipal and Industrial wastewater. Several relationships are developed of BOD5 -COD, BOD_u –BOD_5 for three waste water types. More and more tests should be done for the accuracy of correlation. A simple expression has been derived to demonstrate that the ratio of COD to BOD. An analysis of data reported in literature indicates that this ratio can assume a wide range of values for different substrates. The influence of a selected change in the environmental conditions the ratio, COD/BOD, has been evaluated experimentally, in a bacterial system. There is no uniform relationship between COD and BOD, which can be applied to all wastewaters, as BOD depends on the amount of biodegradable organics present in a specific wastewater. It can be only being said that COD will be greater than BOD. However for a specific waste, COD and BOD can be determined couple of times and COD/BOD ratio can be determined from the average values. For biodegradable compounds, the value of BOD will tend to approach COD as the test period increases. However, ultimate BOD will be less than COD, since a proportion of the compound biodegraded is not oxidized as it used for cell growth.

Keywords: *Title, abstract, methodology, results, conclusions*

1. INTRODUCTION

All natural waterways contain bacteria and nutrients, almost any waste compounds introduced into such waterways will initiate biochemical reactions (such as shown above). Those biochemical reactions create what is measured in the laboratory as the Biochemical oxygen demand (BOD). Such chemicals are also liable to be broken down using strong oxidizing agents and these chemical reactions create what is measured in the laboratory as the Chemical oxygen demand (COD). Both the BOD and COD tests are a measure of the relative oxygen-depletion effect of a waste contaminant. Both have been widely adopted as a measure of pollution effect. It seems that BOD varies linearly with the COD value of a particular waste. Since there is good correlation between BOD and COD, BOD can be calculated from the less time consuming COD. Ultimate BOD is so time consuming, the 5-day BOD has been almost universally adopted as a measure of relative pollution effect. The BOD5, BODu relation gives advantages to measure BODu from BOD5 value.

There is no generalized correlation between the 5-day BOD and the ultimate BOD. Similarly there is no generalized correlation between BOD and COD. It is possible to develop such correlations for specific waste contaminants in a specific waste water stream but such correlations cannot be generalized for use with any other waste contaminants or waste water streams. This is because the composition of any waste water stream is different. As an example and effluent consisting of a solution of simple sugars that might discharge from a confectionery factory is likely to have organic components that degrade very quickly. In such a case the 5 day BOD and the ultimate BOD would be very similar. I.e. there would be very little organic material left after 5 days.

The specific objectives are:

- To establish relationship among BOD5, BODu, COD for different waste water.
- To determine BOD rate constants for different waste water.

2. METHODOLOGY

If any one likes to predict BOD (test takes 3 or 5 days) by doing COD (3 hrs and easy) of a particular sample for routine monitoring purpose, one can establish Correlation Regression equation through statistical procedure by analyzing both tests for about 20 times. It is same for developing BOD5 and BODu correlation. The flow diagram of methodology is shown below.

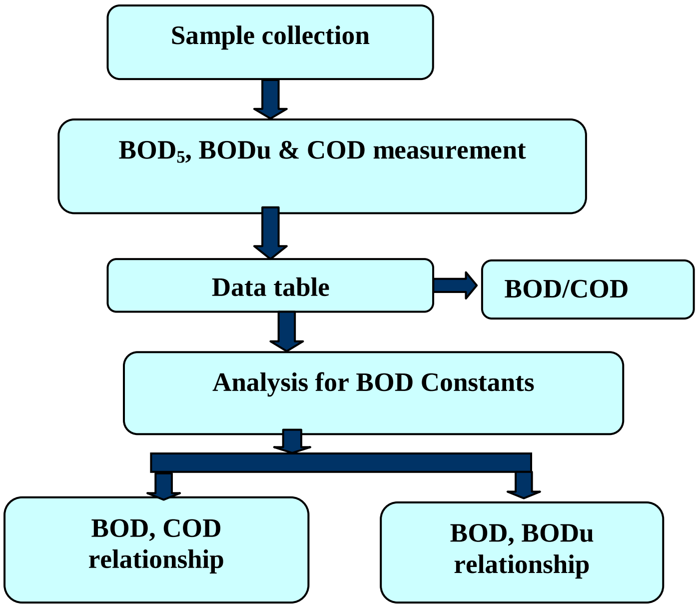

Fig. 2.1: Methodology diagram of work.

Chemical oxygen demand (COD) and biochemical oxygen demand (BOD_5) are two of the most common generic indices used to assess aquatic organic pollution. Concentrations of BOD_5 readings will generally report as lower than COD. This is due to differences in the methods of oxidation of the samples. While BOD_5 provides a good approximation of the biologically consumable organic fraction in waterways, the test takes 5 days. Alternatively, COD is able to provide a rapid and reliable estimate of the biogeochemical interactions in waterways. The total amount of oxygen consumed when the biochemical reaction is allowed to proceed to completion is called the Ultimate BOD.

Establishment of constant relationships among the various measures of organic content depends primarily on the nature of the wastewater and its source. Of all the measures, the most difficult to correlate to the others is the BOD_5 test, because of some problems. For typical untreated domestic wastes, however, the BOD_5/COD ratio varies from 0.4 to 0.8. (Metcalf & eddy, 1996) This ratio can also vary widely depending on the state of biodegradation of the wastewater. It has found this ratio as low as 0.1 after several days of oxidation. If the BOD_5 of a biodegradable wastewater equals zero, the wastewater will be completely biodegraded.

If the BOD is determined after 5 days, this is called "the 5 day BOD" (BOD_5). If the BOD is determined after 20 days, this is called "the 20 day BOD" (BOD_{20}). These are really BOD exerted values.

The "ultimate BOD" is the amount of oxygen required to decompose all of the organic material after "infinite time". This is usually simply calculated form the 5 or 20 day data. The Ultimate BOD is too time consuming, so the 5-day BOD has almost universally been adopted as a measure of relative pollution effect.

Limitations

- *Non-biodegradable organic waste is unaccounted for.*
- *Very dirty water will use up all dissolved oxygen before the 5 days is over.*
- *Other aerobic activity (biological or just chemical) is counted.*

3 Estimation of BOD kinetic Parameters

Using the results obtained from serial BOD test, BOD and time were plotted and through extending the smooth curve passing through the data points to the x-axis time lag involved in the test was estimated. On the basis of the lag obtained the first order BOD kinetic equation was corrected as below:

$BOD_t = L_0 \, (1\text{-}e^{-kt})$.

The corrected kinetics equation was used in all the calculations, except in case of method of moments, the original BOD kinetic equation and homograph for n = 7days was used. Using the results obtained from the serial BOD tests, BOD kinetics parameters (k and L_0) were estimated by the following six different methods, which are commonly used:

(i) Method of Moments
(ii) Least Squares Method
(iii) Thomas Graphical Method
(iv) Daily Difference Method
(v) Iteration Method
(vi) Fujimoto Method

3.1 Method of Moments

This method involves use of Moore's diagram which is actually a nomograph showing relationship between k, ∑BOD/L0 and ∑BOD/∑BOD.t. From the series of BOD measurements for 7-days, ∑BOD/∑BOD.t was calculated and 'k' value and ∑ BOD/L0 value corresponding to this ∑BOD/∑BOD.t value were read from the Moore's diagram specific to 7-days. From the ∑BOD/L0 value obtained, L0 was calculated. Moore's diagrams are constructed through the following equations:

$$\sum BOD/L_0 = n - [10^{-k}\,(10^{-nk} - 1)/\,(10^{-k}-1)] \quad (3.1)$$

$$\sum BOD / \sum BOD.t = \frac{n - [\frac{10^{-k}(10^{-nk}-1)}{10^{-k}-1}]}{\sum_{i=1}^{i-n} i - \sum_{i=1}^{i-n} i * 10^{-ik}} \quad (3.2)$$

Where,
BOD_t = BOD exerted in time't' days of incubation.
n = No. of days of incubation for the serial BOD test.
k = BOD rate constant
L_0 = Ultimate BOD.

The above expressions have been used for calculating ∑BOD/L0 and ∑BOD/∑BOD.t values for n = 7 days. These calculated values have been used for constructing the required Moore's diagram. (Ramallho, 1983)

3.2 Least Squares Method

According to first order kinetics

$$dL/dt = - kL_t \quad (3.3)$$

Where,
$L_t = L_0 - y_t$
$y_t = BOD_t$
$dy/dt = k\,(L_0 - y_t)$

$dy/dt = kL_0 - ky_t$

This is a linear equation. Through use of least squares method k & L_0 values in the above linear equation can be found out. In the calculations the following equation are used:-

$S_{xx} = n \sum y_t^2 - (\sum y)^2$ (3.4)

$S_{xy} = n\sum y_t (dy/dt) - (\sum y_t)(\sum dy/dt)$ (3.5)

Slope $(-k) = S_{xy} / S_{xx}$ (3.6)

Intercept $(kL_0) = \sum (dy/dt)/n + k\sum(y_t)/n$ (3.7)

L_0 = Intercept/ (-slope) (3.8)

(Metcalf Eddy 1991)

3.3 Thomas Graphical Method

This is an approximate method. It is based on the following equation:

$$(t/y)^{1/3} = 1/(2.3\ kL_0)^{1/3} + [(2.3\ k)^{2/3}/6\ L0^{1/3}]\ .\ t \quad (3.9)$$

Plot of $(t/y)^{1/3}$ versus t gives slope as $(2.3\ k)^{2/3}/6\ L_0^{1/3}$ and intercept as $1/(2.3\ kL_0)^{1/3}$.

The kinetics parameters are calculated as follows:

k = 2.61(slope/intercept) (3.10)

L_0 = 1/ (2.3 k. intercept3) (3.11)

(McGhee 1991)

3.4 Daily Difference Method

According to first order equation:

$y = L_0 (1- 10^{-kt})$

$dy/dt = L_0 (-10^{-kt})(\ln 10)(-k)$

$\log (dy/dt) = \log(2.303\ kL_0) - kt$ (3.12)

Plotting log (dy/dt) versus time (mid interval value of‘t’) gives slope as –k and intercept as $\log(2.303\ kL_0)$.

Ultimate BOD (L_0) can then be obtained by the following equation:

$L_0 = 10^{(intercept)} / 2.303\ (k)$. (3.13)

(Ramallho, 1983)

3.5 Iteration Method

This iteration method for the analysis of time series of BOD data and found the results very close to that of least squares method.

Procedure:

(i) Assumed the ultimate BOD (L_0) equal to the last BOD value.

(ii) Calculated k from first order equation.

$y = L_0 (1 - e^{-kt})$ (3.14)

Using L_0 as in step (i) and using first BOD data (y and t).

(iii) Calculated L_0 from equation using k from step (ii).

(iv) Calculate k from equation using L_0 from step (iii).

Repeated the calculation of k using just calculated value of L0 and the given BOD data from start and L_0 using just calculated value of k and the given BOD data from last till all the given data are used up. The values of k & L_0 obtained in the last step are their correct values.

(R.K. Rai, 2000)

3.6 Fujimoto method

Using this method an arithmetic plot was prepared of BOD_{t+1} versus BOD_t. The value at the intersection of the plot with a line of slope 1 corresponds to the ultimate BOD. The rate constant k was determined from the following equation:

$$BOD_t = L_0 (1-e^{-kt}) \text{ --- } (3.15)$$

Where,

BOD_t = BOD exerted in time't' days of incubation,
L_0 = Ultimate BOD, t = time (days) (Metcalf Eddy 1991)

4 Overview of Works

In this project domestic, municipal and industrial waste water are selected for COD & BOD test. For domestic waste water KUET drain water is collected. For industrial waste water, SAF industry effluent is collected and for municipal waste water, khalishpur drain water is collected. Several BOD_5 & corresponding COD values are tested in laboratory for these three samples. The BOD_5 & corresponding COD value are then plotted in plain graph. From the plot BOD_5, COD equation is developed for these three samples. And the error is then compared by RSQ values for each equation. On the other hand BOD_5, COD ratio is also calculated from the previous values.

Table 4.1: BOD5/COD ratios for different sewage

Wastewater type	**Domestic**	**Municipal**	**Industrial**
BOD_5/COD	0.60	0.48	0.25

(Henry & Heinke, 1978)

Table 4.1 shows the BOD5/COD ratio for different waste waters. The smaller ratio means the waste samples are not easy to biodegrade and higher cod values and for higher ratio, vice versa. For ultimate BOD and COD relation and BOD rate constant determination municipal waste water and Industrial waste water is selected. 7 days BOD values are measured for ultimate BOD determination. BOD rate constant is also determined by those values. BOD rate constant is determined by least square method. For test results, 0.01 dilution factor is used for BOD test and 0.5 for COD test is taken.

4.1 Sampling

Sampling is an extremely important consideration in properly characterizing wastewater. Flow rate and wastewater quality changes continuously, and these changes may affect the ability of a wastewater to achieve consistent biological changes. A desirable sampling method is to collect a 3-4 hour composite sample. This will provide data that may be considered representative of average wastewater characteristics throughout the day while minimizing the sample holding time. 500 ml bottles are used to collect sample from the waste water stream. The wastewaters are collected at the maximum flow time of a day.

4.2 Serial BOD and COD testing

For BOD $_5$ measurements same procedure is done. Here 3 BOD bottle is placed in incubator for 5th day DO measurement. For estimating the BOD kinetics parameters, k and Lo, serial BOD measurements for the first 7 days were made for the prepared samples incubated at 20^{o}C. That is, BOD1, BOD2, ---and BOD7 were measured for the sample in question. 24 BOD bottles were used in the experiment for facilitating daily DO measurement. Dilution factor approximating to1/100 was used for diluting the sample. Aerated distilled water was used as dilution water. While analyzing 3 of the bottles for initial DO, rest of the bottles were incubated in a BOD incubator at 20^{o}C for 7 days. Every day 3 of the incubated bottles were taken out and tested for DO while using the BOD measurement system. BOD of the sample was estimated by using the following expressions:

$$BOD_t \text{ at } 20^oC = (DOi-DO_f)/\ DF$$

(3.1)

Where,

BODt = BOD exerted in't' days of incubation.
DOi = DO of the diluted sample immediately after preparation, mg/l.
DO_f = DO of the diluted sample at particular day of incubation, mg/l.
DF = Dilution factor.

(Peavy. H. S., et al , 1985))

For determination of COD values standard method is followed as stated before 3.2.1. here dilution factor is taken 1/2. COD value is calculated by,

$$\frac{(A-B)*8000}{M} \quad (3.2)$$

Where,

A= ml of FAS (Ferrous Ammonium Sulfate) need to titrate distilled water
B= ml of FAS (Ferrous Ammonium Sulfate) need to titrate wastewater
M= 5*0.1/FAS
FAS= molarities of Ferrous Ammonium Sulfate

Ultimate BOD and BOD rate constant determined by Least Square method and Fujimoto method which is stated before. Sample calculation is shown below,

4.2.1 Sample calculation for Least Square Method

The kinetic parameters k & L_0 of the Khalishpur drain effluent were estimated as follows:

Step 1:
Constructing following table:

Table 4.2: Least Square Method for Khalishpur sample.

Time(day)	**y_t(mg/l)**	**dy/dt= $(y_{t+1}-y_{t+1})/2\Delta t$**	**y_t^2**	**$y_t.dy/dt$**
1	206	174	42400	35844
2	348	96	121100	33408
3	398	596	158400	23700
4	467	79	218100	36900
5	556	114.5	309100	63660
6	696	74	484400	51500
7	704*			
sums	2671	597.1	1333500	245000

* Value not included in total and n = 6 is used.

Step 2:
Substituted the values computed in Step 1 in eq. (3.4) and (3.5).

Sxx = 866759
Sxy = -12484.1

Step 3:
k and L_0 is being Calculated by using eq. (3.6), (3.7) and (3.8).

k = 0.144day^{-1}
L_0 = 1136.25 mg/l

4.2.2 Sample calculation for Fujimoto Method

The kinetic parameters k & L_0 of the river Khalishpur drain effluent were estimated as follows:
Step 1:
Prepared and arithmetic plot of BOD_{t+1} versus BOD_t (fig. 4.1) using the following table:

Table 4.3: Fujimoto Method for Khalishpur sample.

Sr. No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**
BOD_t(mg/l)	206	348	398	467	556	696	704
BOD_{t+1}	348	398	467	556	696	704	-

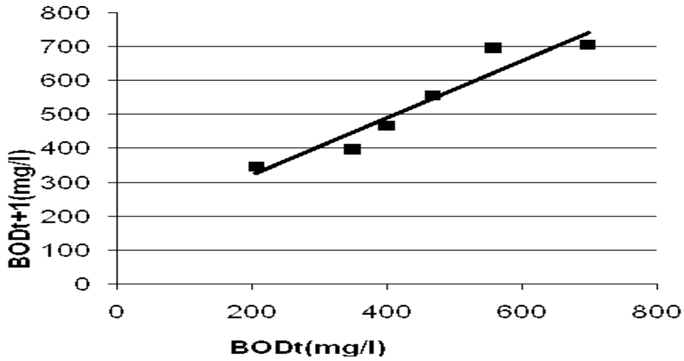

Fig 4.1: Fujimoto method for Khalishpur Sample.

Step 2:
A line with slope of 1 was drawn on the same plot as constructed in step 1. The value at the intersection of the two lines corresponds to ultimate BOD, L_0 = 27 mg/l.
Step 3:
The k value for 5th day data is being determined by using equation 2.15.
$BOD_5 = 5.14 = 27 (1-e^{-5k})$
k = 0.042/day

4.3 Domestic wastewater

Domestic wastewater is the liquid component of waste removed from residences, businesses, and institutions. Essentially, water is considered drinking water or potable water before it is used; after it has been used and discarded it becomes wastewater. Domestic wastewater generally includes material disposed of through toilets, sinks, tubs and showers, washing machines etc. Domestic wastewater must be treated and disposed of in a manner that minimizes potential for affecting public health, as well as minimizes the impact on the environment. Table 3.2 shows BOD corresponding COD values and their ratio for domestic waste.

Table 4.4: BOD_5, COD values and ratio for domestic wastewater.

Sample no	BOD_5(mg/l)	COD(mg/l)	BOD_5/COD	Avg. BOD_5/COD
1	286	320	0.76	0.61
2	387	640	0.61	
3	449	640	0.71	
4	457	960	0.47	
5	468	960	0.48	
6	464	640	0.72	
7	439	960	0.45	

4.4 Municipal wastewater

Municipal wastewater is one type of wastewater which contains domestic sewage and some variety of small industries effluent. The composition of waste is quite different than domestic and industrial wastewater. Table 3.3 shows BOD corresponding COD values and their ratio for Municipal waste.

Table 4.5: BOD, COD values and ratio for municipal wastewater.

Sample no	BOD(mg/l)	COD(mg/l)	BOD/COD	Avg. BOD/COD
1	492	960	0.51	0.37
2	424	1280	0.33	
3	519	1280	0.40	
4	557	1600	0.34	
5	600	1920	0.31	
6	562	1600	0.35	
7	582	1600	0.36	

4.5 Industrial wastewater

Industrial processes generate a wide variety of pollutants. The characteristics and level of pollutants vary significantly industry to industry. The environmental protection agency has grouped the pollutants into three categories: conventional pollutants, non conventional pollutants, and priority pollutants. Wastewater generated from industrial activities can contain pollutants such as BOD (biochemical oxygen demand), suspended solids, nutrients, heavy metals, oils and greases, and other toxic organic and inorganic chemicals. If left untreated, these pollutants can cause serious harm to the environment. BOD is the conventional pollutant, and COD is the non conventional pollutant of industrial wastewater. Table 3.4 shows BOD corresponding COD values and their ratio for Industrial waste.

Table 4.6 BOD, COD values and ratio for Industrial wastewater.

Sample no	BOD(mg/l)	COD(mg/l)	BOD/COD	Avg. BOD/COD
1	664	4480	0.14	**0.14**
2	543	3520	0.15	
3	519	3520	0.14	
4	557	3840	0.14	
5	636	4160	0.15	
6	562	3840	0.14	
7	582	3840	0.15	

4.6 Ultimate BOD and BOD rate constant, k

For ultimate BOD, BOD rate constant determination, 7 days BOD data is calculated by least square method and Fujimoto method. the tasvle below shows various values of BODu and BOD rate constant.

Table 4.7: Ultimate BOD, BOD rate constants for different waste water.

Sample type		Least Square Method	Fujimoto Method
Domestic waste water	**BODu(mg/l)**	628.79	609.18
	k(day^{-1})	0.41	0.37
Municipal waste water	**BODu(mg/l)**	1137.53	975.89
	k(day^{-1})	0.14	0.16
Industrial waste water	**BODu(mg/l)**	829.54	816.33
	k(day^{-1})	0.16	0.17

5 Results & discussion

BOD_5 and COD value of KUET sample, SAF sample and khalishpur sample is being plotted in plain graph. The correlation of BOD_5 and COD is obtained from the graph. The trend line implies the relationship pattern. For each plot developed equation and RSQ values are shown. The plots of BOD vs. COD of the samples are shown below:

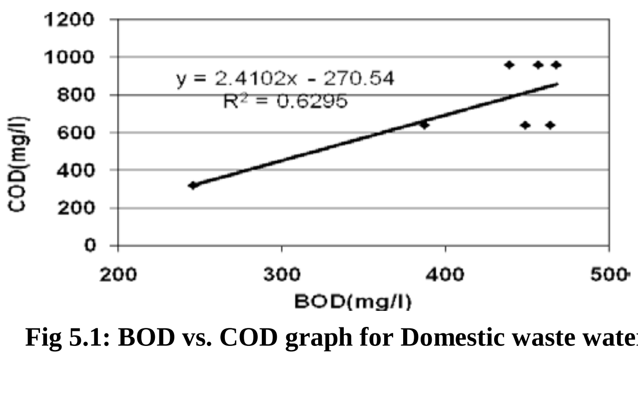

Fig 5.1: BOD vs. COD graph for Domestic waste water.

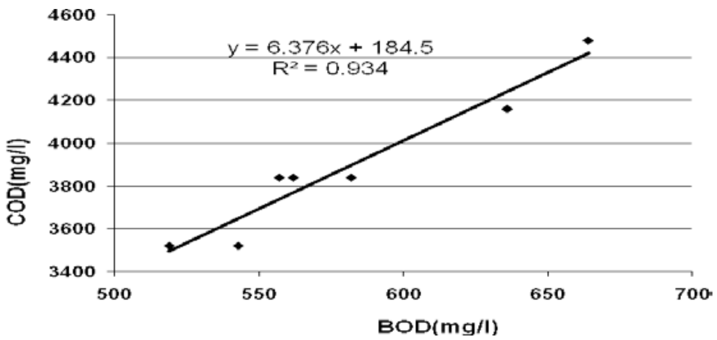

Fig 5.2: BOD vs. COD graph for Industrial waste water.

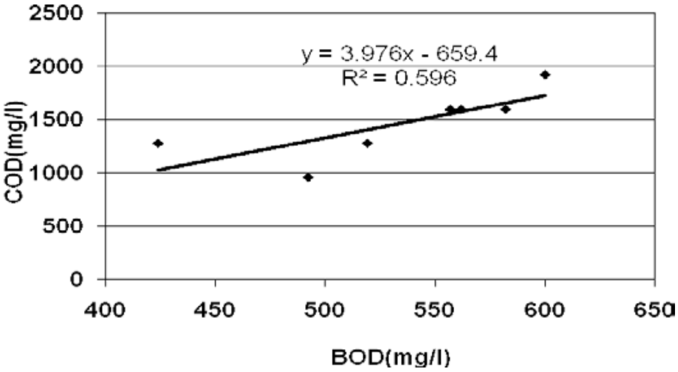

Fig 5.3: BOD vs. COD graph for Municipal wastewater

From figure 5.1, 5.2 and 5.3 it seems that BOD vs. COD curves of three samples are different from each other. The slope is steeper for KUET sample (Domestic Wastewater), the slope of Khalishpur sample (Municipal Wastewater) is less steep, and the slope of SAF Industry sample (Industrial Wastewater) is milder. The slope indicates the ratio of BOD/COD.

Table 5.1: BOD-COD relationship of different wastewater.

	Domestic Wastewater (KUET sample)	**Municipal Wastewater (Khalishpur sample)**	**Industrial Wastewater (SAF sample)**
BOD, COD relation	COD= 2.41BOD - 270.54	COD = 3.97BOD- 659.42	COD = 6.37BOD + 184.59
RSQ value	0.62	0.59	0.93
BOD/COD ratio	0.62	0.38	0.15
Reported value of BOD/COD ratio	0.6	0.48	0.25

(Henry & Heinke, 1978)

From Table 4.6, the ratio of BOD/COD is less deviated from the reported values. And the RSQ values are tends to 1. That means the correlation developed is a good relation. From the BOD/COD ratio it seems that the lowest value is for Industrial wastewater. It means Industrial wastewater is less biodegradable than Municipal and Domestic wastewater.

Table 5.2: BOD kinetic parameters of different waste water.

Sample No.	Domestic Wastewater (KUET sample)			Municipal Wastewater (Khalishpur sample)			Industrial Wastewater (SAF sample)		
	BOD_5 (mg/l)	BOD_u (mg/l)	k (day^{-1})	BOD_5 (mg/l)	BOD_u (mg/l)	k (day^{-1})	BOD_5 (mg/l)	BOD_u (mg/l)	k (day^{-1})
1	514	628.79	0.4161	556	1039.104	0.1626	487	750.158	0.197
2	501	627.94	0.3788	591	1079.973	0.1581	451	854.752	0.1598
3	510	626.29	0.3857	545	1100.65	0.1522	466	799.729	0.1781
4	525	637.28	0.3735	586	1033.482	0.1718	478	819.520	0.1733

The table 4.5 shows the comparison of Least Square method and Fujimoto method for different wastewater. This is quite similar for Municipal, and Industrial wastewater. Table 4.7 shows the values of BODu, k, and BOD5 of different waste water. These values are plotted in graph for BODu and BOD5 relation development.

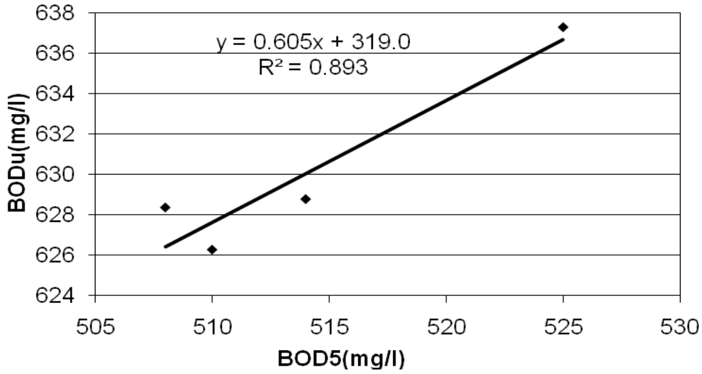

Fig 5.4: BODu Vs. BOD5 graph for Domestic Wastewater

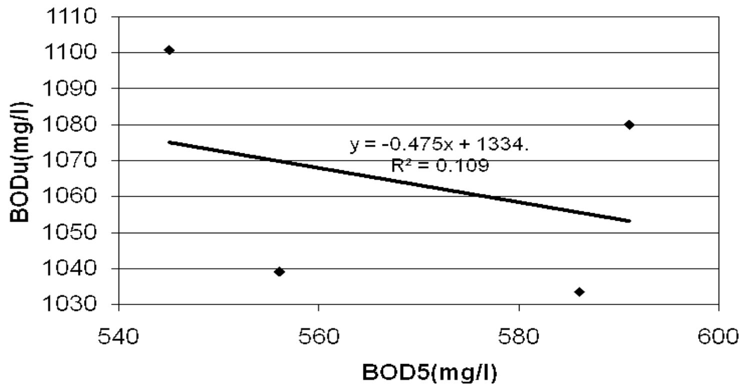

Fig 5.5: BODu Vs. BOD5 graph for Industrial Wastewater

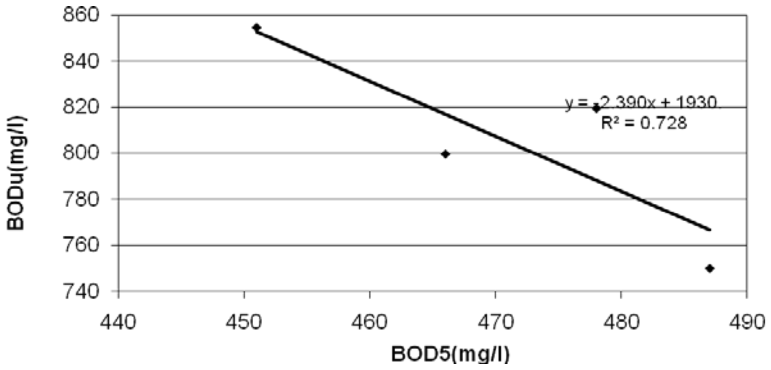

Fig 5.6: BODu Vs. BOD5 graph for Municipal Wastewater

The fig 5.4, 5.5 and 5.6 shows the BODu, BOD_5 relation of three wastewater samples. The relationships are:

Domestic: BODu = 0.605BOD_5 + 319.06

Municipal: BODu = -2.3902 BOD_5 + 1930.6

Industrial: BODu = -0.4759 BOD_5 + 1334.3

The RSQ value of Industrial wastewater (SAF sample) is quite low. More tests will increase the RSQ value of the correlation. But the RSQ values of rest two samples have satisfactory values .That means the correlation of BODu and BOD_5 is well developed.

3. CONCLUSIONS

Methods like COD are quite accurate and take very less time for estimating the organic matter concentration. But they cannot differentiate biodegradable organic matter from non-biodegradable organic matter. Further, COD is not capable of accurately estimating volatile organic matter and organic matter with nitrogen bases. Because of these reasons, BOD is preferred over COD. The correlation of BOD5, COD can be used for determining the waste loading carrying that wastewater. For treatment plant operation it is important to know the waste loading of the effluent. BOD/COD is often measured as a rapid indicator of organic pollutant in water. Its widest application is in measuring waste loadings to treatment plants and in evaluating the efficiency of such treatment systems. The test results imply that BOD/COD value of domestic water is more than the industrial waste. It means industrial waste is less biodegradable than domestic waste. The ratio should increase by treatment for disposal. Ultimate BOD and BOD rate constant is determined by Least squares method and Fujimoto method, because these methods are widely used. The comparison of Least squares method and Fujimoto method sows that, Least Square method gives greater result of BODu and k than Fujimoto method but quite similar to each other.

ACKNOWLEDGEMENTS (OPTIONAL)

The authors tend the best regards and profound compliments to all the teachers of Department of Civil Engineering, KUET, Bangladesh for their cordial support and encouragement.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EVALUTATING THE ADAPTATION APPROACH OF LIVELIHOOD PATTERN IN THE WATER LOGGED AREA OF TALA UPAZILA, SATKHIRA

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ABSTRACT

Now a day's water logging is a burning problem in Bangladesh especially in South-West coastal region. Among South-West(S-W) Coastal Regions most affected district is Satkhira and the study area of this research is Tala Upazila in Satkhira district. The main goal of this study is to focus the present vulnerable condition and to evaluate the adaptation strategies of livelihood pattern in the study area. The data have been collected from primary and secondary sources and the primary data have been collected through questionnaire survey. Peoples in the study area practice alternative adaptation options which minimize their loss and damage caused by water logging. Due to water logging, livelihood options in the study area have been totally changed. The effects of occupational change decrease income level. People in the study area have already adapted with different sectors like agriculture, sanitation, livelihood options, water resource etc. to sustain better livelihood pattern. 65% respondents follow alternative income source to develop their economic conditions. 30% households are fully involved in floating bed agriculture and 80% women are involved in duck rearing. In the year 2000, no one involved in hanging vegetable gardening but now 65% people are involved. 60% respondents are interested in floating bed agriculture in future for their economic development.

Keywords: *Water logging, Adaptation, Livelihood pattern, Environmental impact.*

1. INTRODUCTION

The situation when water remains stagnant for long time due to lack of proper drainage system and creates much adverse impact on daily life is known as water logging. It is one kind of disaster. However, flood and water logging are closely related with each other. Flood is the cause of water logging and water logging is the result of flood. And adaptation is the evolutionary process whereby a population becomes better suited to its habitat. Adaptation is mainly needed in vulnerable condition.

The South-West(S-W) Coastal Region mainly includes the whole of Khulna, **Satkhira** and Bagerhat districts and adjacent southern part of Jessore district. The South - West Costal Region (SWCR) stands out as unique and sensitive in terms of ecology and environment and is the most fertile region in Bangladesh (Tutu, 2004). But in present situation, the most of the water logged area of S- W region is in Khulna, Jessore, and **Satkhira** district. Rapid siltation at the regulators of downstream and poor management of sluice gate, created blockage in the drainage channels which ultimately resulted water logging (Mostafizur, 1995). Water logging in some part of Bangladesh has been also causing an emerging problem for human lives, living condition and affecting income generating function along with changing surroundings over the years. During this situation adaptation is mostly needed in South-West coastal Bangladesh.

There have been several related works on water logging in south-west part of Bangladesh but in Tala upazila, (Dhandia & Sarulia union) the socio-economic and environmental effects and its adaptation due to water logging is scanty. This is why, the work has been undertaken to investigate the present vulnerable conditions and evaluate the adaptation strategy of livelihood pattern in Tala upazila (Dhandia & Sarulia union), Satkhira.

2. METHODOLOGY

Three villages (Barokasipur, Sarulia and Putiakhali) of Sarulia union and two villages (Senergati and Katakhali) of Dhandia union under Tala Upazila of Satkhira District in Bangladesh were considered as study area. Data were collected during the period of February to April 2011. The household was considered as sampling unit and simple random sampling procedure was used for data collection. Out of total 3406 household the sample size was 132, obtained from the following equation (Kothari, 2008).

$$n = \frac{z^2 pqN}{e^2(N-1) + z^2 pq} \qquad (1)$$

[Where, n = Sample size; z^2 = The value of the standard variant at a given confidence level; p = Sample proportion; q = 1-p; e = Acceptance error and N = Population size for the village. When, p =10% of the house hold, *i.e.* 0.1; q = 0.9; z = 1.96 (for 95% confidence level the value of z is 1.96) and e = 0.05 (since the estimate should be within 5% of the true value)].

3. RESULTS AND DISCUSSION

The native impact of water logging on the environment in the study area has been quite far-reaching. The research work has focused on how people themselves see their effects on nature and society and also has focused the adaptation practices which minimize their loss and damage caused by water logging.

3.1Vulnerable Sectors to Water Logging in the study area

Most of the sectors in the study area go by vulnerable situation during water logging time.

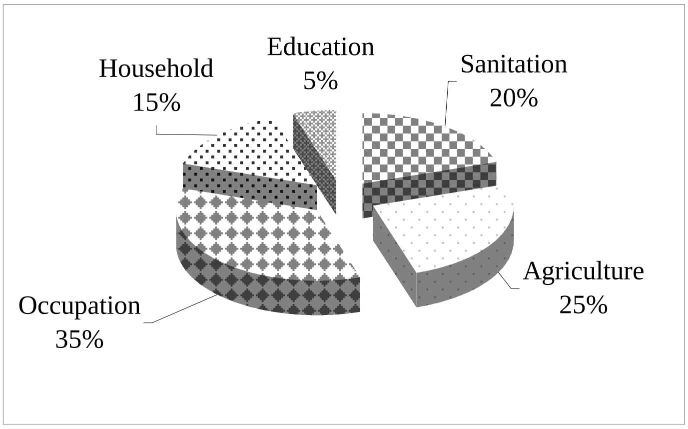

Figure 1: Vulnerable sectors in the study area due to water logging

Figure 1 shows that most of the vulnerable sector is occupation and that is 35%. Another vulnerable sectors are agriculture, sanitation, household, education are 25%, 20%, 15% and 5% respectively.

3.2 Environmental Impacts in the Study Area Caused By Water Logging

3.2.1 Change of cropping pattern

In Rabi season Boro paddy, varieties of pulse grow. In Kharip-1 season jute, Aush paddy grows. In Kharip-2 Amon paddy grows. (Source: Upazila Agricultural office, Tala)

But due to water logging the seasonal crop cannot grow at the accurate time.

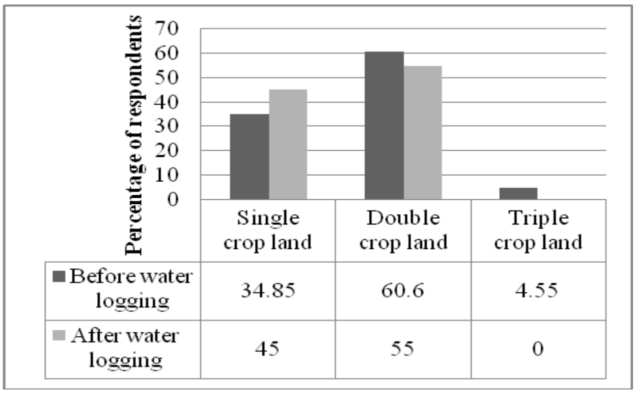

Figure 2: Cropping pattern in the study area

Figure 2 represents that after water logging triple crop land becomes 0% because most of the time land is inundated by water. Rest of the time land grows single and double crops. Double crops grow more because it meets the demand of the people in the study area.

3.2.2 Livelihood options

The people of the study area have various types of livelihood options, such as agriculture, agriculture labor, business, services, others day labor, van puller, fishing, etc. Due to water logging there is a great change in occupational pattern in the study area. Agriculture based work decrease day by day for the present situation. The agriculture and agriculture labor was the main in 2000 and that were 16.67**%** and 40**%** respectively. But now that are 10 %. The new occupation is adding at present situation is fishing. In 2000 people engaged in fishing was 0% but in 2005 and 2010 that is 3.33% and 6% respectively. Another occupation increase in the water logged area that is van pulling. The present percentage of van pulling is 16.67% which was 5.33% before water logging (Table 1).

3.2.3 General income level

Water logging has also a great impact on income level. From the Table 1 it is found that, the income level decreases day by day from 2000 to 2010. In a situation land has lost its utility. Income level <1000 was 3.33% in 2000 but in 2010 this is 20%. <1000 income level increase but another income level such as 1001-5000, 5001-7000, 7001-13000, >13000 decrease from 2000 to 2010 due to lack of proper occupation in the water logged area. This is the general income situation. Peoples in the study area faced this situation for the last 7-8 years. But they also find-out alternative income source to increase their income level and maintain their life better than previous.

3.2.4 Food habit change

The percentages of rice are only 23.33% in 2010 where it was 53.33% in 2000 (Table 1). The rate of vegetable intake is also decreased because low vegetable production. Most of the agricultural and vegetation land are water logged. So the collecting of rice and vegetable are very difficult for the general people. Fish consumption increase because most of the beel and pond are over flowed and fishes are available here and there during water logging condition. But eggs and milk consumption decrease because most of the live stocks are declined due to water logging. So, food condition exists in a very poor condition.

Table 1: Various environmental effects in different years

Category	2000	2005	2010
Livelihood options	Respondents (%)	Respondents (%)	Respondents (%)
Agricultural	16.67	14	10
Agriculture labor	40	10	10
Others day labor	4.67	16.67	20
Van puller	5.33	16.67	16.67
Small business	13.33	12	11.33
Service	2	2.67	2.67
Fishing	0	3.33	6
Others	10	12	8.67
Unemployed	1.33	4.67	4.67
Income level TK/month	Respondents (%)	Respondents (%)	Respondents (%)
<1000	3.33	8.67	20
1001-5000	63.33	68.67	63.33
5001-7000	20	13.33	9.33
7001-13000	10	6.67	5.33
>13000	3.33	2.67	2
Main food items	Respondents (%)	Respondents (%)	Respondents (%)

Rice	53.33	34.00	23.33
Vegetables	19.33	13.33	11.33
Fishes	10.00	33.33	43.33
Eggs	8.67	5.33	1.33
milk	4.67	2.67	.67
Others (muri, bread, gur, banana)	4	11.33	20
Fuel type	Respondents (%)	Respondents (%)	Respondents (%)
Wood	10.00	40.00	66.67
Straw	33.33	16.67	10.00
Cow dung	56.67	36.67	16.67
Others(Kerosene, gas)	0	6.67	6.67
Types of disease	Respondents (%)	Respondents (%)	Respondents (%)
Diarrhoea	23.33	20.00	10.00
Skin diseases	12.00	33.33	63.33
Chicken pox	19.33	6.67	1.33
Typhoid or fever	30.00	33.33	20.67
Viral eye diseases	10.00	3.33	1.33
Others	5.33	3.33	3.33
Water source	Respondents (%)	Respondents (%)	Respondents (%)
Tube well	50.00	25.00	20.00
Deep tube well	0	35.00	40.00
Pond	25.50	35.00	35.00
River	24.50	5.00	5.00
Sanitation condition	Respondents (%)	Respondents (%)	Respondents (%)
Hanging latrine	10.00	20.00	23.33
Pit latrine	32.67	36.67	43.33
Open tank latrine	20.67	17.33	13.33
Septic tank latrine	16.67	9.33	2.67
Open space latrine	9.33	6.00	6.67
Others latrine	10.67	10.66	10.67

3.2.5 Energy Consumption Pattern

Water logging has severely affected the indigenous sources of fuel in the study area. The social forestry around the homestead, bushes and paddy straws has been affected. These are valuable sources of domestic fuel. The number of cows and bullocks has declined. As a result, the supply of cow-dung, another important source of fuel has also dried up. About 56.67% of the households used cow dung as fuel before water logging but in 2010 it is 16.67% (Table 1). In 2010, 10% of the total households' use straw but in 2000 and 2005 it was 33.33% and 16.67% respectively. About 6.67% of households use kerosene, gas as fuel in 2010 but in 2000 it was 0. The 66.67% household use wood in 2010 because any other source of fuel is decreasing for the effect of water logging. So the pressure on wood is increasing day by day and the diversity of trees has decreased in the study area now at present condition.

3.2.6 Health condition

Water logging carried out different types of diseases in the study area. Public health situation in the study area has been deteriorating due to unhygienic condition caused by water logging. There has been a substantial increase in the incidence of various water borne diseases in the study area. In the year of 2010, 63.33% of the respondents in the affected area said that there has been an increased incidence of skin disease (Table 1) during and after water logging period. Typhoid/fever has been identified as the second most important disease by majority of the respondents in the affected area. For the lack of economic solvency the proper treatment cannot be done, so the people are suffering from these diseases for a long time.

3. 2.7 Water source

The surface water bodies of the study area become poisonous due to lack of circulation. The color of the water of the ponds, river has also changed and it smells foul. Table 1 represents that 40% people depend on deep tube well for their domestic purposes after water logging in 2010. But in 2000 the used rate of deep tube well was 0%. In 2000 people used pond, river and tube well were 25.50%, 24.50% and 50% respectively. Dependency decrease on tube well and river as a source of using water because most of the water body is mixed together and water becomes polluted.

3.2.8 Sanitation condition

Most of the households of water logged area use pit latrine and hanging latrine in present condition because different NGO's provide sanitation materials to the water logged area. By those materials the people prepare pit latrine. From (Table 1) it is obtained that the hanging latrine was 10% in 2000 but 2010 it is 23.33% because of the water logging condition. 13.33 % people use open space. Septic tank latrine was used by 16.67% people but in 2010 it is 2.67% because most of the septic latrine was destroyed due to water logging. People's income level decrease day by day, for this they cannot repair their latrine again.

3.3 Adaptation in Different Sector

Local communities (both indigenous and non-indigenous) have been facing common natural hazards of the area including floods, river erosion, droughts, salinity and cyclones for thousands of years. The local people developed many adaptation techniques, innovations and knowledge to address climatic vulnerability.

3.3.1 Adaptation in livelihood pattern

Most of the respondents of the study area are changed their occupational pattern due to water logging. They want to increase their income from any kind of activities. For this they follow some alternative income source during water logging period.

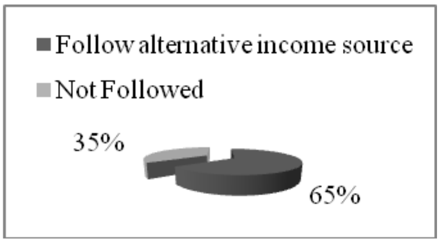

Figure 3: Respondents adaptation in alternative income source

Figure 3 shows 65% respondents are adapted in alternative income source to develop their economic conditions during water logging time. For this purpose they practice some adaptation strategies. These are follows:

a) Adaptation in floating bed agriculture

The floating agriculture practice is a good popular practice of the low-lying areas where lands remain submerged most of time in a year. In the winter season, farmers carry the floating bed comparatively in the high land and break the bed to mix with the soil of the land. As a result, water logging and flooding have been alarmingly increasing day by day. If Soil less agriculture which is called floating bed agriculture can disseminate to different part of Bangladesh then policy maker would change their attitude. In these connections, we need to change our terrestrial agriculture base approach. We have to consider about the resources of wetlands and need national management plan of it based on Soil-less agriculture (Ghosal and Haq, 2000).

The study area remains submerged with a long period, almost 7-8 months, in the year, especially in the monsoon season. People of this village largely depend on agriculture. People of this village largely depend on agriculture. For this reasons they have already adapted with floating bed agriculture.

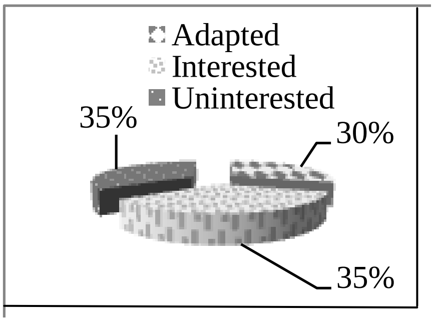

Figure 4: Respondents adaptation in floating bed agriculture

Figure 4 shows that 30% households are fully involved (fully adapted) in floating bed agriculture and 35% are interested to involve in this sector but rest of the portion are not interested because they have poor knowledge about it and most of them are landless poor.

b) Adaptation in duck rearing

Due to water logging most of the land area is submerged under water and it is rarely possible to maintain livestock in the study area. During this situation introduction of duck rearing provides an excellent initiative in the study area because ducks easily travel within water whereas livestock's are not capable to do this. Mainly women are involved in this activity.

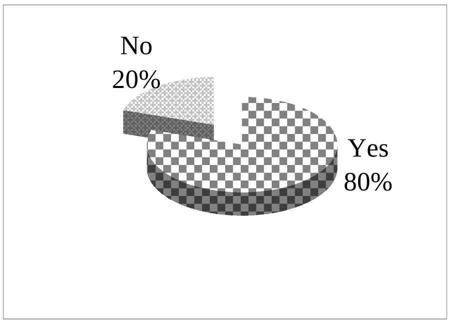

Figure 5: Women's adaptation in duck rearing

Figure 5 shows that 80% women are adapted in duck rearing and only 20% are not. Most of the people in the study area are poor and they have little opportunity to earn money. The women of those families want to help their earning member by any means. So they (women) are adapted in this activity.

c) Adaptation in hanging vegetable cultivation

Farmers in Bangladesh are growing salt-resistant crops to deal with the growing short age of freshwater and also adapting HVG (Hanging Vegetable Gardening) in the waterlogged areas. So, for climate change adaptation strategy, action research on Hanging Vegetable Gardening (HVG) is urgently needed.

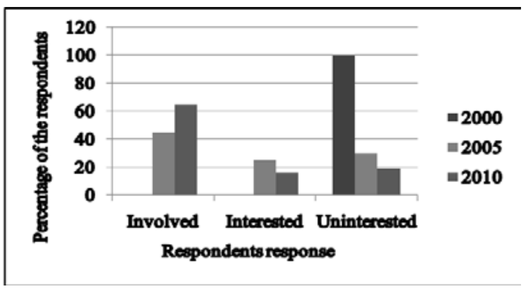

Figure 6: Respondents adaptation in hanging vegetables cultivation

Figure 6 shows that in 2000 no one involved in hanging vegetable cultivation because before 2000 people in the study area were totally unfamiliar about water logging. But now around 6-7 months household's yards are covered by water. For this, they find out hanging vegetable gardening as alternative source of vegetable cultivation and this way they try to meet their needs.

3.3.2 Adaptation in food habit change

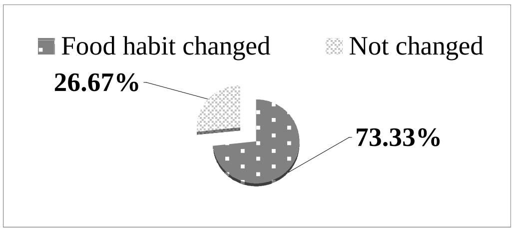

Figure 7: Adaptation in food habit due to water logging

Figure 7 represents that the 73.33% household members agreed about their food habit change and only 26.67% respondents disagreed with the changing. These 26.67% households are higher income group and they are capable to maintain their previous food habit.

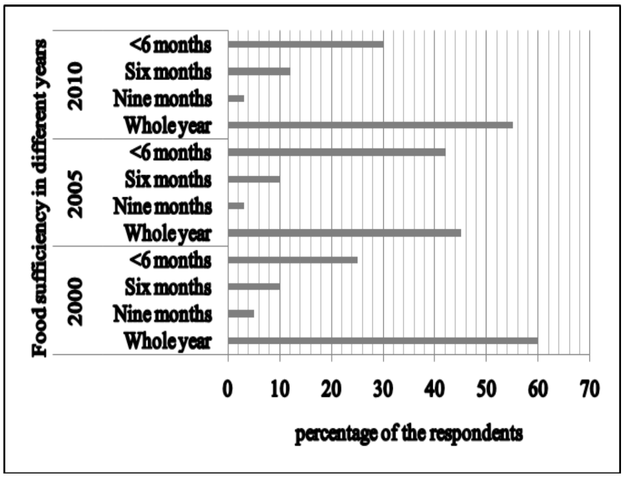

Figure 8: Food sufficiency in different years

Figure 8 represents that adequate food was exist in the study area at past. It was decreased in 2005 due to water logging. But recently it is increasing because of alternative production sources.

3.3.3 Adaptation in water source

People in the study area have already adapted with water source in the study area.

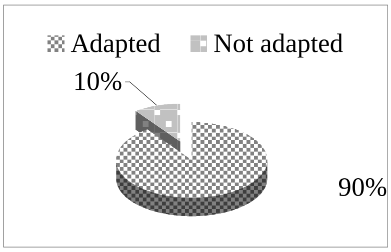

Figure 9: Respondents adaptation in water source

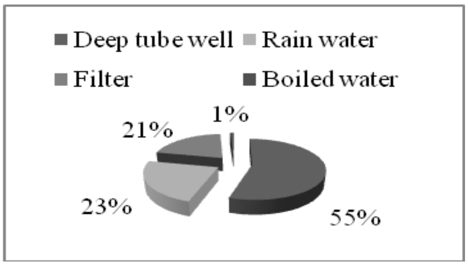

Figure 10: Alternative water sources in present conditions

Figure 9 represents 90% respondents are adapted with water sector and 10% are not. People in the study area find out alternative water source for their drinking and domestic purposes. Figure 10 shows 55% people use deep tube well and only 1% people use boiling water. 23% people consume rain water and among them some respondents told they use rain water mainly cooking and washing purposes.

3.3.4 Adaptation in cooking pattern

Major portion of population in the study area are poor. Their occupations mainly depend on agriculture. But due to water logging agricultural activities have about to stop. Live stocks also decrease due to water logging. Their energy consumption pattern which was mainly depending on straw and cow dung was decrease due to water logging. Some people use gas, kerosin as alternative energy source but they are fewer in number. For this reason, use of wood increase to prepare their meal. People also make improved efficient cooking stoves which are transferable from one place to another place during water logging season and wood is used in this stove.

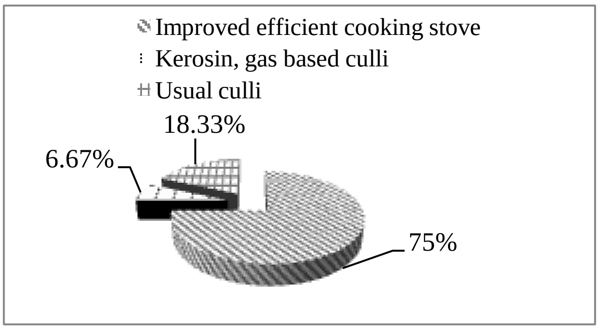

Figure 11: Adaptation in alternative energy consumption stove

Figure 11 shows that 75% respondents use improved efficient cooking stove and only 18.33% use general culli. They make improved efficient cooking stove to adapt with water logging because they can easily transfer it from one place to another place during water logging time.

3.3.5 Adaptation in housing pattern

Most of the people in the study area change their housing pattern due to water logging. They change their housing pattern because they need a safety shelter.

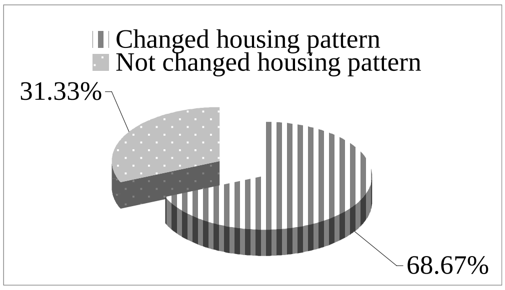

Figure 12: Adaptation in housing pattern due to water logging

Figure 12 represents that the 68.67% households have changed their housing condition for different causes but here water logging is the main cause. 31.33% households have not changed their housing condition.

3.3.6 Adaptation in transportation

Transport is an integral part of the functioning of the society. Main transportation system in the study area are bus, truck, van, cycle, motorcycle, nocimon (one kind of van) etc. The main communication system inundated under water within 6 to 7 months. Sometimes van and boat travel on the road mutually. During this time the main using mode is boat in village area.

3.4 Future Potential Adaptation Strategies

The south-west coastal region of Bangladesh is an active part of Ganges Delta formed by fine clay soil (Flubic) carried in by the upstream flows which are very fertile and rich in biological diversity. The livelihood of the people of this region is highly dependent on the natural resources. Agriculture and fisheries are important economic sectors employing a large portion of the population. Major Agricultural crops include rice, betel and vegetables, mustered and oil seed. But hundreds of thousands of people became unemployed as a result of the water logging, especially the poor and marginal. Some future potential adaptation strategies are mentioned here which will be effective for adaptation.

- Alternative farming systems ,

- Enhance short duration crop varieties
- Re-excavation of choked drainage canals,
- Integrated poultry-fish farming
- Duck-fish farming,
- Hanging vegetable cultivation
- Case aquaculture etc.

3.5 Potential Trend of Livelihood Options in Future

South-West coastal region is mostly vulnerable to water logging. The people in this area always face horrific impacts of water logging. Yet, they find everyday few alternative options to survive with these horrific impacts of water logging. Respondents in the study area choice some future livelihood option as their capability.

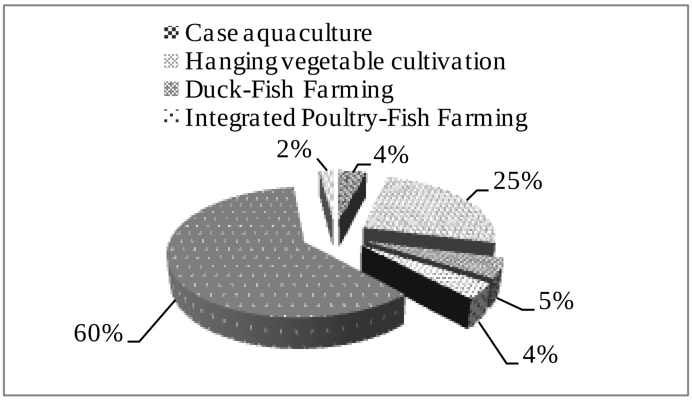

Figure 13: Respondents choice of livelihood option in future

Figure 13 shows that 60% respondents are interested in floating bed agriculture. They have same opinion that they will earn more money from this farming.

3.6 Organizations Involved for Adaptation

Different organizations like GO, NGO (Uttaran, Asa, Setu, SUS etc.) are involved for adaptation in the study area. But most of the respondents in the study area told that mainly NGO's had provided subsidy for their adaptation. These NGO's provide sanitation materials, construction materials of house, water purification filter, low interest loan etc. for their adaptation.

3.7 Barriers in Adaptation

Adaptation is a process by which people minimize their existing effects in daily life. But there are many barriers in adaptation process. Some common barriers are mentioned bellow:

a) Lack of Knowledge
b) Lack of willingness
c) Lack of co-operation between GO and NGO
d) Lack of communication between sufferer and donor group
e) Unavailability of subsidy
f) Corruption etc.

4. CONCLUSION

The South-western regions of Bangladesh are more vulnerable to water logging. From the overall study it has been found that the social and economic conditions have changed due to the impact of water logging problems. The livelihood options have changed and its effect is going on the whole economy of the study area. But people in the study area have already adapted with different sectors like agriculture, sanitation, livelihood options, water source, energy consumption pattern, housing pattern, food habit etc. to minimize their existing problems and to maintain better livelihood pattern.

ACKNOWLEDGEMENT

The authors are grateful to the officials of BWDB, LGED, Upazila Statistical Office of Tala Upazila, Union Parishad of Sarulia and Dhandia Union and respondents of Sarulia and Dhandia Union who spend their valuable time for me during data collection and provided useful information.

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Assessment of Heavy Metal Contamination and Sediment Quality in the Turag River of Bangladesh on the basis of Ecological Risk Index

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ABSTRACT

The environment of Bangladesh is being degraded day by day with the weight of overcrowded people. Untreated and poorly treated effluents from the industries are severely polluting the water courses of Bangladesh. One of such example is the Turag River. Turag River which encompasses north western periphery of Dhaka city receives uncontrolled discharge of pollution, solid waste, sewage and waste water discharged from domestic, commercial and industrial activities of the city. These waste materials are tremendously deteriorating water and sediment quality of the river. The contamination level reaches at such an alarming stage that not only the ecosystem of the river collapse but also the treatment plants of the city based on the river water virtually losses their expected quality of production. Hence the restore of the water quality of the Turag is crucial for its sustainability. In this study concentration of five heavy metals (Pb, Cd, Cr, Cu, and Zn) for sediment from Turag River and for the portion of sediment which passing through the #200 sieve were determined to evaluate their levels and spatial distribution. To assess metal contamination in sediment, US environmental Protection Agency's (USEPA) guidelines were applied. The concentration of Zn, Cr in sediment sample is fall in the criteria of moderately polluted range. Pb, Cd is not in polluted range, on the other hand Cu is highly polluted. It is shown in this study that standard deviation between the concentration of metals at different sites along the Turag river are not significant that may indicate the spatial distribution of metal contamination is almost uniform. The metals contaminations in sediment are also evaluated by applying Pearson's correlation, ecological risk index proposed by Lars Hakanson (1980), contamination factor and degree of contamination proposed by Tomlinson et al. (1980), pollution load index by Wilson and Jeffrey (1987) and mean sediment quality guideline quotient by MacDonald et al. (1998).

Keywords: *Sediment, heavy metal, ecological risk index, contamination factor, pollution load index.*

1. INTRODUCTION

The sediment of the Turag River is increasingly being polluted with the cities thousands of industrial units and sewerage lines dumping huge volumes of toxic wastes which contain lots of heavy metal into it day and night. Heavy metals contamination in aquatic environment is of critical concern, due to toxicity of metals and their accumulation in aquatic habitats. Trace metals in contrast to most pollutant, not biodegradable and they undergo a global ecological cycle in which natural water are the main path ways. Of the chemical pollutants heavy metal being non- biodegradable, they can be concentrated along the food chain, producing their toxic effects at points after far removed from the sources of pollution. (Tilzer and Khondker, 1993). BIWTA interested to clean the polluted sediments of the Turag river bed.

In this study attempts have been taken to estimate the heavy metal contamination of the dredged material and its effect on environment, highlighting the relationship among metals and to compare the data analysis according to different national and international standards.

2. METHODOLOGY

There were five different points in the river basing on the effect of industrial waste for sediment quality analysis. Those are shown in table 2.1.

Table 2.1: Study Area for Sediment Sample Analysis

Station no.	Location	Latitude	Longitude
1	Tongi bridge	23*52'56"N	90*24'3.52"E
2	Ijtema field	23*50'9.2"N	90*22'15.33"E
3	Kamarpara bridge	23*54'28.67"N	90*25'23.71"E
4	Taltola bridge	23*51'37.56"N	90*23'4.48"E
5	Beribadh	23*53'25"N	90*21'32.35"E

Source: Google earth

2.1 Metal Analysis Methods

In choosing the most appropriate analytical method to determine metals, each laboratory must consider the sample type and concentration levels, the number of elements to be determined and the costs the choice implies. As a result flame and graphite furnace atomic absorption spectrometry (AA) and inductively coupled plasma (ICP and ICP-MS) emission spectrometry were used analytical methods for determining trace elements. (Saha P.K. and Hossain, M.D.,2011)

2.2 Toxicity Characteristics Leaching Procedure Test

Toxicity Characteristic Leaching Procedure is designed to determine the mobility of both organic and inorganic analysts present in liquid, solid and multiphasic wastes. This is usually used to determine if a waste may meet the definition of EP Toxicity, that is, carrying a hazardous waste code under RCRA (40 CFR Part 261) of D004 through D052. Sometimes in cleanup actions, businesses are often asked to perform an analysis on their waste using the TCLP, The Code of Federal Regulations (CFR). (Davis, Sherry,2001).

The TCLP test is the assay prescribed by the EPA to determine whether a solid waste is hazardous by toxicity characteristic. The TCLP test involves the extraction of contaminants from a 100-g size-reduced sample of waste material with an appropriate extraction fluid. A 20:1 liquid to solid (L/S) ratio (mass/mass, m/m) is employed, and the mixture is rotated for 18 ± 2 hr at 30 rpm using a rotary agitation apparatus. Where volatile constituents must be evaluated, a smaller sample mass is used (25 g) and a specialized zero-headspace extraction (ZHE) vessel is employed.

Table 2.2: Maximum Concentration of Contaminants for Toxicity Characteristic (the D List) (Method USEPA 1311)

EPA Hazardous Waste code	**Contaminant**	**Regulated Level (mg/l)**
D006	Cadmium (Cd)	1.0
D007	Chromium (Cr)	5.0
D008	Lead (Pb)	5.0
D009	Mercury (Hg)	0.2
D010	Selenium (Se)	1.0
D011	Silver (Ag)	5.0

2.3 Pearson's Correlation

Pearson's Correlation coefficient between two variables is defined as the covariance of the variables divided by the product of their standard deviations. The formula for the Pearson correlation:

$$\rho_{X,Y} = \frac{\mathrm{cov}(X,Y)}{\sigma_X \sigma_Y} = \frac{E[(X-\mu_X)(Y-\mu_Y)]}{\sigma_X \sigma_Y},$$

The above formula defines the population correlation coefficient, commonly represented by the Greek letter ρ (rho). Substituting estimates of the covariance's and variances based on a sample gives the sample correlation coefficient, commonly denoted r:

$$r = \frac{\sum_{i=1}^{n}(X_i - \bar{X})(Y_i - \bar{Y})}{\sqrt{\sum_{i=1}^{n}(X_i - \bar{X})^2}\sqrt{\sum_{i=1}^{n}(Y_i - \bar{Y})^2}}.$$

An equivalent expression gives the correlation coefficient as the mean of the products of the standard scores. Based on a sample of paired data (X_i, Y_i), the sample Pearson correlation coefficient is

$$r = \frac{1}{n-1}\sum_{i=1}^{n}\left(\frac{X_i - \bar{X}}{s_X}\right)\left(\frac{Y_i - \bar{Y}}{s_Y}\right)$$

Where,

$\frac{X_i - \bar{X}}{s_X}, \bar{X},$ and s_X …………………………………… (1) are the standard score, sample mean, and sample standard deviation, respectively.

The Correlation coefficient ranges from -1 to 1.A value of 1 describes the relationship between X and Y perfectly, with all data points lying on a line for which Y increases as X decreases. A value of -1 implies that all data points lie on a line for which Y decreases as X increases.A value of Zero implies that there is no linear relation between the variables.

Table 2.3: Range of correlation

Correlation	Negative	Positive
None	−0.09 to 0.0	0.0 to 0.09
Small	−0.3 to −0.1	0.1 to 0.3
Medium	−0.5 to −0.3	0.3 to 0.5
Strong	−1.0 to −0.5	0.5 to 1.0

2.4 Technical Basis of Test

In developing the original toxicity characteristic, EPA designed the EP based upon a "mismanagement scenario" in which potentially hazardous wastes would be co-disposed with municipal solid waste (MSW) in a landfill with actively decomposing material overlying an aquifer. Consistent with this mismanagement scenario, the EP required that a liquid extract be obtained from solid waste (following particle size reduction, if necessary) by exposing the waste to organic acids (the acids likely to be found in a landfill containing decomposing municipal wastes). In conjunction with the co-disposal scenario, EPA assumed that the most likely pathway for human exposure to toxic constituents would be through drinking water contaminated by leachate from the landfill.

Analyses of the EP extract were to be compared to the National Interim Primary Drinking Water Standards (NIPDWS). To account for the likely dilution and attenuation of the toxic constituents that would occur as they traveled from the landfill to a drinking water source, regulatory limits were established by multiplying the NIPDWS by a "dilution and attenuation factor" (DAF) of 100. The DAF of 100 was not derived from any model or empirical data, but rather was an estimated factor that EPA believed would indicate substantial hazard.The standards for the sediment quality parameters are shown in table 2.4.

Table 2.4: US EPA sediment quality guideline

(mg/kg) dry weights	Zn	Pb	Cu	Cd	Cr
US EPA sediment quality guideline					
Not Polluted	<90	<40	<25	-	<25
Moderately Polluted	90-200	40-60	25-50	-	25-75
Highly Polluted	>200	>60	>50	>6	>75

2.5 Potential Ecological Risk Index (PERI)

The Potential Ecological Risk Index (PERI) is a diagnostic tool suggested by Hakanson (1980, 1988) for contamination control of lakes and coastal systems.PERI is formed by three basic modules: Degree of contamination(C_D); toxic-response factor (Tr^1)*;* and potential ecological risk factor(Er^i).

Its main function is to indicate the contaminant agents and where contamination studies should prioritized.

2.5.1 Pollution load index (PLI)

PLI is an index to assess the extent of pollution by metals in estuarine sediments. According to Tomlinson et.al (1980) sediment pollution load index is calculated by using the equation:

$PLI = (CF_1+CF_2+CF_3+\ldots\ldots\ldots CF_n)^{\wedge(1/n)}$

Where, CF=Contamination factor

$=C_{metal}/C_{background}$ If PLI>1, indicates pollution

PLI<1, indicates no pollution

2.5.2 Mean sediment quality guideline quotient (SQG-Q)

It evaluates the need for detailed sediment quality assessments, assessing the quality of prospective dredged materials, conducting remedial investigations and ecological risk assessments, and developing sediment quality remediation objectives (Long and MacDonald 1998). The consensus-based SQGs were critically evaluated to determine if they would provide effective tools for assessing sediment quality conditions in fresh water ecosystems. Table 2.5 shows the Biological effect at different range of (SQG-Q)

Table 2.5: Biological effect at different range of (SQG-Q)

Range of Q	Impact Level
$Q \leq 0.1$	Unimpacted and lowest potential for observing adverse biological effects
0.1<SQG-Q<1	Moderate impact potential for observing adverse biological effects
$SQG\text{-}Q \geq 1$	Highly impacted potential for observing adverse biological effect

3. ILLUSTRATIONS

3.1 Metal Analysis

The concentration of metals in surface sediment has the trend Zinc (Zn)>Copper (Cu)>Chromium (Cr)> Lead (Pb)> Cadmium (Cd).

Table 3.1: Concentration of heavy metals (mg/kg dry weight) of sediments of Turag River

Location	Lead (Pb)	Cadmium (Cd)	Chromium (Cr)	Copper (Cu)	Zinc (Zn)
T-1	36.40	0.10	36.00	60.00	179.30
T-2	34.40	0.10	33.50	46.30	113.80
T-3	30.40	0.00	75.50	46.60	190.10
T-4	28.30	0.40	32.00	50.00	94.60
T-5	34.40	0.80	38.10	49.30	119.60
Mean	32.78	0.28	43.02	50.40	139.48
Max	36.40	0.80	75.50	60.00	190.10
Min	28.30	0.00	32.00	46.30	94.60
SD	3.32	0.33	18.31	5.62	42.48
Background concentration (Mertin and Meybeck, 1979)	20.00	0.20	97.00	32.00	129.00

3.2 Metal Analysis after TCLP Test

In this study, heavy metal concentrations from leachate of Turag River were tested in the laboratory and pollution levels were assessed with comparison of US EPA standard. Results of the TCLP test are presented in the Table 3.2.

Table 3.2: TCLP test result (mg/L) for the sediment sample of the Turag River (Method USEPA 1311)

Location	Pb	Cd	Cr	Cu	Zn
T-1	0.09	0.02	0.00	0.12	3.10
T-2	0.08	0.01	0.02	0.18	2.33
T-3	0.04	0.00	0.00	0.18	3.77
T-4	0.01	0.00	0.00	0.13	0.61
T-5	0.01	0.00	0.01	0.12	1.95
Mean	0.04	0.01	0.01	0.15	2.35
Max	0.09	0.02	0.02	0.18	3.77
Min	0.01	0.00	0.00	0.12	0.61
SD	0.04	0.01	0.01	0.03	1.20
Regulated Level (USEPA)	5.00	1.00	5.00	-	-

Results of TCLP test for Turag River shows that, concentrations of heavy metal in the leachate are not exceeded the permissible US EPA standard. That indicate regarding the readily toxicity pollution by heavy metal, Turag river sediment condition is not at the severe state.

3.3 Pearson's Correlation

Pearson's correlation coefficient matrix among the selected heavy metals of Turag River sediments is presented in Table 3.3. Significant correlations between the contaminations of Cr and Zn (r=0.71), Pb and Cu (r =0.50), Zn and Cu (r =0.34) could indicate the same or similar source input.

Table 3.3: Correlation matrix between heavy metals in sediment samples from Turag River

	Pb	**Cd**	**Cr**	**Cu**	**Zn**
Pb	1				
Cd	0.01	1			
Cr	-0.32	-0.42	1		
Cu	0.50	-0.07	-0.36	1	
Zn	0.27	-0.58	0.71	0.34	1

3.4 Heavy Metal Contamination and US EPA Quality Guideline

In absence of any local standards for pollutants, the metal levels in sediment sample were compared with the sediment quality guideline proposed by United States EPA. These criteria are shown in Table 3.4.

Table 3.4: Comparison between US EPA sediment quality guideline and present study

(mg/kg dry weights)	**Zn**	**Pb**	**Cu**	**Cd**	**Cr**
US EPA sediment quality guideline					
Not Polluted	<90	<40	<25	-	<25
Moderately Polluted	90-200	40-60	25-50	-	25-75
Highly Polluted	>200	>60	>50	>6	>75
Present Study					
Turag river	94.60-190.10	28.30-36.40	46.30-60.00	0.00-0.80	32.00-75.50

3.4.1 Variation of Cu

Here table 3.5 and figure 3.1 show that only four locations along Turag river are moderately polluted and Tongi bridge is highly polluted with Cu.

Table 3.5: Variation of Copper along Turag River in comparison to US EPA

Location	**Cu (mg/kg)**	**US EPA standard**
Tongi bridge	60	Highly polluted
Ijtema field	46.3	Moderately polluted
Kamarpara	46.4	Moderately polluted
Taltola bridge	48.7	Moderately polluted
Beribadh	49.3	Moderately polluted

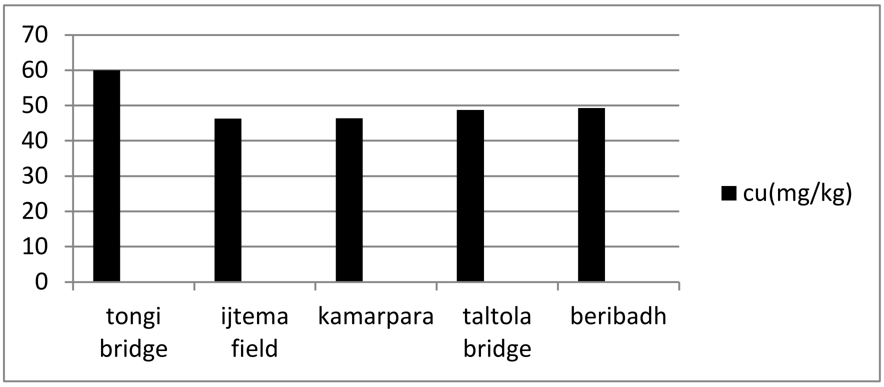

Figure 3.1: Variation of Copper along Turag River

3.4.2 Variation of Cr

From table 3.6 and fig. 3.2 it is seen that almost all the locations of the Turag river are moderately polluted. Only one point along Turag river is highly polluted.

Table 3.6: Variation of Chromium along five points of the Turag River in comparison to US EPA

Location	Cr(mg/kg)	US EPA standard
Tongi bridge	36	Moderately polluted
Ijtema field	33.5	Moderately polluted
Kamarpara	75.5	Highly polluted
Taltola	32	Moderately polluted
Beribadh	38.1	Moderately Polluted

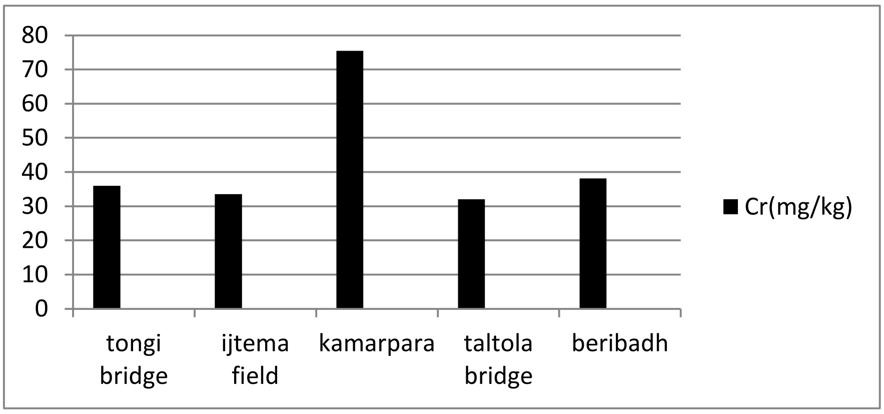

Figure 3.2: Variation of chromium along Turag river

3.4.3 Variation of Zn

From table 3.7 and fig. 3.3 it is seen that all the points along the Turag River are moderately polluted.

Table 3.7: Variation of Zinc along Turag River in comparison to US EPA

Location	Zn(mg/kg)	US EPA standard
Tongi bridge	179.3	Moderately polluted
Ijtema field	113.8	Moderately polluted
Kamarpara	190.1	Moderately polluted
Taltola	94.6	Moderately polluted
Beribadh	119.6	Moderately Polluted

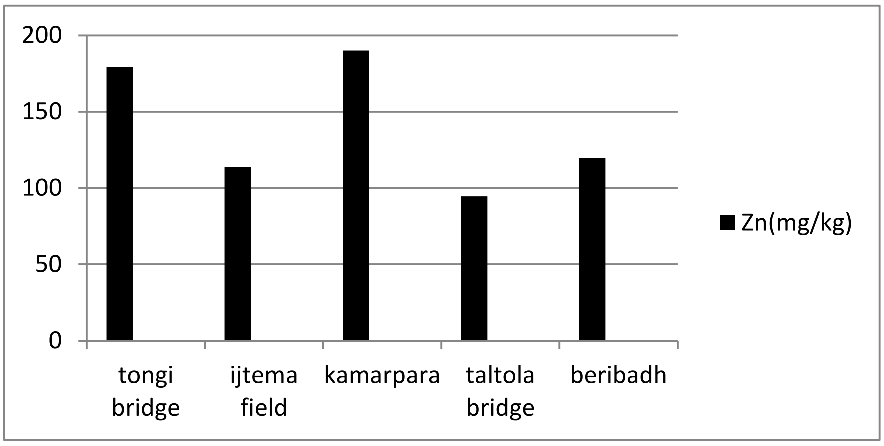

Figure 3.3: Variation of zinc along Turag River

3.4.4 Variation of Cd

There is no limit of US EPA sediment quality guideline for unpolluted and moderately polluted with cadmium. But Table 3.8 and fig. 3.4 shows that only one location along Turag River is highly polluted with cadmium according to the guideline.

Table 3.8: Variation of cadmium along Turag river in comparison to US EPA

Location	Cd(mg/kg)	US EPA standard
Tongi bridge	0.1	-
Ijtema field	0.1	-
Kamarpara	0.0	-
Taltola	0.4	-
Beribadh	0.8	Highly Polluted

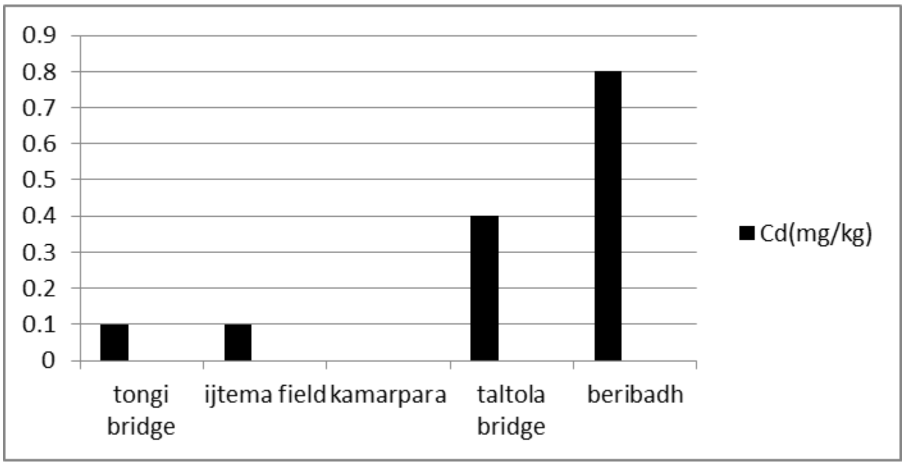

Figure 3.4: Variation of cadmium along Turag river

3.4.5 Variation of Pb

Table 3.9 and fig. 3.5 shows that all the five locations along Turag are not polluted for lead.

Table 3.9: Variation of Lead along Turag River in comparison to US EPA

Location	Pb(mg/kg)	US EPA standard
Tongi bridge	36.4	Not polluted
Ijtema field	34.4	Not polluted
Kamarpara	30.4	Not polluted
Taltola	28.3	Not polluted
Beribadh	34.4	Not polluted

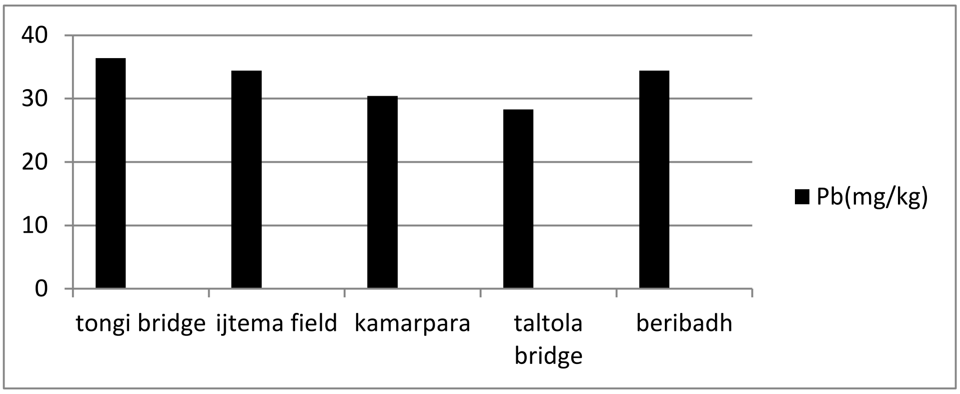

Figure 3.5: Variation of lead along Turag River

3.5 Ecological risk indices calculation

Ecological risk assessment is defined as a process that evaluates the likelihood that adverse ecological effects may occur or are occurring as a result of exposure to one or more stressors.
In this study potential ecological risk index proposed by Lars Hakanson (1980), contamination factor and degree of contamination proposed by Tomlinson *et al*. (1980), pollution load index by Wilson and Jeffrey (1987) and mean sediment quality guideline quotient by MacDonald *et al*. (1998) was calculated.

3.5.1 Potential ecological risk index

A potential ecological risk assessment was conducted based on the analyzing results of heavy metal (Pb, Cu, Cr, Cd and Zn) contents in the sediments of Turag river using the index number techniques of single factor and Hakanson method for the quality status of the sediments. is shown in table 3.10.

Table 3.10: Ecological risk index of Turag river

Location	**Zn**	**Cd**	**Cr**	**Cu**	**Pb**	**RI**	**Grade Hakanson (1980)**
	$C^i_f \times T^i_f$	$C^i_f \times T^i_f$	$C^i_f \times T^i_f$	$C^i_f \times T^i_f$	$C^i_f \times T^i_f$		
T-1	1.39	15.0	0.74	9.38	9.10	35.61	Low
T-2	0.88	15.0	0.69	7.23	8.60	32.41	Low
T-3	1.47	0.0	1.56	7.25	7.60	17.88	Low
T-4	0.73	60.0	0.66	7.81	7.08	76.28	Low
T-5	0.93	120.0	0.79	7.70	8.60	138.02	Moderate

From Table 3.10, it can be seen that the order of potential risk, factor of heavy metals in sediments of the Turag river is Cd> Cu> Pb> Zn> Cr; Cd is the most important one and its risk factor is upto the appreciable grade. The results indicate that the range of Cd pollution in Turag river is low to appreciable. Other heavy metals (Zn, Cr, Cu and Pb) have low potential ecological risk. This index range of potential risk is from 17.88 to 138.02, and the average index of potential ecological risk factors (RI) is 60.04. Turag river has a low to moderate potential ecological risk due to heavy metal contamination.

3.5.2 Contamination factor and degree of contamination

The contamination factor and degree of contamination were used to determine the contamination status of sediment in the present study. Contamination factor was calculated according to Tomlinson *et al.*(1980). Contamination factor, pollution load index and degree of contamination proposed by Tomlinson *et al.*(1980) were applied for Turag river sediments.
Table 3.11 shows contamination factor, pollution load index and degree of contamination at different locations along Turag river. All locations are moderately contaminated with lead.T-1, T-2 and T-3 are less contaminated, T-4 is moderately contaminated and T-5 is considerably contaminated with Cadmium. All locations have low contamination due to chromium and moderate contamination for Copper. T-1 and T-3 are moderately contaminated; T-2, T-4 and T-5 are less contaminated with Zinc.
As per pollution load index, T-4 and T-5 locations sediment are polluted and T-1, T-2and T-3 are not polluted. Highest pollution load index value is observed at T-5 and minimum at T-3. Degree of contamination values range from 4.89 to 8.58. Maximum value of degree of contamination was found at T-5 and minimum at T-2. As per degree of contamination it is found that T-4 and T-5 are the locations of moderate contamination and T-1, T-2 and T-3 are the locations of low contamination.

Table 3.11 pollution load index and degree of contamination of Turag river sediments

Sample Location	Contamination factor of single metal					PLI		Degree of contamination by Tomlinson *et al.* (1980).	
	Pb	Cd	Cr	Cu	Zn				
T-1	1.82	0.50	0.37	1.88	1.39	0.97	Unpolluted	5.96	Low
T-2	1.72	0.50	0.35	1.45	0.88	0.82	Unpolluted	4.89	Low
T-3	1.52	0.00	0.78	1.45	1.47	0.00	unpolluted	5.22	Low
T-4	1.42	2.00	0.33	1.56	0.73	1.01	polluted	6.04	Moderate
T-5	1.72	4.00	0.39	1.54	0.93	1.31	polluted	8.58	Moderate

3.5.3 Pollution load index

Pollution load index was calculated in this study by using the method of Wilson and Jeffrey (1987) for Turag river sediments. Pollution load index value of zero indicates highly polluted while PLI value 10 indicates unpolluted sediments. Values within zero to 10 are for moderately polluted sediments.
Pollution load index at different locations along Turag River is shown in Table 3.12. For Turag River, all the locations are moderately polluted as per PLI proposed by Wilson and Jeffrey (1987).

Table 3.12: Pollution load index at different locations along Turag River

Location	Pollution load index of single metal					Combine PLI by Wilson and Jeffrey (1987)	
	Pb	Cd	Cr	Cu	Zn		
T-1	5.13	25.12	2.04	0.60	0.10	1.74	Polluted
T-2	5.37	25.12	2.34	1.21	0.67	3.03	Polluted
T-3	5.89	31.62	0.27	1.20	0.07	1.35	polluted
T-4	6.31	12.59	2.51	1.00	1.17	2.98	Polluted
T-5	5.37	5.01	1.84	1.04	0.57	1.97	polluted

3.5.4 Mean sediment quality guideline quotient (SQG-Q)

In this study Wisconsin consensus based probable effect level is used for SQG-Q calculation. Sediment locations are then scored according to their impact level
SQG-Q < 0.1: Nonimpact and lowest potential for observing adverse biological effects;
0.1<SQG-Q<1: Moderate impact potential for observing adverse biological effects;
SQG-Q> 1: Highly impacted potential for observing adverse biological effects.
Mean sediment quality guideline quotient at five locations along Turag river is shown in tab-3.13. SQG-Q at locations T-1, T-2, T-3, T-4 and T-5 are 0.28, 0.23, 0.33, 0.23 and 0.27, respectively. All locations along Turag river are moderately impacted for observing adverse biological effects.

Table 3.13: SQG-Q in Turag River

Location	PEL-Q					SQG-Q	Sediment quality by long and MacDonald (1998)
	Pb	Cd	Cr	Cu	Zn		
T-1	0.28	0.02	0.33	0.40	0.39	0.28	Moderately impacted
T-2	0.26	0.02	0.30	0.31	0.25	0.23	Moderately impacted
T-3	0.23	0.00	0.69	0.31	0.41	0.33	Moderately impacted
T-4	0.22	0.08	0.29	0.33	0.21	0.23	Moderately impacted
T-5	0.26	0.16	0.35	0.33	0.26	0.27	Moderately impacted

The spatial variation of sediment quality guideline quotient along Turag river is shown in Fig. 3.6. As per SQG-Q, The points of the Turag river are moderately impacted.

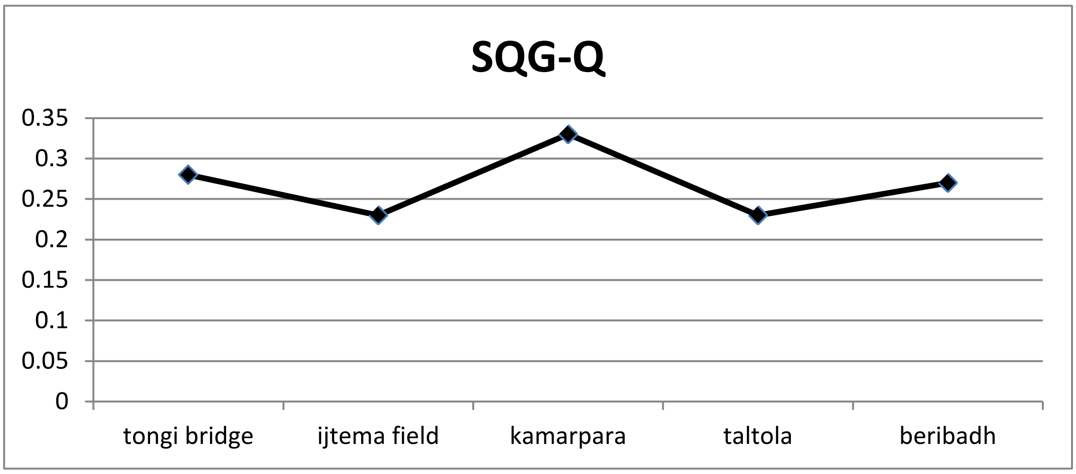

Fig 3.6: Spatial variation of sediment quality guideline quotient along Turag River.

4. CONCLUSIONS

From the study it is seen that according to the US EPA guideline the sediment of the Turag river is moderate to highly polluted by Cu and Cr, moderately polluted by Zn and not polluted by Cd and Pb.

By using the methods suggested by Hakanson (1980, 1988) through Potential Ecological Risk Index (PERI) it was found in this study that the Turag river has a low to moderate potential ecological risk due to heavy metal contamination. Cd is the most important one and its risk factor is upto the appreciable grade. Mean sediment quality guideline quotient at all locations along Turag river are moderately impacted for observing adverse biological effects. By using Pollution load index method of Tomlinson *et al.*(1980) shows that three points of the Turag river are unpolluted and two points are polluted. But all the locations are moderately polluted as per PLI proposed by Wilson and Jeffrey (1987).

In the sediment analysis by Ecological risk index it is found that the results from the test data vary a little with the values found in the analysis of Heavy Metal Contamination according to the US EPA Quality Guideline. This variation may be due to the difference in the equation of calculating the PERI. To overcome these variations further studies are needed.

5. ACKNOWLEDGEMENT

The authors thank to the BUET and MIST authority and the staffs of the Environmental Engineering laboratory of BUET and MIST for providing all kinds of assistance to conduct this research work.The authors sincerely acknowledge the valuable suggestions of Mr. P.K.Saha, lecturer, Department of Civil engineering, BUET and Jainab Banu, M.Sc. student of the Department of Civil Engineering, BUET.

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Assessment of Heavy Metal Contamination & Sediment Quality in the Shitalakhya River on the basis of Geo-accumulation index, Bangladesh

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ABSTRACT

Over the last few decades environment and its pollution is the governing concern for the human being in this planet. Shitalakhya River which encompasses eastern periphery of Dhaka city receives uncontrolled discharge of pollution, solid waste, sewage and waste water discharged from domestic, commercial and industrial activities of the city. These waste materials are tremendously deteriorating water and sediment quality of the river. The contamination level reaches at such an alarming stage during dry periods that not only the ecosystem of the river collapse but also the treatment plants of the city based on the river water virtually losses their expected quality of production. This study on Shitalakhya is approaching to assess the heavy metal contamination water and sediment quality of this river which will help to establish processing plants in future for possible remedial measures to reduce water and sediment pollutions, such as physical, chemical and biological pollution of the river. Concentration of five heavy metals (Pb ,Cd, Cr, Cu, Zn) for sediment from Shitalakhya river and for the portion of sediment which passing through the #200 sieve were determined in this study to evaluate their levels and spatial distribution. The ranges of the measured concentrations in the total sediments are as follows: 30.4-150.6 mg/kg for Zn, 5.6-94.8 mg/kg for Pb, 14.8-67.2 for Cu, 0.00-0.20 for Cd, 13.8-46.0 mg/kg dry weight of Cr contain higher heavy metal concentration. To assess metal contamination in sediment, US Environmental Protection Agency's (USEPA) guidelines were applied. The concentration of Pb and Cu in all sediment samples are above the EPA guideline for heavily polluted sediment and concentration of Zn, Cd and Cr fall in the criteria of moderately polluted range. The metal contaminations in sediment are also evaluated by applying Index of geo-accumulation (Igeo), contamination factor (Cf) and toxicity characteristics leaching procedure test (TCLP test). According to Geo-accumulation index all locations are unpolluted (Igeo≤0) with respect to Cr, Cd and Zn.In some places sediment quality of Pb and Cu were found moderately polluted (Igeo range: 1-2) and from unpolluted to moderately polluted (Igeo range: 0-1). TCLP results show that the Shitalakhya river sediments are not likely to leach hazardous concentration of particular toxic constituents into the environment as a result of improper management. For all the sites, concentrations of heavy metal in the leachate are not exceeded the permissible USEPA standard. From the result of TCLP test it can be said that after dredging the leaching effect of sediment is not so harmful to environment.According to the geo-accumulation index ($_{Igeo}$) some locations indicate unpolluted sediment for some heavy metals. It does not adjust with other heavy metal test. So the background concentration of heavy metals according to World Surface Rock Average, Matrin and Meyback is not suitable for our country.Sieve analysis of sediment samples, moisture content and organic matter content test were also performed in this study to determine the physical characteristics of sediment samples.

Keywords: *Sediment, heavy metal, organic content, contamination factor, geo-accumulation index, TCLP test*

1. INTRODUCTION

The river Shitalakhya running by the side of the Narayangonj district is one of the most polluted rivers in Bangladesh. The length of the river is about 110 km and the width near Narayangonj is about 300 m but reduces to about 100 m in the upper reach. BIWTA interested to clean the polluted sediment of Shitalakhya river bed. In this study attempts have been taken to estimate the heavy metal contamination of the dredged material and its effect on environment.

2. METHODOLOGY

The river Shitalakhya is one of the most prominent rivers in the flood plain region of Bangladesh. It is located in Narayangonj City, the second most vital industrial zone of the country. There were five different points in the river basing on the effect of industrial waste for sediment quality analysis. Those are: Narayangonj Bandar, South Rupshi Rupganj, Taitka Tarabo Narayangonj, Demraand Kaliganj. The study area of Shitalakhya River was located between 23^0 36' 58" N to 23^0 43' 38" N latitudes and $90^0$30'21" E to $90^0$27'53" E longitudes. The study area is also interconnected with the river Brahmaputra, Lakhya, Balu, and Turag on the northeast.

2.1 Metal Analysis Method

In choosing the most appropriate analytical method to determine metals, each laboratory must consider the sample type and concentration levels, the number of elements to be determined and the costs the choice implies. As a result flame and graphite furnace atomic absorption spectrometry (AA) and inductively coupled plasma (ICP and ICP-MS) emission spectrometry are the most widely used analytical methods for determining trace elements.

2.2 Assessment of pollution indices

2.2.1 Background enrichment indices

a) Sediment pollution according to toxic unit, where toxic unit, $TU = \sum_{i=1}^{i=n} \frac{C_{metal}}{PEL}$

b)) Sediment pollution according to geo-accumulation index

Geo-accumulation index proposed by Muller (1979) to determine metals contamination in sediments, by comparing current concentrations with pre-industrial levels and can be calculated using the following formula: $I_{geo} = \log_2 \left[\frac{C_n}{1.5B_n} \right]$, Where C_n is the measured concentration of the sediment for metal *(n)*, B_n is the geochemical background value of metal (n) and factor 1.5 is the possible variations of background data due to lithogenic impacts. World surface rock average given by Martin and Meybeck (1979) is considered as the background values of the heavy metals in this study.

2.2.2 standard for sediment quality paramters

Table 2.1: US EPA sediment quality guideline

(mg/kg) dry weights	**Zn**	**Pb**	**Cu**	**Cd**	**Cr**
US EPA sediment quality guideline					
Not Polluted	<90	<40	<25	-	<25
Moderately Polluted	90-200	40-60	25-50	-	25-75
Highly Polluted	>200	>60	>50	>6	>75

3. ILLUSTRATIONS

3.1 Metal Analysis

The total metal concentrations for each sampling site found in sediments in this study are shown below.Metal contents were ranging over following intervels:

Pb: 5.6-94.8 mg/kg; Cd: 0.02-0.0mg/kg;
Cr: 13.8-46mg/kg; Cu: 14.8-67.2mg/kg;
Zn: 30.4-150.6mg/kg;

Table 3.1: concentration of heavy metals (ppm dry weight)

Station no.	Concentration of heavy metals (mg/kg dry weight)				
	Pb	**Cd**	**Cr**	**Cu**	**Zn**
1	94.8	0	30.3	30.1	68.7
2	23.2	0	37.1	49.	150.6
3	23.2	0	46	67.2	150.2
4	6.8	0	15.8	15.6	34.7
5	5.6	0.2	13.8	14.8	30.4
Mean	30.72	.04	28.6	35.46	86.92
Max	94.8	0.2	46	67.2	150.6
Min	5.6	0	13.8	14.8	30.4
SD	36.81891	.089443	13.79112	22.67902	59.81828
Background	20	.2	97	32	129

3.2 Metal Analysis after TCLP Test

Toxicity Characteristics Leaching Procedure (TCLP) is a very important tool for assessing readily contaminated heavy metal for sediment samples. In this study, heavy metal concentrations from leachate of Shitalakhya River were tested in the laboratory and pollution levels were assessed with comparison of US EPA standard. Results of the TCLP test are presented in the Table 3.2.

Table 3.2: concentration of heavy metals (ppm dry weight)

Station no.	TCLP test result for the sediment sample of the Shitalakhya river				
	Pb	**Cd**	**Cr**	**Cu**	**Zn**
1	3.9	0	.38	5.98	22.86
2	2.08	.02	0	3.76	53.02
3	1.48	.08	0	3.54	70.44
4	.64	0	0	3.66	25.88
5	1.72	.02	0	3.06	16.76
Mean	1.964	.024	.076	4	37.792
Max	3.9	.08	.38	5.98	70.44
Min	.64	0	0	3.06	16.76
SD	1.205023	.032863	.169941	1.138947	22.94008
Regulated Level (USEPA)	5	1	5	-	-

Results of TCLP test for Shitalakhya River shows that, concentrations of heavy metal in the leachate are not exceeded the permissible US EPA standard. That indicate regarding the readily toxicity pollution by heavy metal, Shitalakhya river sediment condition is not at the severe state.

3.3 Heavy Metal Contamination and US EPA Quality Guideline

In absence of any local standards for pollutants, the metal levels in sediment sample were compared with the sediment quality guideline proposed by United States EPA. These criteria are shown in Table 3.3.
In table 3.3 it is seen that two locations of Shitalakhya River are heavily polluted with Pb and Cu, while two other locations are moderately polluted with Zn and Cr.

Table 3.3: Comparison between US EPA sediment quality guideline and present study

(mg/kg) dry weights	**Zn**	**Pb**	**Cu**	**Cd**	**Cr**
US EPA sediment quality guideline					
Not Polluted	<90	<40	<25	-	<25
Moderately Polluted	90-200	40-60	25-50	-	25-75
Highly Polluted	>200	>60	>50	>6	>75
Present Study					
Shitalakhya river	30.40-150.6	5.60-94.8	14.80-67.2	0.00-0.2	13.80 -46

3.3.1 Variation of Cu

Table 3.4 and Fig 3.1 shows that only 4 locations along Turag river are moderately polluted and Tongi bridge is highly polluted with Cu.Buriganga river are highly polluted with Cu.Upstream of Shitalakhya river is unpolluted and downstream section is moderately to highly polluted with Cu.

Table 3.4: Variation of Copper along Turag, Buriganga and Shitalakhya River in comparison to US EPA

location	**Cu (mg/kg)**	**US EPA standard**
Tongi bridge	60	Highly polluted
Ijtema field	46.3	Moderately polluted
Kamarpara	46.4	Moderately polluted
Beribadh	49.3	Moderately polluted
Wachpur	107.7	Highly polluted
Kolatiya	85	Highly polluted
K.char(end)	70	Highly polluted
K.char(north)	313.4	Highly polluted
Badamtoli	346	Highly polluted
Bandar	30.1	Moderately polluted
South rupshi	49.6	Moderately polluted
Tarabo	67.2	Highly polluted
Demra	15.6	Not polluted
Kaligonj	14.8	Not polluted

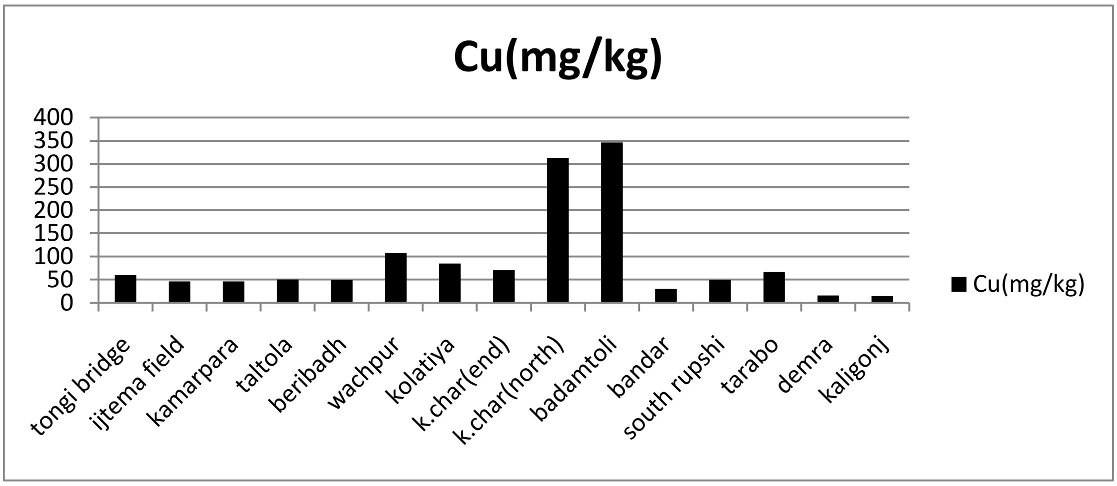

Figure 3.1: Variation of Copper along Turag, Buriganga and Shitalakhya River

3.3.2 Variation of Cr

From table 3.5 and fig. 3.2 it is seen that almost all the locations along three of the rivers are moderately polluted. Only one point along Turag, one point along Buriganga and two pints along Shitalakhya are highly polluted wile one point Shitalakhya is not polluted.

Table 3.5: Variation of Chromium along Turag, Buriganga and Shitalakhya River in comparison to US EPA

	Cr(mg/kg)	US EPA standard
Tongi bridge	36	Moderately polluted
Ijtema field	33.5	Moderately polluted
Kamarpara	75.5	Highly polluted
Taltola	32	Moderately polluted
Beribadh	38.1	Moderately Polluted
Wachpur	129.9	Highly polluted
Kolatiya	57.9	Moderately polluted
K.char(end)	52.8	Moderately polluted
K.char(north)	125.8	Highly polluted
Badamtoli	139.6	Highly polluted
Bandar	30.3	Moderately polluted
South rupshi	37.1	Moderately polluted
Tarabo	46	Moderately polluted
Demra	15.8	Not polluted
Kaligonj	13.8	Not polluted

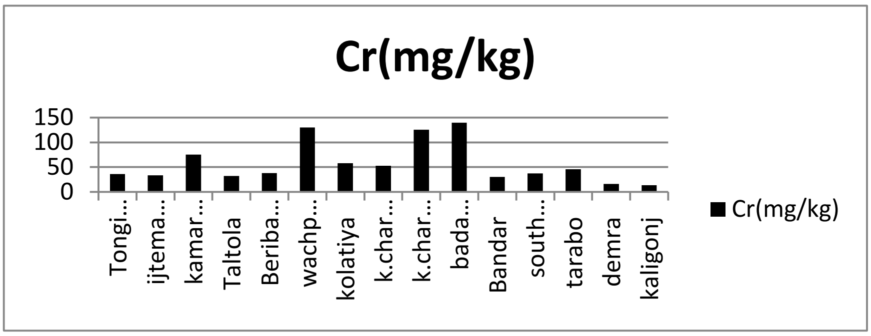

Figure 3.2: Variation of chromium along Turag, Buriganga and Shitalakhya River

3.3.3 Variation of Zn

From table 3.6 and fig. 3.3 it is seen that all the points along the Turag River and only two points along Shitalakhya are moderately polluted. Other three points of Shitalakhya Rivers are not polluted. But all the points along Buriganga are highly polluted.

Table 3.6: Variation of Zinc along Turag, Buriganga and Shitalakhya River in comparison to US EPA

Location	**Zn(mg/kg)**	**US EPA standard**
Tongi bridge	179.3	Moderately polluted
Ijtema field	113.8	Moderately polluted
Kamarpara	190.1	Moderately polluted
Taltola	94.6	Moderately polluted
Beribadh	119.6	Moderately Polluted
Wachpur	329.6	Highly polluted
Kolatiya	276	Highly polluted
K.char(end)	245	Highly polluted
K.char(north)	675.8	Highly polluted
Badamtoli	984.9	Highly polluted
Bandar	71.2	Not polluted
South rupshi	152.7	Moderately polluted
Tarabo	140.9	Moderately polluted
Demra	49.4	Not polluted
Kaligonj	56.6	Not polluted

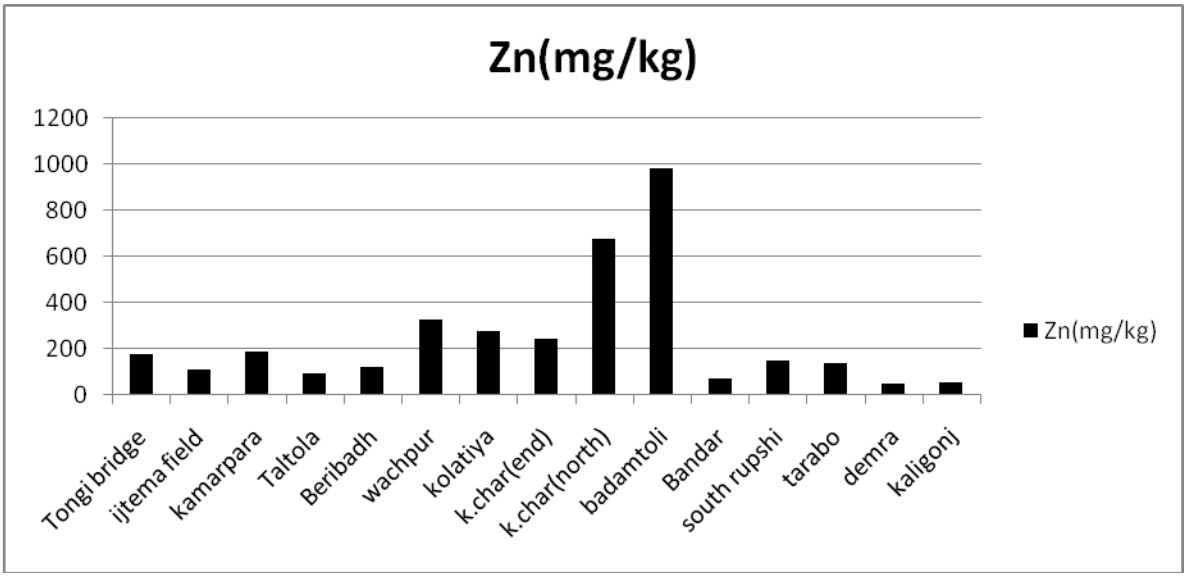

Figure 3.3: Variation of zinc along Turag, Buriganga and Shitalakhya River

3.3.4 Variation of Cd

There is no limit of US EPA sediment quality guideline for unpolluted and moderately polluted with cadmium. But Table 3.7 and fig. 3.4 shows that only one location along Turag River and two locations along the Buriganga River are highly polluted with cadmium according to the guideline.

Table 3.7: Variation of cadmium along Turag, Buriganga and Shitalakhya river in comparison to US EPA

Location	Cd(mg/kg)	US EPA standard
Tongi bridge	0.1	-
Ijtema field	0.1	-
Kamarpara	0.0	-
Taltola	0.4	-
Beribadh	0.8	Highly Polluted
Wachpur	0.4	-
Kolatiya	0.5	-
K.char(end)	0.4	-
K.char(north)	1.2	Highly polluted
Badamtoli	1.6	Highly polluted
Bandar	0.0	-
South rupshi	0.0	-
Tarabo	0.0	-
Demra	0.0	-
Kaligonj	0.2	-

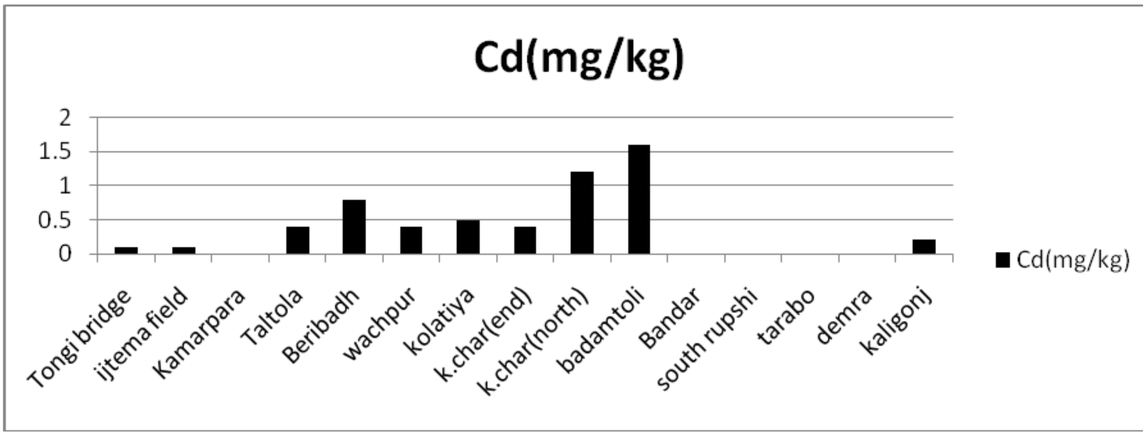

Figure 3.4: Variation of cadmium along Turag, Buriganga and Shitalakhya river

3.3.5 Variation of Pb

Table 3.8 and fig. 3.5 shows that only one point of Shitalakhya river is moderately polluted. All the other points along Shitalakhya and all the five locations along Turag are not polluted for lead. But all the five points along Buriganga are highly polluted for lead.

Table 3.8: Variation of lead along Turag, Buriganga and Shitalakhya River in comparison to US EPA

	Pb(mg/kg)	US EPA standard
Tongi bridge	36.4	Not polluted
Ijtema field	34.4	Not polluted
Kamarpara	30.4	Not polluted
Taltola	28.3	Not polluted
Beribadh	34.4	Not polluted
Wachpur	82.3	Highly polluted
Kolatiya	70.4	Highly polluted
K.char(end)	60.3	Highly polluted
K.char(north)	80.6	Highly polluted
Badamtoli	105.6	Highly polluted
Bandar	49.3	Moderately polluted
South rupshi	36.5	Not polluted
Tarabo	37.7	Not polluted
Demra	17.1	Not polluted
Kaligonj	18.9	Not polluted

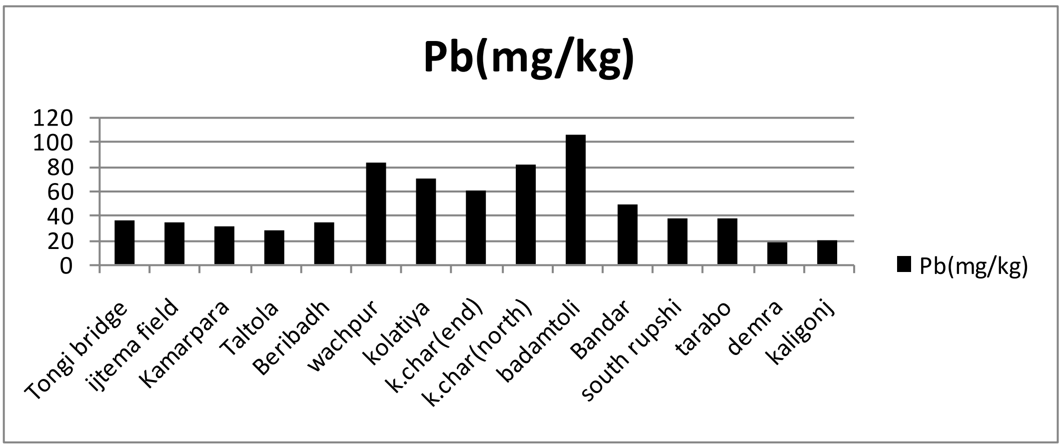

Figure 3.5: Variation of lead along Turag, Buriganga and Shitalakhya River

3.4 Indices Calculation

In this study contamination indices, background enrichment indices and ecological risk indices have been calculated.

3.4.1 Contamination indices calculation

In this study pollution load index by Tomlinson et al.(1980),metal pollution index by Usero *et al.* (1996) and marine sediment pollution index by Shin and Lam (2001) have been calculated.

Metal pollution index (MPI)

In order to evaluate the overall degree of stream sediment metal contamination, the metal pollution index (MPI) is calculated according Usero *et al.* (1996). Metal pollution index of the Turag,Buriganga and Shitalakhya rivers are shown in table 3.9. MPI ranges from 0 to 76.24 in the rivers. As metal concentration is comparatively high, Buriganga river sediments have a higher index than Turag River. On the other hand MPI for Shitalakhya are zero and in the other location it is low, so Shitalakhya sediments have a lower index than Turag River.

Table 3.9: Metal Pollution Index in rivers

Metal Pollution Index by Usero *et al.* (1996)			
location	Turag	Buriganga	Shitalakhya
1	16.98	43.27	0
2	14.34	34.34	0
3	0	29.36	0
4	17.65	76.24	0
5	22.82	95.72	5.87

Metal pollution index without considering Cadmium is shown in table 3.10. It is found that range of MPI Turag river is 45.49-67.08, which is comparatively lower than the ranges for Buriganga river (85.96-266.23) and higher than the ranges for Shitalakhya river (13.66-57.29).This index suggests that metal pollution in Turag river is less than Buriganga river but it is more than Shitalakhya river. All the selected locations along Turag River have almost uniform metal pollution index.

Table 3.10: Metal Pollution Index in rivers

Metal Pollution Index except Cadmium by Usero *et al.* (1996)			
location	Turag	Buriganga	Shitalakhya
1	61.28	139.57	49.37
2	49.64	98.89	50.35
3	67.08	85.96	57.29
4	45.49	215.27	15.53
5	52.72	266.23	13.66

The level of metal pollution has the trend: Buriganga > Turag > Shitalakhya

3.4.2 Background enrichment indices calculation

Background enrichment indices compare the results for the contaminants with different baseline or background levels, available in literature that can be used for any study area. Sediment pollution according to toxic unit proposed by Pederson *et al.* (1998), geo-accumulation index by Muller (1979), integrated pollution index (PIN index) by DR, 1995 have been calculated in this study.

3.4.2.1 Sediment pollution according to toxic unit

Wisconsin Consensus Based PEL has been used for toxic unit calculation.
From toxic unit values, it is shown that Turag River is comparatively more polluted than Shitalakhya River and less polluted than Buriganga River.

Table 3.11: Toxic unit at different sampling location

Toxic Unit by Pederson *et al.* (1998)			
location	Turag	Buriganga	Shitalakhya
1	1.42	3.33	1.35
2	1.15	2.33	1.17
3	1.64	2.02	1.37
4	1.13	5.56	0.38
5	1.36	6.85	0.37

3.4.2.2 Sediment pollution according to geo-accumulation index

A common criterion to evaluate the heavy metal pollution in sediments is the geo-accumulation index by Muller (1979).

Geo-accumulation Index (I_{geo}) at different sampling location in Shitalakhya River is shown in Table 3.12 According to I_{geo} all locations are unpolluted with Cr,Cd and Zn. Location S-2,S-3,S-4,S-5 are unpolluted and location S-1 is unpolluted to moderately polluted with Pb. Locations S-1, S-4 and S-5 are unpolluted, location S-2,S-3 are unpolluted to moderately polluted with Cu.

Table 3.12: Geo-accumulation Index (I_{geo}) at different sampling location in Shitalakhya River by Muller (1979)

Location	Pb	Cd	Cr	Cu	Zn
S-1	0.28	-1.58	-2.01	0.32	-0.11
S-2	0.20	-1.58	-2.12	-0.05	-0.77
S-3	0.02	-	-0.95	-0.05	-0.03
S-4	-0.08	0.42	-2.18	0.06	-1.03
S-5	0.20	1.42	-1.93	0.04	-0.69

Table 3.13: Muller's classification for the geo-accumulation index

I_{geo} value	Class	Sediment quality
≤0	0	Unpolluted
0-1	1	From unpolluted to moderately polluted
1-2	2	Moderately polluted
2-3	3	From moderately polluted to strongly polluted
3-4	4	Strongly polluted
4-5	5	From strongly to extremely polluted
>6	6	Extremely polluted

The spatial variation of the geo-accumulation index of selected heavy metals over Shitalakhya River is shown in the fig 3.6,3.7,3.8,3.9,3.10.

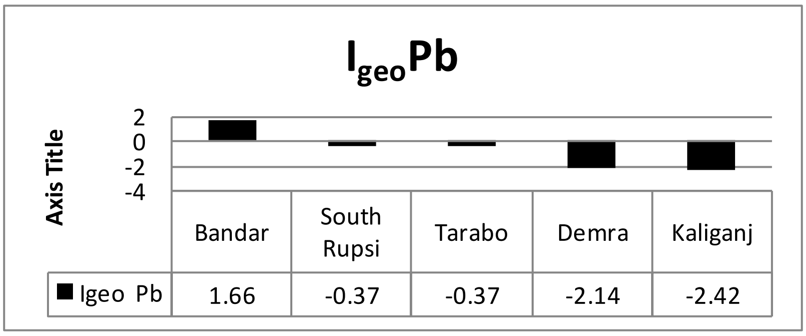

Figure 3.6: Spatial variation of Geo-accumulation index of lead in Shitalakhya River

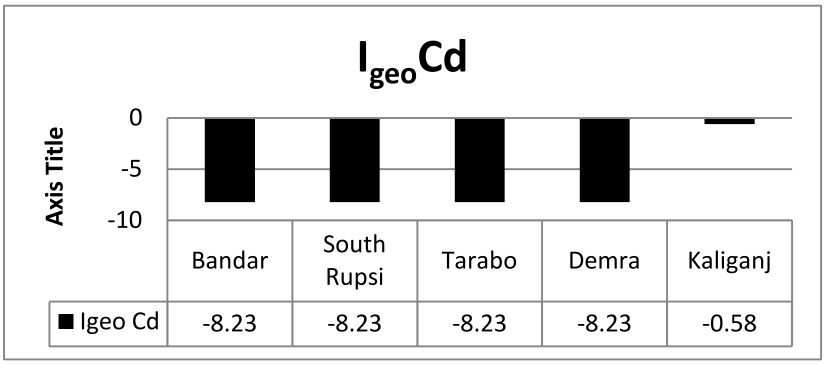

Figure 3.7: Spatial variation of Geo-accumulation index of Cadmium in Shitalakhya River

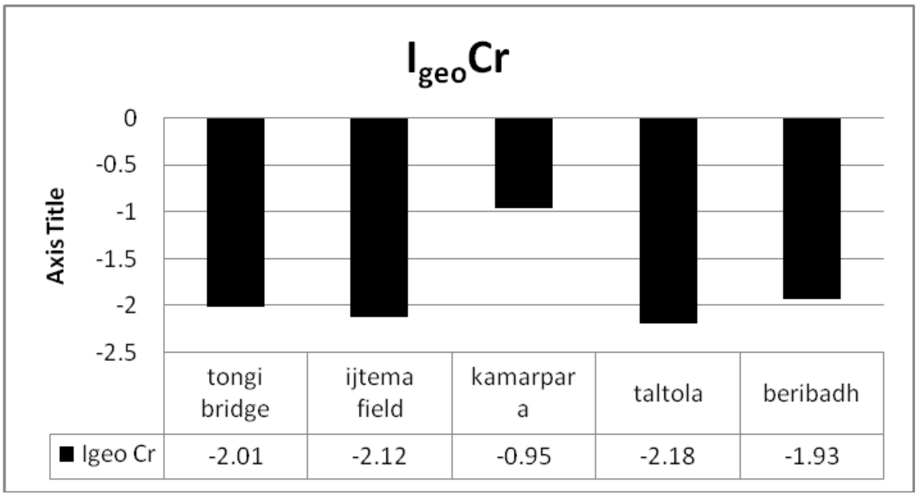

Figure 3.8: Spatial variation of Geo-accumulation index of Chromium in Shitalakhya River

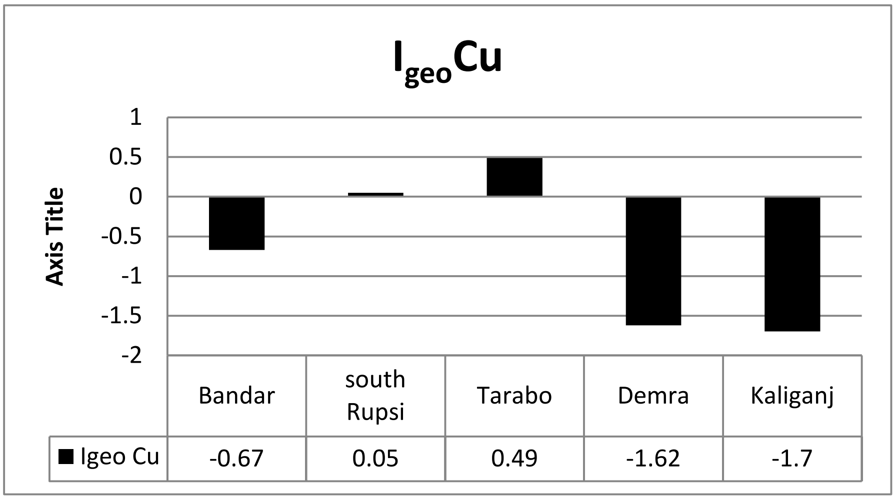

Figure 3.9: Spatial variation of Geo-accumulation index of Copper in Shitalakhya River

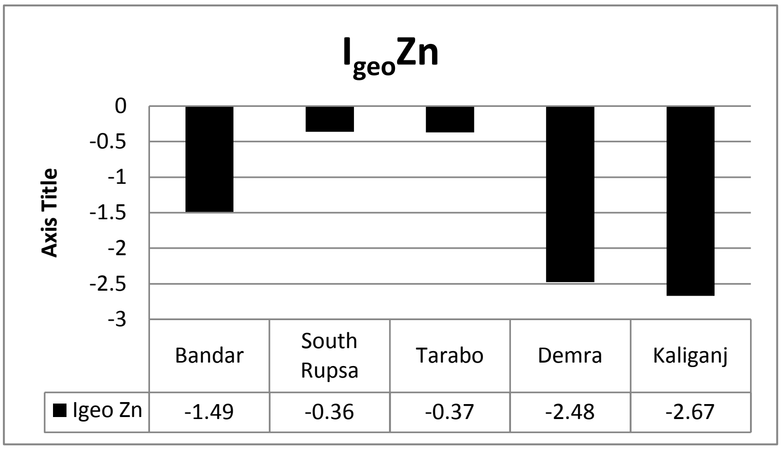

Figure 3.10: Spatial variation of Geo-accumulation index of Zinc in Shitalakhya River

From the fig 3.6 it is seen that Shitalakhya river is unpolluted to moderately polluted with Pb ($0\leq I_{geo} <1$).From the fig34.7 it is seen that Shitalakhya river is unpolluted with Cd. From fig 3.8 it is seen that the river is unpolluted with Cr. Again from fig 3.9 it is seen that river is unpolluted to moderately polluted with Cu ($0< I_{geo} <1$). From fig 3.10 it is seen that the river is unpolluted with Zn ($I_{geo}\leq 0$).
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3.4.2.3 Contamination factor and degree of contamination

Sediment pollution load index was calculated using the equation:

$CF=C_{metal}\div C_{background}$

$PLI= (CF_1+CF_2+CF_3+\ldots\ldots\ldots\ldots CF_n)^\wedge(1/n)$

Where, CF is the contamination factor,C_{metal} is the concentration of pollutant in sediment,$C_{background}$ is the background value for the metal and n is the number of metals.According to pollution load index,if PLI value>1 indicates polluted whereas <1 indicates no pollution.

As per pollution load index all locations are unpolluted. Highest pollution load index value is observed at S-3 and minimum at S-4.Degree of contamination values range from 1.26 to 6.53. Maximum value of degree of contamination was found at S-1 and minimum at S-4. As per Degree of contamination all locations are low contaminated.

Table 3.14: pollution load index and degree of contamination of Shitalakhya river sediments

Sample Location	**Contamination factor of single metal**					**PLI**		**Degree of contamination by Tomlinson *et al.*(1980).**	
	Pb	**Cd**	**Cr**	**Cu**	**Zn**				
S-1	4.7	0.00	0.3	0.9	0.5	0.33	Unpolluted	6.53	Low
S-2	1.2	0.00	0.4	1.6	1.2	0.33	Unpolluted	4.26	Low
S-3	1.2	0.00	0.5	2.1	1.2	0.37	Unpolluted	4.90	Low
S-4	0.3	0.00	0.2	0.5	0.3	0.13	Unpolluted	1.26	Low
S-5	0.3	1.00	0.1	0.5	0.2	0.34	Unpolluted	2.12	Low

3.4.2.4 Pollution load index

Pollution load index at different locations along Shitalakhya River is shown in Table 3.15. For Shitalakhya river,S-1,S-2 and S-3 locations are moderately to highly polluted and S-4,S-5 locations are unpolluted sediments. Metals contributing in pollution of sediments are arranged in the order as Zn>Cu>Cr>Pb>Cd.This PLI is proposed by Wilson and Jeffrey (1987).

Table 3.15: Pollution load index at different locations along Shitalakhya River

Location	**Pollution load index of single metal**					**Combine PLI by Wilson and Jeffrey (1987)**	
	Pb	**Cd**	**Cr**	**Cu**	**Zn**		
S-1	1.15	31.62	2.75	2.77	2.46	3.09	Polluted
S-2	7.10	31.62	1.95	1.02	0.23	2.53	Polluted
S-3	7.10	31.62	1.23	0.42	0.24	1.93	polluted
S-4	10.96	31.62	5.75	5.81	6.55	9.46	Unpolluted
S-5	11.22	19.95	6.37	6.06	7.41	9.15	Unpolluted

4. CONCLUSIONS

The concentrations of pb and cu in all sediment samples are above the epa guideline for heavily polluted sediment and on the other hand concentration of zn, cd and cr fall in the criteria of moderately polluted range. For all the sites, concentrations of heavy metal in the leachate are not exceeded the permissible usepa standard**.**

From the result of TCLP test it can be said that after dredging the leaching effect of sediment is not so harmful to environment.According to the geo-accumulation index ($_{Igeo}$) some locations indicate unpolluted sediment for some heavy metals. It does not adjust with other heavy metal test. So the background concentration of heavy metals according to World Surface Rock Average, Matrin and Meyback is not suitable for our country.

Heavy metals concentration in the river water samples like Mercury, Lead, Cobalt, Nitrate etc may be measured for the further studies at different locations where there is a waste water outfall from industries.
The Turag River has a low to moderate grade for potential ecological Risk Index and Pollution Load Index.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

HEAVY METAL ASSESSMENT IN SEDIMENT OF RIVER SHITALAKKHYA

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ABSTRACT

The objective of this study is to assess heavy metal contamination in sediment of river Shitalakkhya. It also provides a review of the available data on the sediment quality and water quality of this river. In this study samples were collected from five different locations along Shitalakkhya river in April, 2011. Samples were analyzed for heavy metals, i.e, Pb, Cd, Cr, Zn, Cu using Atomic Absorption Spectrophotometer. Aqua Regia digestion (USEPA method 3050) has been performed for the dissolution of the sediment samples. As per USEPA sediment quality guideline it can be summarized that in Sitalakkhya river sediment, pollution status is - Cr unpolluted to moderately polluted, Cu moderately to heavily polluted, Zn not polluted to moderately polluted, Pb not polluted to heavily polluted and Cd belongs to not polluted. Toxicity characteristics leaching procedure (TCLP) test (USEPA 1311) for sediment samples have been performed for heavy metals to determine the readily toxicity level. The test results show that the metal concentrations are well below the regulated level as per USEPA. Concentration data of the heavy metals were also analyzed statistically by using principal component analysis (PCA) and hierarchical cluster analysis (HCA) methods. According to geo-accumulation index, river is unpolluted with Cd, Cr and Zn, unpolluted to moderately polluted with Pb and Cu. According to PIN Index, river sediments are clean to trace contaminated. Analyzing degree of contamination, it can be showed that contamination of Sitalakkhya river by heavy metal is low to moderate. As per sediment quality guideline quotient (SQG-Q) Sitalakkhya river is not impacted to moderately impacted.

Keywords: *Heavy metal assessment, Sediment quality, geo-accumulation index , Pollution, Shitalakkhya river.*

1. INTRODUCTION

Many industries have set up in and around the Dhaka city during the last decade, and the number of new industries are continually increasing. Shitalakkhya river is strategically the most important river for Dhaka dwellers in terms of its use in domestic life. The adjacent areas of Shitalakkhya river have been termed as "Environmental Pollution Protected Zone" by Dhaka Metropolitan Development Plan (DMDP) (from 1995 to 2015). But this river is increasingly being polluted with the city's increasing number of industrial units and sewerage lines dumping huge volumes of toxic wastes which contain lots of heavy metal into it day and night.Heavy metal contamination in rivers is a major water quality issue in many fast growing cities. This is because improvements in water and sanitation infrastructure have not kept pace with population growth and urbanization in most developing countries (Mintz and Baier, 2000). Heavy metals are one of the serious pollutants in natural environment due to their toxicity, persistence and bioaccumulation problems (Pekey 2006; Nouri et al. 2006). Trace metals enter in river from variety of sources; it be can be either natural or anthropogenic (Bem *et al*., 2003; Wong *et al*., 2003; Adaikpoh *et al*., 2005; Akoto *et al.*, 2008). Main anthropogenic sources of heavy metal contamination are mining, disposal of untreated and partially treated effluents contain toxic metals, as well as metal chelates from different industries and indiscriminate use of heavy metal-containing fertilizer and pesticides in agricultural fields (Hatje *et al*., 1998; Amman *et al*., 2002; Nouri *et al.*, 2006; Nouri *et al.*, 2008). Rivers in ur ban areas have also been associated with water quality problems because of the practice of discharging of untreated domestic and small scale industries into the water bodies which leads to the increase in the level of metals concentration in river water (Rim-Rekeh *et al.*, 2006; Khadse *et al.*, 2008; Juang *et al.*, 2009; Venugopal *et al.*, 2009; Sekabira *et al.*, 2010). Heavy metal contaminations are important due to their potential toxicity for the environment and human beings (Gueu *et al.*, 2007; Lee *et al.*, 2007; Adams *et al.*, 2008; Vinodhini and Narayanan, 2008). Metals such as Cr, Mn, Co, Cu, Fe and Zn play biochemical roles in the life processes of aquatic plants and animals, and their presence in trace amounts in the aquatic environment is essential (Nurnberg, 1982). However, heavy metals are non-degradable and can accumulate in the human body system, causing damage to nervous system and internal organs (Lee *et al.*, 2007; Lohani *et al.*, 2008).

The aim of this study is to assess the level of heavy metal concentrations and sediment quality as well as surface water quality of river Shitalakkhya and to assess the pollution status of the area and to highlight relationships among metals pollution.

2. METHODOLOGY

2.1 Study Area

The study area is Shitalakkhya river, located beside Dhaka city, capital of Bangladesh. This river (also known as Lakhya River) is a distrybutary of the river Brahmaputra. The river is about 110 km long and at it widest, near Narayangani, it is 300 meters in width. Its highest discharge has been measured at 2,600 cu ft/s (74 m^3/s) at Demra. It remains navigable year round. The river's maximum depth is 70 feet (21 m) and average depth is 33 feet (10 m) (Wikipedia). Due to indiscriminate disposal of untreated industrial waste as well as domestic waste, this river is polluting day by day and its ecological and environmental conditions are being affected by these activities.

2.2 Field Sampling

In order to achieve the research objective, samples were collected from five different locations of the Shitalakkhya river.Lattitudes and Longitudes for each sites are illustrated in Table 1. Criteria for selection of sampling stations were based on the locations of industrial units and land use pattern to quantify heavy metal concentration. Samples were taken from river Shitalakkhya using washed plastic container to avoid unpredictable changes in characteristics of samples. Samples were collected in April, 2011.

Table 1: Global positioning system (GPS) data of sample collection in Sitalakhya river

Designation	Location	Latitude	Longitude
S-1	Bandar, Narayanganj	23º36'57.18"N	90º30'22.62"E
S-2	South Rupsi, Rupganj, N. Ganj	23º43'59.91"N	90º30'31.4"E
S-3	Tarabo, Narayanganj	23º42'57.87"N	90º30'22.45"E
S-4	Demra Ghat, Dhaka	23º43'18.58"N	90º30'E
S-5	Kaliganj	23º56'46.38"N	90º37'1.75"E

2.3 Laboratory Works

After collection, sediment samples were dried in a vacuum oven at 105°C until constant weight, lightly ground in an agate mortar for homogenization and prepared for analysis of heavy metals. For heavy metal test, 5 gm of dried samples was digested with acid and prepared 500 ml solution. Finally, heavy metal concentrations were determined in the Environmental Engineering Laboratory, BUET by using Atomic Absorption Spectrophotometer.

3. RESULTS AND DISCUSSIONS

3.1 Sediment Composition

Sediment composition was determined for all samples by sieve analysis (shown in Table 2) and it is found that on an average, 25.424% materials passing through the #200 sieve whereas 74.58% materials retain on #8 to #200 sieves which indicates the sediment size of Shitalakkhya river ranges over medium to fine. A grain size distribution curve for sample site 1 is shown in the following Figure 1.

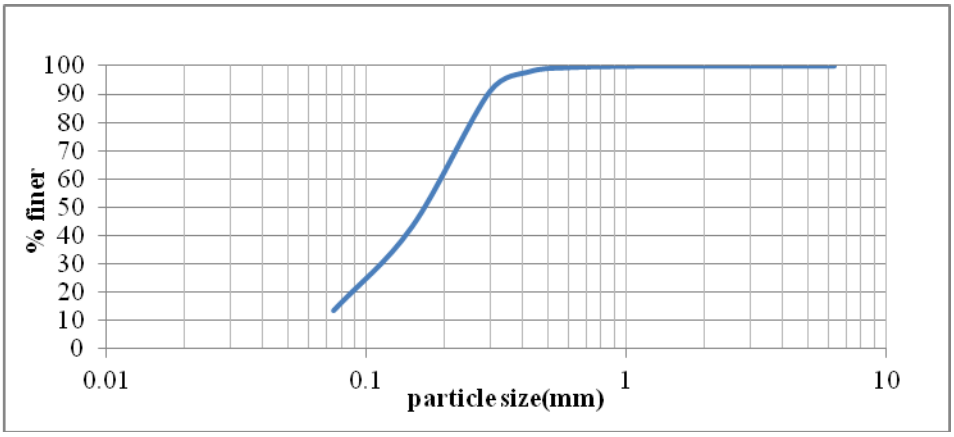

Figure 1: Grain size distribution curve of S1

Table 2 : Sieve Analysis Result of Sediment Sample of the River Shitalakkhya

	S1	S2	S3	S4	S5	
	Bandar, N.Ganj	South Rupsi, Rupganj	Tarabo, N. Ganj	Demra	Kaliganj	
Sieve size (ASTM)	% retains (gms)	% retains (gms)	% retains (gms)	% retains (gms)	% retains (gms)	average
1/4”	0	0	0	0	0	0
No. 4	0	0	0	0	0	0
No.8	0	0	0	0	0	0
No. 16	0	0	0	0	0	0
No. 30	0.51	0.43	2.24	0.06	0.85	0.818
No. 40	1.47	1.48	9.95	0.82	6.15	3.974
No. 50	6.93	2.04	7.13	4.21	18.73	7.808
No. 100	48.14	9.33	11.83	51.81	61.05	36.432
No. 200	29.56	23.43	28.88	33.64	12.21	25.544
Pan	13.39	63.29	39.97	9.46	1.01	25.424
Total	100	100	100	100	100	100
Coarse Particle	0	0	0	0	0	0
Medium Particle	86.61	36.71	60.03	90.54	98.99	74.576
Fine Particle	13.39	63.29	39.97	9.46	1.01	25.424

3.2 Heavy Metal Concentration

The total metal concentrations for each sampling site found in sediments in this study are shown in Table 3. Metal contents were ranging over following intervals: Pb: 5.6-94.8 mg/kg; Cd: 0.0-0.2 mg/kg; Cr: 13.8-46.0 mg/kg; Cu: 14.8-67.2 mg/kg; Zn: 30.4-150.60 mg/kg dry weights. Mean concentration of the metals were: Pb: 30.72 mg/kg; Cd: 0.04 mg/kg; Cr: 28.60 mg/kg; Cu: 35.46 mg/kg; Zn: 86.92 mg/kg dry weights, allowing to arrange the metals from higher to lower mean content in this area as: Zn > Cu > Pb> Cr > Cd.

Table 3: Concentration of heavy metals (mg/kg dry weight) of sediments of Sitalakhya river

Location	Lead (Pb)	Cadmium (Cd)	Chromium (Cr)	Copper (Cu)	Zinc (Zn)
S-1	94.80	0.00	30.30	30.10	68.70
S-2	23.20	0.00	37.10	49.60	150.60
S-3	23.20	0.00	46.00	67.20	150.20
S-4	6.80	0.00	15.80	15.60	34.70
S-5	5.60	0.20	13.80	14.80	30.40
Mean	30.72	0.04	28.60	35.46	86.92
Max	94.80	0.20	46.00	67.20	150.60
Min	5.60	0.00	13.80	14.80	30.40
SD	36.82	0.09	13.79	22.68	59.82
Background Concentration (Martin and Meybeck, 1979)	20	0.2	97	32	129

Pearson's corelation coefficient matrix among the selected heavy metals is presented in Table 4. Significant correlation between the contaminants of Cu-Cr (r = 0.98), Zn-Cr (r= 0.94) and Zn-Cu (r=0.96) pairs indicate the same or similar source input.

Table 4: Pearson's Correlation matrix between heavy metals in sediment samples from the Sitalakkhya River

	Pb	Cd	Cr	Cu	Zn
Pb	1				
Cd	-0.38	1			
Cr	**0.29**	-0.60	1		
Cu	0.09	-0.51	**0.98**	1	
Zn	0.06	-0.53	**0.94**	**0.96**	1

The metal concentrations for sediment passing through No.200 sieve are shown in Table 5. Metal contents were ranging over following intervals: Pb 17.10-49.30 mg/kg; Cd 0.00-0.1 mg/kg; Cr 0.7-3.3 mg/kg; Cu 48.10-75.40 mg/kg; Zn 49.40-152.70 mg/kg dry weights. Mean concentration of the metals were: Pb-31.90 mg/kg; Cd-0.02 mg/kg; Cr-2.16 mg/kg; Cu-64.28 mg/kg; Zn-94.16 mg/kg dry weights. By comparing the result obtain from metal concentrations of total sediment(without sieve) and metal concentrations of sediment passing though No.200 sieve, it is shown that the average concentration for all the metal in fine portion is higher than total sediment except Cd and Cr. It is also shown in this study that the SD of the metals at different sites are very high(except Cd) that may indicate the spatial distribution of metal contamination is not uniform.

Table 5: Concentration of Heavy Metals (mg/kg dry weight) sediments passing through # 200 sieve

Station No.	Pb	Cd	Cr	Cu	Zn
1	49.30	0.10	2.30	64.70	71.20
2	36.50	0.00	2.80	74.60	152.70
3	37.70	0.00	3.30	75.40	140.90
4	17.10	0.00	0.70	48.10	49.40
5	18.90	0.00	1.70	58.60	56.60
Mean	31.90	0.02	2.16	64.28	94.16
Max	49.30	0.10	3.30	75.40	152.70
Min	17.10	0.00	0.70	48.10	49.40
SD	13.65	0.04	1.01	11.45	48.87
Background Concentration	20	0.2	97	32	129

3.3 Assessment According to USEPA

The chemical contaminations in the sediments were evaluated by comparison with the sediment quality guideline proposed by USEPA. These criteria are shown in Table 6.

Table 6 : EPA Guideline for Sediments (MG/KG Dry Weights)

Metal	Not Polluted	Moderately Polluted	Heavily Polluted	Present Study
Pb	<40	40-60	>60	5.6-94.8
Cd	. . .	. . .	>6	0-.2
Cr	<25	25-75	>75	13.8-46
Cu	<25	25-50	>50	14.8-67.2
Zn	<90	90-200	>200	30.4-150.6

From Table 6, comparing with EPA guidelines, present study shows that –For Pb, only station 1 was heavily polluted and no pollution was occurred in all other stations. For Cd, all stations were free from pollution where for Cr and Zn, all sites were ranged over not polluted to moderately polluted conditions. Again, for Zn stations 1 and 2 were moderately polluted, station 3 was highly polluted, stations 4 and 5 were not polluted.

3.4 Assessment According to Geo-accumulation Index (I_{geo})

To understand the current environmental status and the metal contamination with respect to natural environmental, other approaches should be applied.

A common criterion to evaluate the heavy metal pollution in sediments is the geo-accumulation index (I_{geo}), which was originally defined by Muller(1979) to determine metals contamination in sediments, by comparing current concentrations with pre-industrial levels and can be calculated by the following equation (Muller,1979):

$I_{geo}=log_2[C_n/1.5B_n]$
(I)

Where, C_n is the concentration of element 'n' and B_n is the geochemical background value [In this study, consider B_n=world surface rock average given by Martin and Meybeck(1979)]. The factor 1.5 is incorporated in the relationship to account for possible variation in background data due to lithogenic effect. The geo-accumulation index (I_{geo}) scale consists of seven grades (0-6) ranging from unpolluted to highly pollute (shown in Table 7).

Table 7: MULLERS Classification for the Geo-accumulation Index

I_{geo} value	class	Sediment quality
≤0	0	unpolluted
0-1	1	From unpolluted to moderately polluted
1-2	2	Moderately polluted
2-3	3	From moderately to strongly polluted
3-4	4	Strongly polluted
4-5	5	From strongly to extremely polluted
>6	6	Extremely polluted

Table 8: Geo-accumulation Index values for the sediment samples of Shitalakkhya river

station	Igeo				
	Pb	Cd	Cr	Cu	Zn
1	1.66	-	-2.26	-0.67	-1.49
2	-0.37	-	-1.97	0.05	-0.36
3	-0.37	-	-1.66	0.48	-0.37
4	-2.14	-	-3.20	-1.62	-2.48
5	-2.42	-0.58	-3.40	-1.69	-2.67
Mean	-.73	-	-2.5	-0.69	-1.47

According to the Muller scale, the calculated results of I_{geo} values (Shown in Table 8) indicate, for Pb sediment quality be considered as unpolluted (I_{geo}<0) for all stations (except station 1,where sediment is moderately polluted) while for Cd, Zn and Cr sediment quality were recorded unpolluted for all stations (I_{geo}<0). For Cu, sites 2 and 3 are ranges from unpolluted to moderately polluted sediment quality. On the basis of the mean values of I_{geo}, sediments are enriched for metals in the following order: Cu > Pb > Zn > Cr >Cd. Variation of Igeo value in different sites are shown in the following figure.

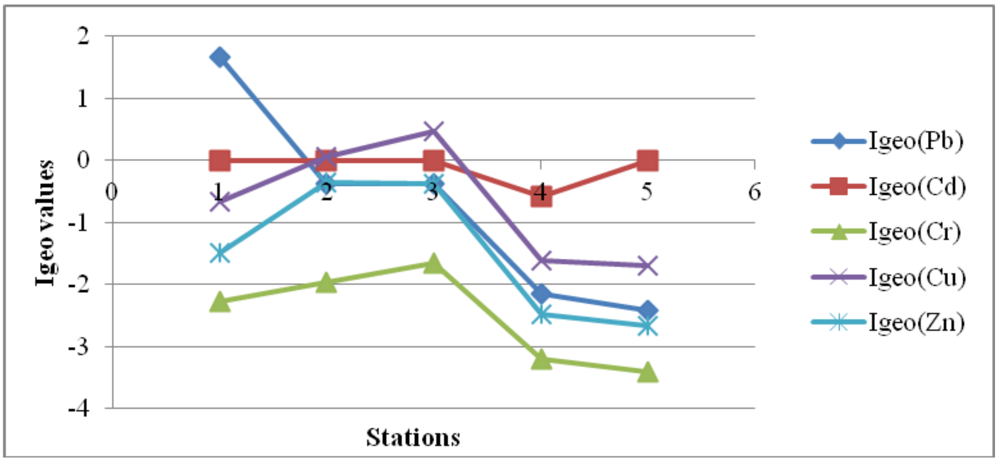

Figure 2 Variation of Igeo value at different sites of the Shitalakkhya river for different heavy metals

3.5 Assessment According to Contamination Factor (C_f) and degree of contamination (C_d)

The contamination factor and degree of contamination are used to determine the contamination status of sediment in current study. The degree of contamination was defined as the sum of all contamination factors.C_f values for describing the contaminations level are shown in Table 9. The contamination factor are calculated according to the equation (II). Calculated contamination factor (C_f) and degree of contamination (C_d) for this study is shown in the Table 10.

C_f = *(measured concentration)/(background concentration)* *(II)*

Where, Background concentration value of the metal = world surface rock average given by Martin and Meybeck (1979).

Table 9: Contamination Factor & Level of Contamination (HAKANSON 1980)

Contamination Factor (Cf)	Level of Contamination
Cf <1	Low contamination
1≤ Cf <3	Moderate contamination
3≤Cf<6	Considerable contamination
Cf>6	Very high contamination

Table 10: Contamination Factor values for the sediment sample of Shitalakkhya river

Station	Contamination Factor, (C_f)					Degree of contamination(C_d)	PLI
	Pb	Cd	Cr	Cu	Zn		
1	4.74	0	.31	.94	.53	6.52	0
2	1.16	0	.38	1.55	1.17	4.26	0
3	1.16	0	.47	2.1	1.16	4.89	0
4	.34	0	.16	.49	.27	1.26	0
5	.28	1	.14	.46	.24	2.12	0.337
Mean	1.54	.2	.29	1.11	.67	3.81	0.067

Table 10 represents the contamination level of heavy metals in sediment sample of Shitalakkhya river. Contamination Factor, Cf>6 (very high contamination) was not found in any station of sediment samples. Low contamination levels were found in all stations of Cd, Cr and no.4 and 5 stations of Pb. For Cu and Zn, all stations except 2 and 3 were low contaminated where station 2 and 3 had moderate contamination level. For Pb, considerable contamination was found in station 1and station 2 and 3 had moderate contamination level.

3.6 Assessment according to PLI

Tomlinson *et al.* (1980) had employed a simple method based on pollution load index to assess the extent of pollution by metals in estuarine sediments. Sediment pollution load index was calculated using the following equation:

$$PLI= (C_{f1}+C_{f2}+C_{f3}+....C_{fn})^{(1/n)} \quad (III)$$

Where, C_f is contamination factor. According to pollution load index, if PLI value > 1 indicates polluted, whereas <1 indicates no pollution. From the Table 10, it is shown that for all sites, PLI value <1, indicates that the sites are not polluted with heavy metals.

3.7 Assessment According to MSPI

Marine sediment pollution index (MSPI) at different sampling location in Sitalakhya river is shown in Table 13. According to Marine sediment pollution index (MSPI), sediments at S-1, S-2 and S-3 have poor condition and locations S-4 and S-5 are in good condition. So, sediments of Shitalakkhya river have a range of good to poor condition.

Table 11: The weight calculation for each selected variables of Sitalakhya river

PC	Eigenvalue	Relative Eigenvalue	Variable	Loading Value	Relative Loading value on same PC	Weight, w_i (Relative Eigenvalue x Relative Loading value)
1	3.372	0.75	Cr Cu Zn	0.950 0.985 0.982	0.33 0.34 0.33	0.25 0.26 0.25
2	1.105	0.25	Pb	0.949	0.949	0.24
Total	4.477	1				1

Table 12: Sediment Quality Rating (q_i) for each variable: Percentile of each variable

Pollutant	S-1	S-2	S-3	S-4	S-5
Pb	100	80	80	40	40
Cr	80	80	90	60	60
Cu	80	80	80	60	60
Zn	60	80	80	40	40

Table 13: MSPI for the Shitalakkhya river

MSPI	63.7	64	68.06	25.20	25.20
Comments as per Shin and Lam (2001)	Poor	Poor	Poor	Good	Good

3.8 Assessment According to TCLP

Toxicity characteristics leaching procedure(TCLP) is a very important test to assess the contamination level by heavy metals in sediment. This test was done in the laboratory to determine the concentration of different heavy metals from leachate of Shitalakkhya River bed sediments and the test results shown in the Table 15 were also compared with the EPA standard values.

Table 15: TCLP test result (mg/L) for the sediment sample of the Shitalakkhya River

Station No.	Pb	Cd	Cr	Cu	Zn
1	0.195	0.00	0.019	0.299	1.143
2	0.104	0.00	0.00	0.188	2.651
3	0.074	0.00	0.00	0.177	3.522
4	0.032	0.00	0.00	0.183	1.294
5	0.086	0.00	0.00	0.153	0.838
Mean	0.10	0.00	0.00	0.20	1.89
Max	0.20	0.00	0.02	0.30	3.52
Min	0.03	0.00	0.00	0.15	0.84
SD	0.06	0.00	0.01	0.06	1.15
EPA Standard	5	1	5		

Comparing the test result with the EPA standard values (shown in Table 6), concentration of heavy metals in leachate did not exceed the permissible EPA standard. This indicates that the pollution by heavy metals in Shitalakkhya River sediment is not severe.

3.9 Ecotoxicological Sense of Heavy Metal Contamination

The ecotoxicological sense of heavy metal contamination in sediments was determined using sediment quality guidelines (SQG) developed for marine and estuarine ecosystem. Bakan and Koe (2007). These effects are as follow:
a) The effect range low (ERL) / effect range median(ERM) b) The threshold effect level (TEL) / probable effect level (PEL)
Where, ERL and TEL are concentrations below which adverse effects upon sediment dwelling fauna would infrequently be expected whereas, ERM and PEL represent chemical concentration above which adverse effects are likely to occur.
Toxic unit is determined by the ratio of the determined concentration to PEL value. Pederson et al. (1998), where n is the number of heavy metals tested at a particular site.

$$TU = sum(C_{metal}/PEL)\text{.......................}(IV)$$

Location	Pb	Cd	Cr	Cu	Zn	TU
S-1	94.80	0.00	30.30	30.10	68.70	1.58
S-2	23.20	0.00	37.10	49.60	150.60	1.45
S-3	23.20	0.00	46.00	67.20	150.20	1.67
S-4	6.80	0.00	15.80	15.60	34.70	0.43
S-5	5.60	0.20	13.80	14.80	30.40	0.432
Mean	30.72	0.04	28.60	35.46	86.92	
TEL	30.2	0.68	52.3	18.7	124	
PEL	110	4.2	160	110	270	
ERL	46.7	1.2	81	34	150	
ERM	218	9.6	370	270	410	

From the toxic unit values, the station 1,2 and 3 are more polluted than station 4 and 5. Again from the above table, it is found that average concentration of Pb and Cu are above TEL limit whereas concentration of Cd,Zn and Cr are bellow TEL limit. Variation of toxic unit at different sites are shown in the figure below.

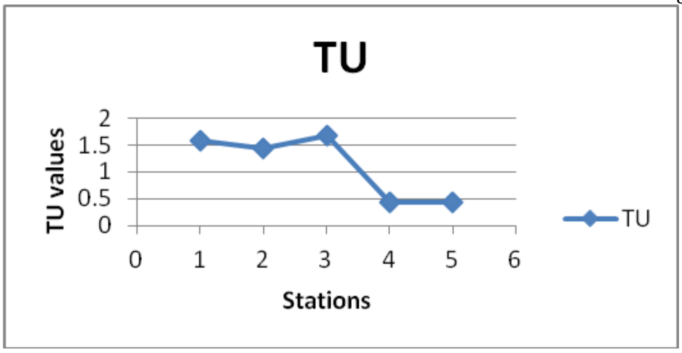

Figure 3 : Variation of TU at different sites of river Shitalakkhya.

4. CONCLUSION

The results of this study supply valuable information about metal heavy contents and physical characteristics of sediment from different sampling stations of Shitalakkhya river. The order of the mean concentrations of tested heavy metals: Zn > Cu > Pb> Cr > Cd and it is found in this study that, the fine portion of the sediment contains more heavy metal contents than total sediments except Cu and Cr. USEPA guideline, Geo-accumulation Index, contamination factor and degree of contamination, TCLP, PLI and TU are successfully applied for the assessment of contamination by heavy metals. The results obtained from I_{geo} method emphasized, to a large extent, the results obtained from the contamination factor. It could be concluded that both I_{geo} and C_f are powerful tools for the assessment of contamination of heavy metals. As showing, the two methods of calculations (C_f and I_{geo}) are useful and successful but I_{geo} is more specific to determine the degree of pollution than the contamination factor, where I_{geo} is classified to 6 classes while the later containing four classes only. TCLP test showed that there is no considerable risk of contaminations from leachate of Shitalakkhya river sediments. Considering all assessment criteria, Pb, Cu and Zn are responsible for moderate to considerable amount of heavy metal contaminations while Cr and Cd are responsible for no to moderate level of contaminations. Station 1 (Bandar, Narayanganj) contains highest amount of heavy metal contamination and station 4 (Demra Ghat, Dhaka)and 5 (Kaliganj) contain lowest amount of heavy metal contamination.

ACKNOWLEDGEMENTS

The Authors are grateful to the Bangladesh University of Engineering and Technology (BUET) Authority and the staffs of the BUET Environmental Engineering Laboratory, for providing all kinds of assistance for this research work.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF WATER QUALITY INDEX OF WATER BODIES ALONG JOYDEVPUR-CHANDRA-TANGAIL-HATIKAMRUL ROAD

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ABSTRACT

To compare the water quality of different water bodies water quality index (WQI) can be used as a tool. It provides information to the general public about the possible problems of the particular area. The objective of this study is to find out water quality parameters of 16 different water stations along the Joydevpur-Chandra-Tangail-Hatikamrul (JCTH) road and also to determine the water quality index. Based on WQI an attempt was made to find out the domestic usage acceptability of these 16 stations. WQI turns complex water quality into simple information which is comprehensible and functional by users. Five most important parameters such as pH, total dissolved solids (TDS), dissolves oxygen (DO), biochemical oxygen demand (BOD), electrical conductivity (EC) were taken for the calculation of WQI. Whenever there are human activities, water gets polluted to some extent, so the value of WQI increases. Physico-chemical parameters were monitored for the calculation of WQI for the pre-monsoon season. The WQI values varied between 45 to 109. The values of the WQI showed that the water of the maximum stations can be classified as poor and very poor, only a few of them can be referred as good, but unfortunately among the all water stations none of the station contains excellent water quality parameter for human consumption. The results revealed that WQI of the most of the water bodies along the Joydevpur-Chandra-Tangail-Hatikamrul road are within the acceptable limit though it is essential to purify when they will be used for drinking water purpose.

Keywords: *Water quality index (WQI), Pre-monsoon, Dissolved Oxygen (DO), Biochemical Oxygen Demand (BOD), Total Dissolved Solids (TDS).*

1. INTRODUCTION

Wetlands are vital water bodies, as crucial in a natural ecosystem as a kidney in a human body (Sisodia 2006). The water within the wetland is generally collected from precipitation through surface runoff, groundwater recharge and expulsion of store water in natural reservoirs, such as a glacier. Not only from being highly productive as the habitat of birds, fishes and a variety of other aquatic life forms, including microorganisms; wetlands maintain the natural balance to sustain human livelihoods. In recent times these water bodies are being neglected due to the undesirable activities by human. Many of the precious water bodies are being treated as a waste lands as they have been converted to other uses.

Wet lands are of great environmental issue and studied by a wide range of specialists including hydrologists, engineers, ecologists, geologists and geo morphologists ((Ashwani Kumar and Anish dua, 2009): It has become important issue for them as it affects not only human uses but also plant and animal life.

Quality of water is defined in terms of its physical, chemical, and biological parameters. However, the quality is difficult to evaluate from a large number of samples, each containing concentrations for many parameters (Almeida. 2007). Horton (1965) proposed the first Water Quality Index (WQI), a great deal of consideration has been given to the development of index methods. A water quality index provides a single number that expresses overall water quality at a certain location on several water quality parameters and turn complex water quality data into information that is understandable and useable by the general people. According to Nives (1999) WQI is a mathematical instrument used to transform large quantities of water quality data into a single number which represents the water quality level while eliminating the subjective assessments of water quality and biases of individual water quality experts. Basically a WQI attempts to provide a mechanism for presenting a cumulatively derived, numerical expression defining a certain level of water quality (Miller et al., 1986). Comparison can be made through the WQI among the water bodies and a general analysis of water quality on different levels can be made.

Importance of water bodies along the roadside is evident in terms of water quality, biodiversity conservation and use for aquaculture, as maximum of the water bodies of country are expected to be productive. So utilization of the existing resources is very much vital. In the way to improving the condition of these water resources, its proper management is very much necessary to have all information on the resources namely physico-graphic, chemical and biological characteristic of these water resources. The objective of the study is to investigate the water quality of the various water bodies along the Joydevpur-Chandra-Tangail-Hatikamrul road by using water quality index (WQI) and also find out the domestic usage acceptability of these 16 stations.

2. MATERIALS AND METHODS

The present study was conducted along the Priority road which is of 110 Km in length touching 3 districts namely Gazipur, Tangail, and Sirajgonj along with 8 upazilla's namely Gazipur sadar, Kaliakoir, Kalihati, Mirzapur, Tangail Sadar, Delduar, Rayganj and Ullahpara. The parameters - water temperature, pH, dissolved oxygen, total dissolved solids (TDS) and electrical conductivity (EC) of 16 different locations along JCTH road during pre monsoon (March-April, 2011) were analyzed immediately at the sampling site using standard equipment. For BOD measurement, a 500ml bottle was used for collection of water samples and the oxygen was fixed at the sampling site before being carried to the laboratory. A satellite image of the 16 stations has been shown in figure.1.

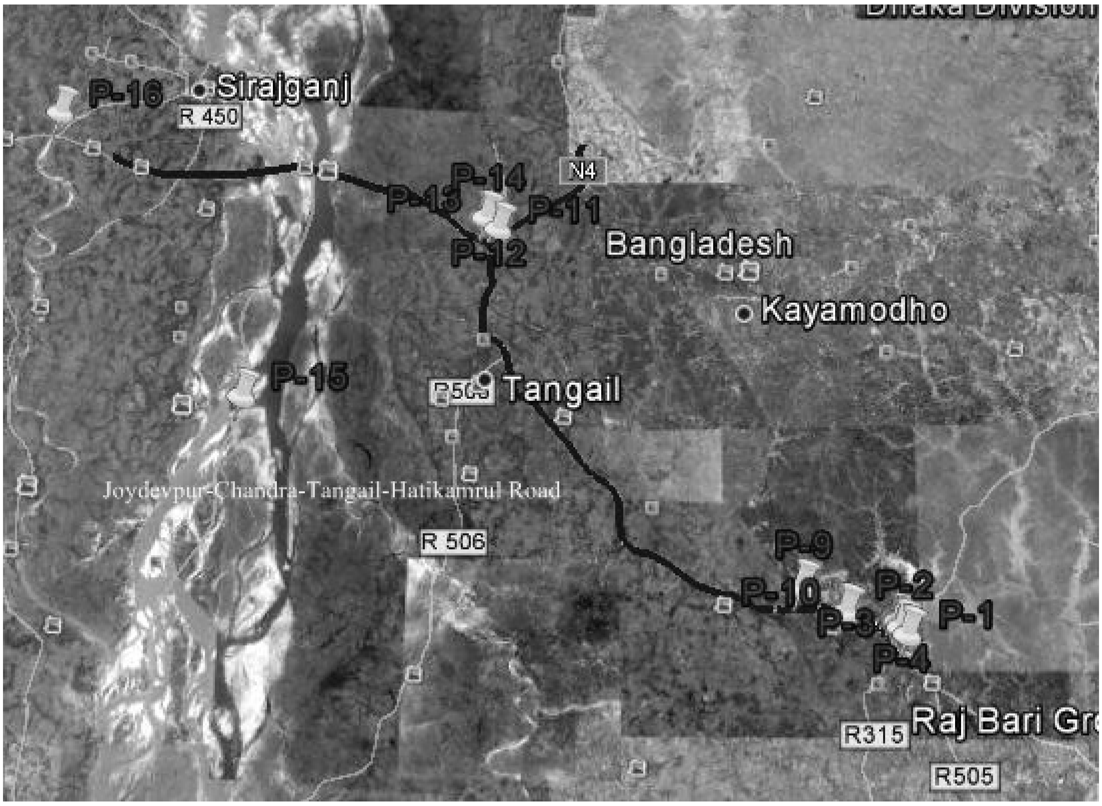

Figure.1 Satellite image of 16 stations along the JCTH road.

The analysis of the water bodies was done as per the standard methods of APHA (1989) and Trivedy and Goel (1986).The calculation of WQI was made using a weighted arithmetic index method. (Brown et. al.1972**).** Finally assessment of surface water quality based on water quality index was done. Table 1 shows the details of analysis methods and necessary equipments. Measured values of five parameters at 16 stations are given in appendix table 1. Also table 2 in appendix represents the maximum, minimum and average value of different parameters.

Table 1: Details of the analysis methods and the required equipment for the physic-chemical parameters

Serial Number	Temperature	Methodology	Equipments
1	Temperature	Visible	Centigrade Thermometer
2	Salinity	Visible	Sensaso-CL 410,HACH,USA
3	pH	Visible	Sensaso-CL 410,HACH,USA
4	Transparency	Visible	Secchi Disk
5	Dissolved Oxygen	Visible	Dissolved Oxygen Meter (Model-YK22 DO),USA
6	BOD	Laboratory	Dissolved Oxygen Meter (Model-YK22 DO),USA
7	Conductivity	Visible	Conductivity Meter (Model-CD4302,USA)
8	TDS	Visible	Sensaso-CL 410,HACH,USA

3. WQI CALCULATION

For calculation of WQI, selection of parameters has great value. The water quality index will widen if too many parameters are being used, importance of various parameters depends on the intended use of water, five physiochemical parameters namely - pH, TDS, EC, DO, BOD were used to calculate the WQI. The calculation of WQI was made using a weighted arithmetic index method given below (brown et al., 1972) in the following steps.

3.1 Calculation of Sub Index of Quality Rating (q_n)

Let there be n water quality parameters where the quality rating or sub index (q_n) corresponding to the nth parameter is a number reflecting the relative value of this parameter in the polluted water with respect to its standard permissible value. The value of qn is calculated using the following expression.

$$q_n = 100[(V_n - V_{io}) / (S_n - V_{io})] \quad (1)$$

Where,
q_n = quality rating for the nth water quality parameter.
V_n = estimated value of the nth parameter at a given sampling station.
S_n = standard permissible value of nth parameter
V_{io} = ideal value of nth parameter in pure water.

All the ideal values (V_{io}) are taken as zero for drinking water except for pH = 7.0 and dissolved oxygen=14.6mg/L. (*Journal of Environmental Hydrology, Volume 14 Paper 23 November 2006)*

3.2 Calculation of Quality Rating for pH

For pH the ideal value is 7.0 (for natural water) and a permissible value is 8.5 (for polluted water). Therefore, the quality rating for pH is calculated from the following relation:

$$q_{pH} = 100\ [(V_{pH}\ -7.0)/(8.5\ -7.0)] \quad (2)$$

Where,
V_{pH} = observed value of pH during the study period.

3.3 Calculation of Quality Rating for Dissolved Oxygen

The ideal value (V_{DO}) for dissolved oxygen is 14.6 mg/L and standard permitted value for drinking water is 5 mg/L. Therefore, quality rating is calculated from following relation:

$$q_{DO} = 100\,[(V_{DO} - 14.6)/(5 - 14.6)] \quad (3)$$

Where,
V_{DO} = measured value of dissolved oxygen

3.4 Calculation of Unit Weight (W_n)

Calculation of unit weight (W_n) for various water quality parameters are inversely proportional to the recommended standards for the corresponding parameters.

$$W_n = K/S_n \quad (4)$$

Where,
W_n = unit weight for nth parameters
S_n = standard value for nth parameters
K = constant for proportionality

3.5 Calculation of WQI

WQI is calculated from the following equation:

$$WQI = \sum_{n-1}^{n} q_n W_n \Big/ \sum_{n-1}^{n} W_n \quad (5)$$

4. RESULT AND DISCUSSION

4.1.1 *pH*

pH is one of the most important factors that serves as an index for the pollution. The experimental water bodies were found to be approximately neutral or slightly alkaline. The highest value of pH was 10.5 at station P-9 and lowest was 7 at P-14. The mean value of pH was 8.29. A pH between 6.7 and 8.4 is suitable, while pH below 5.0 and above 8.3 is detrimental. In the present investigation pH values were within the ICMR standard which is 7.0~8.5 (Sisodia, 2006*)* in all 16 stations. Maximum Sub water quality index for pH is found 53 at P-9 station. Minimum Sub water quality index for pH was found 0 at station P-14.Values of sub index water quality of different parameters are shown in appendix in table 4.

4.1.2 *Total dissolved solids*

The TDS level found to fluctuate from 2.12 mg/l to 779 mg/l within the water bodies. The TDS content was maximum at P-1station and minimum at station P-9 with an average of 210.56 mg/l. The amounts of total solids are influenced by the activity of the plankton and organic materials. Slightly high value of TDS were recorded at only one sampling stations and other values were less than the WHO limit. Water containing more than 500 mg/L of TDS is not considered desirable for drinking water supply. Only station P-1 contained TDS more than 500 mg/L so it can be said easily that most water bodies are suitable for domestic usage. Maximum Sub water quality index for TDS is found 1 at P-1, 16 no station. Minimum Sub water quality index for TDS is found 0 at rest of the 14 stations.

4.1.3 *Dissolved oxygen*

The value of DO varied from 0.5 mg/l to 10.5 mg/l. The maximum DO value (9mg/l) was recorded in P-4 and minimum value (0.5mg/l) was recorded in P-9. The mean value of DO was 6.19 mg/l. Concentrations below 5 mg/L may adversely affect the performance and survival of biological communities and below 2 mg/L may lead to fish mortality. Out of 16 stations 4 stations were found having DO less than 5 mg/L and out of the 4 stations 2 of them(Station P-9 and P-13) were found deadly as they contain 0.5mg/L and 0.7 mg/L DO respectively. Water without adequate DO may be considered wastewater. So 12 stations' water can be considered as good enough for domestic use. Maximum Sub water quality index for DO was found 56 at P-13and minimum sub water quality index for DO was found 16 at P-4.

4.1.4 *Biochemical oxygen demand*

BOD varied between 1.6 mg/l to 7.3 mg/l among the different sampling stations. The minimum values were found at stations P-12 and P-13. The Maximum value was recorded in P-4. The mean value of BOD was 4.23 mg/l. Maximum Sub water quality index for BOD was found 56 at P-4. Minimum Sub water quality index for DO was found 0 at P-11.

4.1.5 *Electrical conductivity*

Conductivity is measured in terms of conductivity per unit length, and meters typically use the unit micro siemens /cm. The values of water conductivity (2ms) varied from 4.1 µs /cm to 670 µs /cm among the water bodies. The value of conductivity was recorded lowest in P-4 and maximum in P-9. The mean value was 365.17µs/cm. Sub water quality index for Electrical Conductivity is 0 at all stations.

Standard and ideal values of different water quality parameters have been shown in table 3 in appendix.

4.2 Assessment of Water Quality Based on WQI

WQI has been classified into 5 classes quoted by Mishra and Patel, 2001. Table.2 represents the 5 classes of water quality based on WQI.

Table 2: Status of water quality based on WQI (Mishra and Patel, 2001)

Water quality index	Status
0-25	Excellent
26-50	Good
51-75	Poor
76-100	Very poor
Above 100	Unsuitable for drinking and propagation of fish culture

The observed range of water quality index along the road in pre monsoon is 45 to 109. Maximum WQI was 109 at station P-9 and minimum is 45 at station P-14. Not a single station's water quality can be expressed as excellent as the minimum WQI is 45. Only water quality of station P-14 can be called as good water. P-11, P-12 have been classified as poor water. Rest of the stations except P-9 have been classified as containing very poor water, but all of them can be used for domestic purpose by taking proper disinfection procedure, where as P-9 has been classified as unsuitable for drinking and all the other domestic use. P-9 station is located under bridge. Local people use it as open defecation place. For this reason the WQI value was found unacceptable.Table.3 shows the WQI values of the 16 stations measured in pre monsoon period. Stations with WQI values more than 90 can be classified as unsuitable for both domestic and aquaculture purposes. 5 of the stations turned out to be unsuitable as there WQI values are near 90 or more.

Table 3: Location wise calculated values of Water Quality Index for pre monsoon period

Serial	Name of the station	WQI
P-1	Shitalakha	77
P-2	Shimultali jamiderbari pokur (Left)	89
P-3	Lal matir pokur (Right)	78
P-4	Low land (Left)	93
P-5	Pokur (Right)	94
P-6	Hotel sided pokur (Right)	93
P-7	Low land (left)	76
P-8	Under bridge	89
P-9	Under bridge	109
P-10	Pungli	79
P-11	Low land (Left)	56
P-12	Pokur (Left)	64
P-13	Pokur (Left, kachuri)	79
P-14	Pokur (Left)	45
P-16	Jamuna	77
P-16	Phulzore	76

5. CONCLUSION

WQI have some margins as it may not carry enough information about the actual situation of the water bodies. Also many other uses of water quality data cannot be met with an index. Despite of having such problem WQI are more recompense than its drawback. An index is tool for "communicating water quality information to the public and to legislative decision makers;" it is not "a complex predictive model for technical and scientific application" (McClelland, 1974). Physico Chemical parameters of the statyions along the WQI for the pre monsoon season was found high, as concentration of water quality parameters are maximum during pre monsoon. It is found from the calculation that parameter which shows the highest favorable value gives a low statistical value to the index. BOD, DO was found to be the most important parameter as it contributes the most for the WQI calculation among the five parameters. Out of the 16 stations almost 10 stations were found suitable for domestice and aquaculture pusrpose. So if proper treatment is done then all the 16 water bodies could become usefull and could help people in rural areas during time of crysis.

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Appendix

Table 1: Measured values of five parameters at 16 stations

Serial	Name of the station	Area (decimal)	GPS	pH	DO (mg/l)	TDS	Conductivity	BOD
P-1	Shitalakha	Cannel	N24°03·916′ E90°12·906′	8.05	7.8	779	157.5	4.3
P-2	Shimultali jamiderbari pukur (Left)	33	N 24°04125′ E90°12·701′	8.39	5.1	196.6	509	3.9
P-3	Lal matir pukur (Right)	100	N 24°04207′ E90°12·690′	8.02	6.8	219	454	4.2
P-4	Low land (Left)	Beel	N24°04·360′ E90°12·587′	8.39	10.5	325	670	7.3
P-5	Pokur (Right)	80	N 24°04352′ E90°12·619′	8.56	9.0	252	522	6.2
P-6	Hotel sided pukur (Right)	120	N 24°04558′ E90°14·619′	8.39	7.0	245	524	5.5
P-7	Low land (left)	Beel	N24°04·620′ E90°12·497′	7.45	6.4	252	522	4.7
P-8	Under bridge	Barrow pit	N 24°04699′ E90°12·575′	8.32	5.6	179.8	376	4.3
P-9	Under bridge	Cannel	N24°05·740′ E90°08·510′	10.5	0.5	2.12	4·10	3.7
P-10	Pungli	River	N24°13·468′ E89°57·854′	8.33	5.3	232	481	2.9
P-11	Low land (Left)	Barrow pit	N24°20·355′ E89°55·459′	8.72	7.0	107.2	224	3.6
P-12	Pukur (Left)	120	N24°20·823′ E89°54·880′	8.91	8.8	86.6	181·6	1.6
P-13	Pukur (Left, kachuri)	80	N24°20·823′ E89°54·880′	7.73	0.7	85	177	1.6
P-14	Pukur (Left)	110	N24°20·823′ E89°54·880′	7.0	6.5	178	376	1.7
P-15	Jamuna	River	N24°23·944′ E89°44·347′	8.14	4.9	113.6	237	2.6
P-16	Phulzore	River	N24°25·413′ E89°35·460′	7.8	4.8	116	244	3.2

Table 2: Maximum, minimum and average values of diffrent water quality parameters

Groups	Maximum	Minimum	Average
Air Temperature(ºC)	27.1	22.6	25.6
Water temperature(ºC)	30.4	24.9	28.88
Secchi depth(cm)	115	5	31.81
pH	10.5	7	8.29
DO(mg/l)	10.5	0.5	6.19
BOD(mg/l)	7.3	1.6	4.23
Conductivity(2ms)	670	4.1	365.17
Salinity(mg/l)	2.2	0.1	0.3
TDS	779	2.12	210.56

Table 3: Drinking water standards and unit weight

Parameter	Standard value (S)	Ideal value	1/S	Assigned weightage factor
p^H	8.5	7	0.117647	0.226087822
TDS	500	0	0.002	0.003843493
EC	1400	0	0.000714	0.001372676
DO	5	14.6	0.2	0.384349297
BOD	5	0	0.2	0.384349297
K			0.52036	1

Table 4: Sub water quality Index of the physico-chemical parameters

Serial	Name of the station	Sub water quality Index(pH)	Sub water quality Index (TDS)	Sub water quality Index (EC)	Sub water quality Index (BOD)	Sub water quality Index (DO)
P-1	Shitalakha	16	1	0	33	27
P-2	Shimultali jamiderbari pokur (Left)	21	0	0	30	38
P-3	Lal matir pokur (Right)	15	0	0	32	31
P-4	Low land (Left)	21	0	0	56	16
P-5	Pokur (Right)	24	0	0	48	22
P-6	Hotel sided pokur (Right)	21	0	0	42	30
P-7	Low land (left)	7	0	0	36	33
P-8	Under bridge	20	0	0	33	36
P-9	Under bridge	53	0	0	0	56
P-10	Pungli	20	0	0	22	37
P-11	Low land (Left)	26	0	0	0	30
P-12	Pokur (Left)	29	0	0	12	23
P-13	Pokur (Left, kachuri)	11	0	0	12	56
P-14	Pokur (Left)	0	0	0	13	32
P-16	Jamuna	17	1	0	20	39
P-16	Phulzore	12	0	0	25	39

THE REMOVAL OF COD OF REACTIVE DYES BY COAGULATION USING ALUM

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ABSTRACT

Textile manufacturing units involved in finishing processes are profuse water consumers. The effluents from these plants are major source of water pollution. This study measures the influence of pH and coagulant dosage on the effectiveness of COD removal of reactive dyes. Alum is used as a coagulant for the removal of Chemical Oxygen Demand (COD) of selective reactive dyes, which are used for cotton yarn dyeing, at different doses. The results indicate that the removal of COD of two dyes Reactive Red 6BL & Reactive Orange 3R are 55.65and 65.657 percent, respectively at their optimum doses and 74.02729 and 92.238 percent, respectively at their optimum pH & doses, respectively. The effect of initial COD concentration was also investigated and the COD removal was characterized by Langmuir approach.Thje floc formation mode was also analyzed at various pH and coagulant dosesb for both dye.

Keywords: *Alum, Coagulant, Reactive dye removal, COD, Coagulation / Flocculation.*

1. INTRODUCTION

Day by day the rapid and steady up growing trend of gradual industrialization and urbanization leading more & more production of waste material and exposing it to the environment. The introduction of these waste products in the environment is a worldwide problem that has been highlighted by various environmentalist groups.Dye is considered to be particularly one of the most dangerous organic compounds for the environment.

Dyes are widely used in many industries such as textile, rubber, paper, plastic, cosmetic etc. Among them, textile ranks first in usage of dyes (Saiful Azhar et al., 2005). Presently, more than 10,000 of different commercial dyes and pigments are available (Eren and Acar, 2006; Ozer et al., 2006), and more than 7×10^5 tons per year are produced world wide (Crini, 2006; Saiful Azhar et al., 2005). The amount of dyes discharged into the environment from the textile industry is about 146,000 tons per year (Marc, 1996) Reactive dyes are the most commonly applied, among more than 10,000 dyes applied, in the textile processing industries (Vander zee, et al., 2001).This type of dyes are particularly problematic in dye wastewaters as many of these are resistant to biodegradation processes (Grau, 1991). Presence of very small amounts of dyes in water (less than 1 ppm for some dyes) is highly visible and undesirable (Crini, 2006).Two percent of dyes that are produced are discharged directly in aqueous effluents (Crini, 2006). These colored compounds are not only aesthetically displeasing, but they also impede light penetration, retard photosynthetic activity and inhibit the growth of biota. Some dyes are also toxic and carcinogenic (Eren and Acar, 2006).

The selection of proper methods which ensure effective colour removal from textile wastewater is very complicated. Data in literature show that biological methods employed for colour removal are ineffective. For this reason supportive physico-chemical methods are often used. Although one of the most frequently employed methods is coagulation, little is known about the optimal operation conditions of the process and particularly about the influence of different types of dyes on coagulation effectiveness. They lead to greater public concern and present legislation problems.

Coagulation-flocculation is always used in conjunction with and preceded by coagulation. The process of coagulation allows particle agglomeration and enhances subsequent particle, this process is complex and may be involve several mechanisms to achieve destabilization of negatively charged particles, it result a repulsive force that tend to stabilize the suspension matter and prevent particle agglomeration Semerjian et Ayoub (2003). Flocculation is the physical process of bringing the destabilized particles in contact to form larger flocks that can be easily removed from suspension. At Last the treated wastewater strained through a filtration system in order to separate aqueous phase from solids matters Mesdaghinia et al (2005).

Chemical Oxygen Demand (COD) is a measure of oxygen requirement of a sample that is susceptible to oxidation by strong chemical oxidant. Combined methods, such as activated sludge + coagulation activated sludge + adsorption and coagulation + chemical oxidation are being used by most dyeing industries in the effluent treatment (Lin and Chen, 1997, Popali et al., 2002). Ferric chloride, aluminum chloride, and lime + ferrous sulphate are widely used as coagulants for dye colour removal with varying degrees of success (Popoc, et al., 2000, Popali et al., 2002, Gurses et al., 2003). However, no investigation has been made to optimize coagulant dose which depends highly on the extent of soluble colour contributing COD. However, this parameter of textile waste is most problematic compared to others and removal of it from wastewater is a major environmental challenge. Hence, there is a constant need to have an effective process that can efficiently remove these CODs. Also, tightening government legislation is enforcing to treat textile wastewater to an increasingly high standard.

The objective of the present investigation is to assess the amenability and response of chemical coagulation of two selective reactive dyes using alum as a coagulant. The study focused on the effect of different parameters such as pH, coagulant dosage, and initial COD concentration on the dye removal efficiency for chemical coagulation of two selective reactive dyes as model, generally used for yarn dyeing. Also the COD removal was characterized by Langmuir approach. The effectiveness of coagulant is an indicator of floc formation and settling characteristics. The floc formation mode with different condition of coagulation was also studied by using alum as a coagulant.

2. MATERIALS AND METHODS

2.1 Coagulant:

A commercial grade Alum (*Aluminium sulphate*) was collected from local market and used as coagulant with varying doses of 0.1 to 4.0 g/L.

2.2 Reactive dyes

(i)Reactive Red 6BL, (ii) Reactive Orange 3R are used as model dyes. These dyes are extensively used for cotton yarn dyeing. Dye solutions are prepared in distilled water to a strength of 1000mg/L for both dye. The COD of all the dyes are estimated as per the standard procedures as described in next section.

2.3 Experimental method

2.3.1 Chemical Oxygen Demand (COD) measurement:

The COD was measured by titrimetric method. This includes washing of ampoules and the caps with 20% H_2SO_4 to prevent contamination followed by addition of 2.5ml sample, 1.5ml digestion solution(0.4913g $K_2Cr_2O_7$ (preheated at 103^oC for 2hr) in 50ml distilled water + 16.7ml H_2SO_4 + 3.33g $HgSO_4$ + cool + dilute to 100ml) and 3.5ml H_2SO_4 reagent(0.275g $AgSO_4$ + 7ml H_2SO_4). Then the ampoules were sealed and shaked vigorously to mix it well. These ampoules were heated in oil bath at 145-155^OC for 2hr cooling to room temperature. Then the solutions were titrated against FAS standard solution(1M) (3.92g $Fe(NH_4)_2(SO_4)_2.6H_2O$ in distilled water + 2ml conc. H_2SO_4 + cool and dilute to 100 ml) (this solution was prepared in time of use) with 1-2 drops Ferrion indicator(0.7425g 1, 10-phenanthroline monohydrate + 0.347g $FeSO_4$ in distilled water and dilute to 50ml) until the colour changed (when indicator used it becomes blue green to reddish brown, after titration reddish brown colour changes into blue green colour). In the same manner, a blank containing the reagent and a volume of distilled water equal to that of the sample was titrated. The COD was calculated by following equation:

$$\text{COD (mg } O_2 \text{ /L)} = \frac{(b-s)\,M}{8000} \times dilution\ factor \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Where,
b, s, M are ml FAS used for blanks, ml FAS used for sample, molarity of FAS respectively.

2.3.2 Coagulation study:

Coagulation studies were conducted in duplicate using Jar-test Apparatus with six beakers of one litre capacity. A 500ml. dye solutions were taken in each beaker and different doses of Alum solutions were added. The samples were stirred for three minute at 250-300 rpm followed by 60 minutes slow mixing of 50 to 70 rpm. The contents are then settled for one hour. At the end of one hours, the supernatant is with drawn, filtered and was used for COD analysis .All the experiments were conducted at room temperature of 27 ± 3^{0}C.

Percentage of COD removal was calculated by Eq. (2).

$$percent\ removal\,(\%) = \frac{COD_r - COD_t}{COD_t} \times 100 \ldots\ldots.. (2)$$

Where, COD_r and COD_t are the COD of the dye in raw and treated solutions, respectively.

3. RESULTS AND DISCUSSIONS

The result of the COD analysis of the two dyes used in the experimental system is presented in Table-1. The results indicate that exertion of COD for different reactive dyes is highly varied depending upon the dye constituents such as phenol derivatives, organic acid and benzene derivatives.

Table 1: COD of the experimental reactive dyes

Dye	Concentration(mg/L)	COD(mg O_2/L)
Reactive Red 6BL	100	77.20
Reactive Orange 3R	100	176.68

The results of the Alum coagulation on COD removal of both dyes are presented in Figures 1 to 4. The results indicate that the removal of COD for both dye is highly varied, indicating that the solubility of these dyes in water is quite different. The performance of COD removal by chemical coagulation depends upon the solubility of the dyes in water.

In the first series of the experiment the influence of pH and coagulant dosage on the effectiveness of COD removal was examined. Concentration of the tested dyes was constant along the whole series, i.e. 100 mg/dm^3 at each examined coagulant dosage. The applied dosages of coagulant ant the resulted COD removal for individual dyes are presented in Figure: 1(a) and Figure: 1(b) respectively.

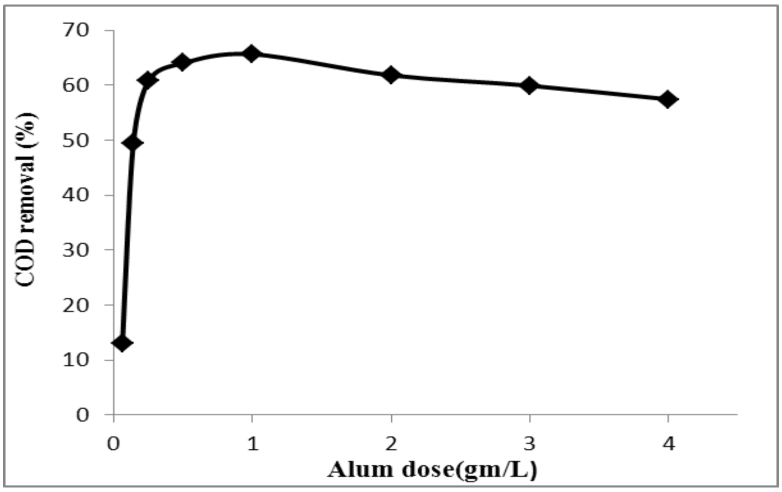

Figure 1(a): COD removal efficiency (%) for Orange 3R dye with various initial alum doses.

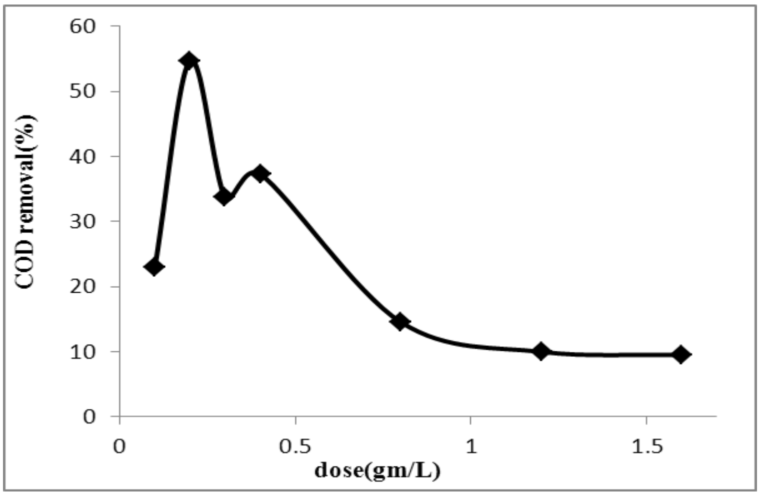

Figure 1(b) : COD removal efficiency (%) for Reactive Red 6BL dye with various initial alum doses.

pH plays an important role in coagulation/flocculation process using inorganic coagulants. Charge on hydrolysis products and precipitation of metal hydroxides are both controlled by pH variations. According to Sthendal et al. (1995) the stability of colloidal aggregates of soluble particles results from intermolecular forces which hold the particles in suspension. These pollutants cannot agglomerate unless the pH is adjusted to the isoelectric point. We can therefore assume that for a particular type of wastewater there is an optimal range of pH value in which the coagulation process is the most effective. Thus, pH must be controlled to establish optimum conditions for coagulation (Li and Gregory, 1991).

To study the effect of pH on COD removal efficiency for Reactive Red 6BL dye, dosages of Alum and dye concentration were kept constant at 0.200mg/L and 100 mg/L respectively, while varying pH of the samples using H_2SO_4 and NaOH. Dye concentration was 100 mg/l for all solutions during the experiment. The effect of pH on dye removal is shown in Figure: 2 where for reactive Orange 3R dye Alum & dye concentration was 0.5 gm./L & 100 mg/L respectively .

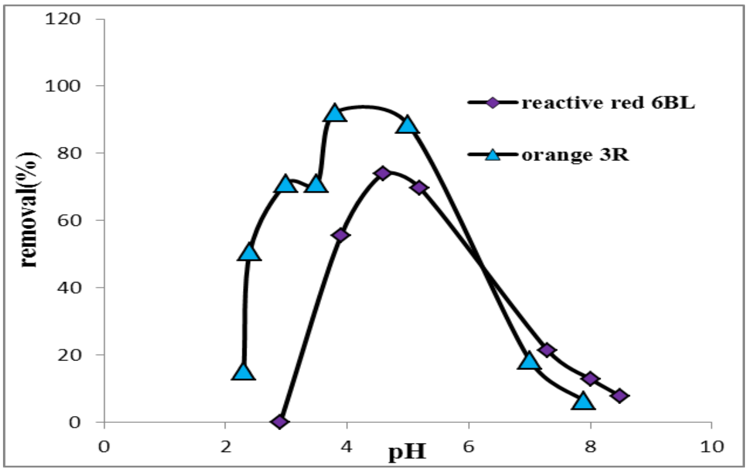

Figure 2 : COD removal efficiency (%) for Reactive Red 6BL an Reactive Orange 3R dye with various operating p^H.

In the third series of the study experiments were performed determining the influence of initial dye concentrations on COD removal efficiency for both dye at constant pH and optimal alum dosage. The conditions of the experiment are presented in Table 3.The results are shown in Figure: 3(a) and Figure: 3(b).

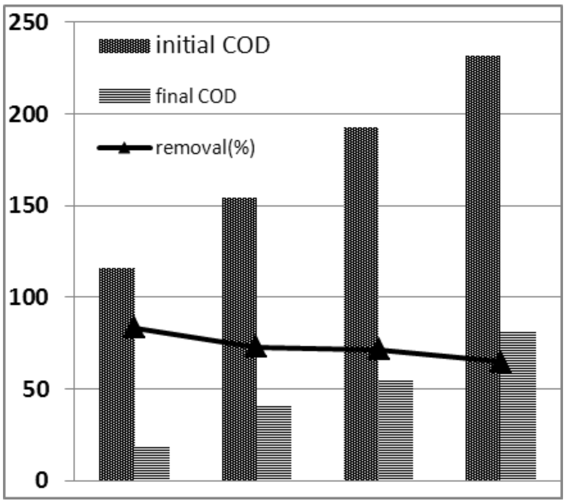

Figure 3(a): Effect of initial COD concentration on COD removal at optimum pH and Alum dose for Reactive Red 6BL.

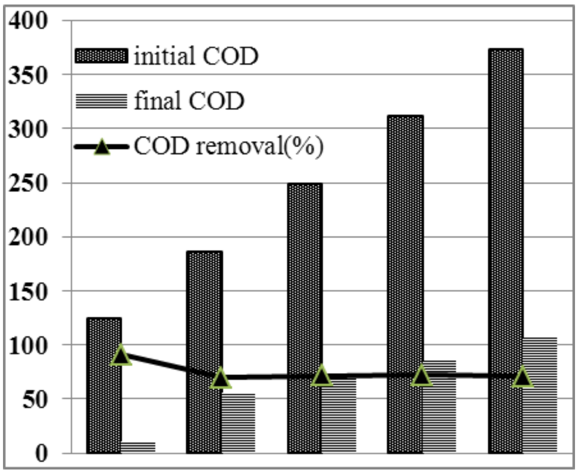

Figure 3(b): Effect of initial COD concentration on COD removal at optimum pH and Alum dose for Orange 3R.

In addition, the variations of the amount of the removed COD per unit mass of coagulant (Q), versus the initial COD for both dyes are also analyzed. The results are illustrated in Figure: 4(a) & (b).

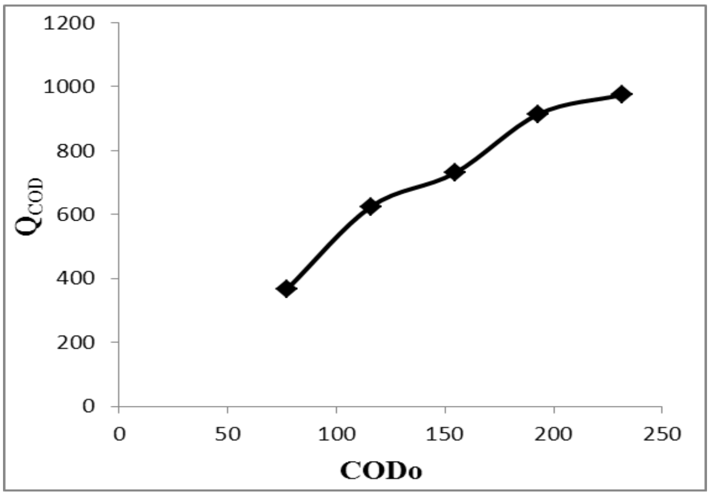

Figure 4(a): Effect of Initial COD for reactive red 6BL.

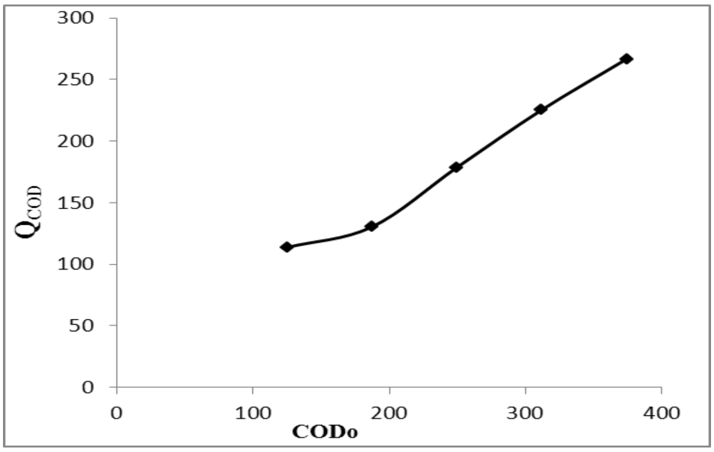

Figure 4(b): Effect of Initial COD for reactive Orange 3R.

For alum, with the increase of initial COD concentration from 124.74 mg O_2/L to 374.22 mg O_2/L & from 77.25 mg O_2/L to 231.75 mg O_2/L for Reactive Red 6bl and Orange 3R dye the Q_{COD} for both dyes also increases as shown. Here, the both curves relatively showed some changing trends. This might may change in their structure and existence of various groups.

The results of the experiments showing the relation between the initial concentration of COD (COD_o) and the amount of the COD removed calculated per unit mass of Alum (Q_{COD}) are presented in Figure 4(a) & (b) are treated with Langmuir's equation and used for the approximation of the measurement points in the case of Reactive Red 6BL & Reactive Orange 3R.

$$Q_{COD} = \frac{Q_{max} \cdot K_c \cdot COD_o}{1 + K_c \cdot COD_o} \text{......... (3)}$$

Where, Q_{COD}, Q_{max}, K_c are the COD removed per unit weight of Alum (mg/mg Alum), the maximum mass of dye removed per unit weight of Alum (mg/mg Alum), and constant in Langmuir's equation (dm^3/mg) and is a direct measure of the intensity of the process, energy of sorption (L/mg) respectively.

Linearization of this equation gives:

$$\frac{1}{Q_{COD}} = \frac{1}{Q_{max}} + \frac{1}{Q_{max} \cdot K_C} \cdot \frac{1}{COD_O} \text{......... (4)}$$

This equation is plotted for both dye in Figure 5(a) & (b) and the determined parameters are listed bellow:

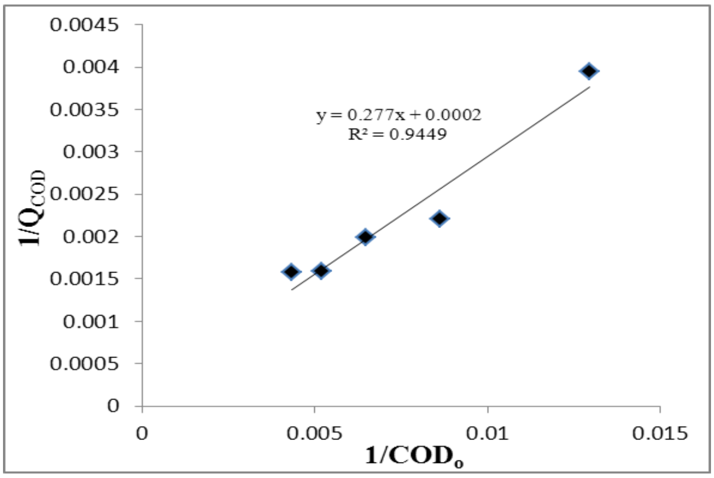

Figure 5(a): Langmuir sorption model for COD removal of Reactive Red 6BL.

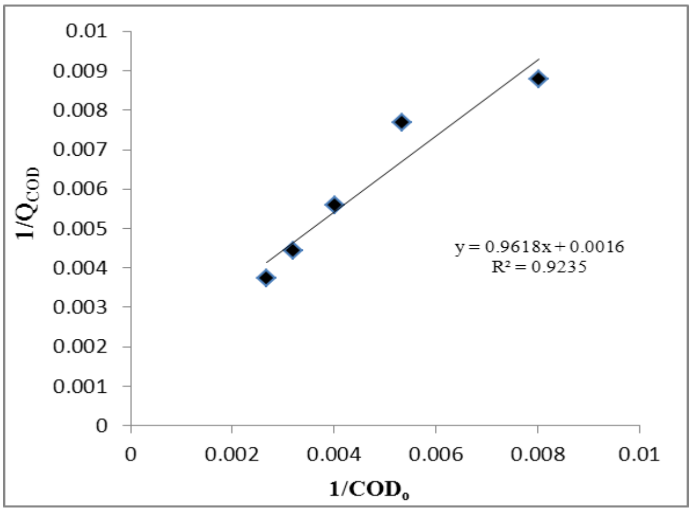

Figure 5(b): Langmuir sorption model for COD removal of Reactive Orange 3R.

Table 2: Langmuir sorption model parameters:

Dye	Q_{COD} (mg./gm)	K_C (L/mg)	**R^2**
Reactive Red 6BL	4998.90	0.000722022	0.9449
Reactive Orange 3R	625	0.001663548	0.9235

The performance of COD removal by chemical coagulation depends upon the solubility of the dyes in water. The reactive dyes which are highly soluble in water can not be easily flocculated by coagulants (Al degs, et al., 2000). This is quite evident in floc formation in the present investigation. The nature of floc formation for different doses of Alum with different operating pH is shown in Table-3 and the optimum dose of Alum and operating pH with the percent removal of Chemical Oxygen Demand (COD) are given in the Table-4.

Table 3 : Nature of Floc formations for different doses of Alum & different pH.

<table>
<tr><th colspan="4">Nature of Floc formations for different doses of Alum</th><th colspan="4">Nature of Floc formations for different operating p^H</th></tr>
<tr><td>SL No.</td><td>Dye</td><td>Dose of Alum (gm/L)</td><td>Nature of flocs</td><td>SL No.</td><td>Dye</td><td>Operating pH</td><td>Nature of flocs</td></tr>
<tr><td rowspan="3">1</td><td rowspan="3">Reactive Red 6BL</td><td><.10</td><td>Very low floc formation .</td><td rowspan="3">1</td><td rowspan="3">Reactive Orange 3R</td><td><2.00 .</td><td>No floc formation</td></tr>
<tr><td>0.15-0.25</td><td>Slightly visible flocs with settlement.</td><td>3.8-5.0</td><td>visible flocs with good settlement.</td></tr>
<tr><td>>1.00</td><td>Highly turbid with no settlement.</td><td>>8.5</td><td>No floc formation.</td></tr>
<tr><td rowspan="4">2</td><td rowspan="4">Reactive Orange 3R</td><td><.10</td><td>Very low floc formation .</td><td rowspan="4">2</td><td rowspan="4">Reactive Red 6BL</td><td><2.00</td><td>No floc formation.</td></tr>
<tr><td>0.2-2.0</td><td>visible flocs with good settlement.</td><td>2.4-3.5</td><td>Slightly visible flocs with little settlement.</td></tr>
<tr><td rowspan="2">>4.5</td><td rowspan="2">Highly turbid with no settlement.</td><td>3.7-5.2</td><td>Slightly visible flocs with good settlement.</td></tr>
<tr><td>>8.00</td><td>No floc formation.</td></tr>
</table>

Table 4: Optimum dose of Alum & optimum pH and percent removal of COD.

Serial .No.	**Dye**	**Optimum dose(gm Alum/L)**	**% COD removal**
1	Reactive Red 6BL	0.2	55.65
2	Reactive Orange 3R	1.0	65.657
Serial No.	**Dye**	**Optimum p^H and alum dose**	**% COD removal**
3	Reactive Red 6BL	4.6	74.02729
4	Reactive Orange 3R	3.8	92.238

4. CONCLUSIONS

In this paper, affectivity of the removal of COD from model waste water with two dyes (Reactive Red 6BL & Reactive Orange 3R) has been analysed. This study shows that the COD removal for two type of dye varies according to their type, structure and presence of various phenolic groups. The optimum pH and alum dose for the coagulation process was found to be 4.6, 3.8 and 0.2, 1.0 gm. /L for reactive Red 6BL and Reactive Orange 3R respectively. The nature of the floc formation with various operating condition was analyzed.

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FIRE VULNERABILITY ASSESSMENT OF HIGH-RISE APARTMENTS IN CHITTAGONG CITY: A CASE OF RESIDENTIAL AREA

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ABSTRACT

Fire vulnerability assessment of high-rise apartments (above 6 Storey) is most significant for the safety of urban people's lives and property. A huge number of factors involve in the fire susceptibility assessment such as number of vulnerable population, availability of escape ways, accessibility to fire car, active and passive fire suppression equipments etc. In case of fire safety of high-rise building, some factors (accessibility of fire trucks, active fire suppressing system) are most significant. To get a present real picture of fire safety in Chittagong city, Bangladesh a study was conducted in high-rise residential area of Nasirabad Housing Society, Sugandha, Khulshi, Mehedibag and O.R. Nizam Road. In this study a fire vulnerability map was created for the high-rise apartments of the above selected area. To make a vulnerability map here some steps have been applied. At first, five maps were prepared by using five attributes namely accessibility, transformer and power lines, open space in between two apartments, emergency exit and fire alarm and in this case ARC VIEW software was used. After that, on the basis of expert opinion, using Analytical Hierarchy Process (AHP), maps were overlaid and vulnerability map was prepared with some parameters. This represents how much an apartment is vulnerable compare to another. Finally, the overall vulnerability of the apartments was assessed and it was found that high percentage of the apartments is highly vulnerable. In spite of it all, this study could provide a guideline to mitigate the fire safety problem of high-rise apartments.

Keywords: *Fire, Chittagong, high-rise, apartments, vulnerability, fire safety, AHP*

1. INTRODUCTION

Fire hazard is a great threat to lives and property of high-rise buildings (Lei, Z. et. al, 2009). At first view, a critical component in fire safety of high-rise building is the evacuation of people from the hazardous zone of the buildings which will reduce the ill effects of disaster (Lo, S. M, et. al, 2009). Chittagong City has been experiencing high rate of urbanization since the last few decades. Presently, more than 4 million people are living in Chittagong City. The development trend of Chittagong City is deliberately shifting vertical direction to cope with the extensive population pressure. Nowadays, high-rise buildings are being constructed in every parts of the city but in most cases dwellings are being constructed without maintaining the planning rules and regulations. Fire safety is most important component of high-rise building safety. The most significant key element of building fire safety includes building a facility according to local building code, maintaining facility with the provisions of the fire code, maintaining proper fire exits and exit signs etc. Fire hazard vulnerability of Chittagong City dwellers has been increased due to reckless building construction and non conformation of Fire Protection Act, 2003. The City has been experiencing many fire accidents at present and in most cases lack of proper precautionary measures along with the institutional inefficiency, insufficient equipment support and lack of public awareness are causing this situation more complex. The statistics showed the number of fire incidents was 778 in 2007, 836 in 2008, and 1053 in 2009 and extent economic loss was 141383600, 393360066 and 341671500 respectively(FSCD, Chittagong 2010). Most of the dwellers do not know how to use the fire fighting equipments, in most cases all precautionary measures for combating fire generally go into vein. To estimate the fire vulnerability of high-rise apartments of the residential area (ward no 08 & 15) a vulnerability map has prepared on the basis of vulnerability score. AHP was applied to calculate the vulnerability score of the apartment. Fire safety management can be done by passive building construction and active fire protection system in the apartments. In the fire safety plan to maintaining the protection equipments must need staff training, fire prevention and evacuation procedures (Chow, W.K, 2004). Fire hazards can be avoided if proper fire safety is practiced. The best fire safety is to eliminate fire hazards.

2. OBJECTIVES

The first main objective of the current paper is to investigate the present fire safety condition of high-rise apartments in different residential area of Chittagong City. Secondly this paper aims to know the level of fire vulnerability of the apartments by using AHP of the aforesaid area.

3. RESULT AND DISCUSSION

3.1 Fire Safety Condition

To evaluate the fire safety condition, accessibility is one of the most important attribute of high-rise. Fire affected apartments are supposed to be served by fire engines coming from fire stations which carry water, ladder and various fire fighting equipments. If the roads leading to the affected apartment are not accessible to fire engines, the apartment cannot be served effectively by fire fighters and it become vulnerable to fire. In our country, fire fighting engines which required for high-rise apartments can go through roads at minimum 30 feet width. This information was taken from a fire fighting expert. So less than 30 feet wide road is not accessible for firefighting engines. The width is not same along the whole length of the road. For example at some point the road may be 30 feet wide but this same road may become 20 feet wide at another point. As a result fire engines cannot go through the road. That is why accessibility is an important factor for vulnerability assessment. In the study area it is found that the number of the apartments that get access is 9.6%. Rest of the apartments (90.4%) is not accessible for fire engines.

Open Space in between Multi-level Building is another important factor for the fire safety. Large spaces among the apartments are most important in fire detection and suppression of the fire safety engineering (Shi, C. L.; et.al. 2009).This is because if an apartment is affected by fire it can spread its neighboring apartment. From the field survey it is found that in many cases there was little space between apartments. So fire can easily spread up quickly in such densely built apartments. According to the fire expert “It is to be needed 15 feet distance among neighboring apartments.” The study reveals that around 82.7% apartments are vulnerable for densely built up in the study area and rest only 17.3% are not vulnerable. So a lot of apartments are vulnerable considering fire open space among neighboring apartments in the study area.

Another important attribute for the fire vulnerability of high-rise apartments is the location of transformers and power lines. This is selected because the connection of power lines at the electricity poles may be loose for many reasons. It can create problem to fight against fire affected apartments because Snorkel and Turntable Ladder appliances needs over headed open space. Besides these appliances mainly use to fight against fire in high-rise buildings but if over headed space in front of buildings is blocked by electricity line it’s impossible to rescue and fire fighting. Again transformers can explode because of high voltage and also by the heat of the fire affected buildings. That can also start a fire and also spread out to another apartment. The assumptions of fire expert in this case is that transformer is to be laid at 18 feet distance from the original apartment and over headed electricity lines not to be laid in front of original apartment**.** The study reveals that around 27.4% apartments are vulnerable considering over headed electricity lines where as 7.7% apartments adjacent to transformers are vulnerable.

At the time of fire accident fire fighters role is most significant. Authorities have got to know new and modern building designs to ensure all suitable level of fire safety of the inhabitants. They have to be well prepared for the low probability of high consequences in case of fire accidents and need necessary tools to help them to estimate the extent of fire in the buildings (Daniela, H., 2009). The location of fire fighting station is important attribute to the vulnerability assessment of the buildings because local community cannot take necessary steps to control high-rise building fire accident. Fire Service and Civil Defense (FSCD) have divided Chittagong City into several zones. Mehedibug and O.R Nizam Road residential area is under Chandanpura fire station zone. Nasirabad, Suganda and Khulshi hill’s residential area is under Chandanpura and Agrabad fire station zones. These residential areas distance from fire stations is within the range of 1 to 5 km. So these areas apartments have almost the same fire fighting scenario in terms of distance from the fire station. So it does not affect the vulnerability of high-rise apartments considering fire hazard.

In the high-rise building emergency exit is a special exit for emergencies such as a fire accident. At the time of fire hazard, emergency evacuation is a primary concern in the safety system design of high-rise buildings (Wong, KHL; et.al, 2005). In this case emergency exit is important attribute for the rapid evacuation and it also provides an alternatives if the rout to the regular exit is blocked by fire. That is why emergency exit is an important factor for the vulnerability assessment of high-rise apartments. In the study area it is found that 86.5%

apartments are without emergency exit and only 13.5% apartments having emergency exit. So lots of apartments are vulnerable because there is no way to escape from the apartments during fire disaster.

Automatic fire alarm system is an important part of a security to fire protection. This system provides real time observation, monitoring and automatic alarm as well as it sends early alarm when fire occurs. This helps to reduce the fire damage (Lei, Z. et.al, 2009). Besides at the initial stage of fire if residents of apartment perceive sound of this appliance they can escape from fire. I this study it is found that around 35.6% percent apartments having fire alarm and rest 64.4% apartments without fire alarm. A number of defective fire alarms also found in the study area during the field survey.

3.2. VULNERABILITY MAP PREPARATION USING ANALYTICAL HIERARCHY PROCESS (AHP)

Analytical Hierarchy Process (AHP) is a multicriterial decision-making theory developed by Thomas Satty that carefully delimitates the scope of the problem environment (Alphonce, C.B. 1996). It is based on the concrete mathematical structure of uniform matrices and the capability to create true or fairly accurate weights. This is a theory of measurement through pair wise comparisons. It depends on the judgment of experts to develop priority scales. These scales measure insubstantial in relative terms. By using a scale of absolute judgments, the comparison is prepared. This represents how much more; one element dominates another with respect to a given attribute (Saaty, 2008).

3.2.1 Superiority of AHP

Alphonce (1996) added three advantages of using AHP. These are:

- This technique promotes accurate judgments because it formalizes and generates systematic
- From this method, decision makers receive information about the absolute weights that are placed on the evaluation criteria, and
- This can be applying to conduct sensitivity analysis by using computer technology.

3.2.2 Use and Application of AHP

The Analytic Hierarchy Process (AHP) is most useful where people are working on complex problems, especially those with high stakes, involving human perceptions and judgments, whose resolutions have long-term repercussions. It has unique advantages when important elements of the decision are difficult to quantify or compare, or where communication among team members is impeded by their different specializations, terminologies, or perspectives. Scientist Alphonce (1996) uses the AHP in the following sectors:

a. Determination of farm areas for each of the food crops in a farm land;
b. Resource allocation for agricultural activities (horticulture, dairy farming, food crops production and cash crops production);
c. Identification of the best location of village store is another application of AHP considering accessibility, distance from the furthest farmer (distance), security of stored crops (security), availability of water and building materials in the neighborhood (basic essentials) etc;
d. Determination of the crop production technology;
e. Subsistence farming and cash crops production.

Decision situations to which the AHP can be applied include;

According to Wind and Saaty (1980), the AHP is applied in marketing decisions of a firm. Three specific marketing applications are:

a. The determination of a target product, market and distribution range.
b. Generation and evaluation of new product concepts.
c. Marketing mix determination

According to Mustafa and Al-Bahar (1991), the AHP provides a flexible and easily understood way to analyze project risk and assist the contractor in the evaluation of the riskiness of the project. It also use in construction management, evaluation of bidder or biddings and in the selection of the best crashing scheme when factors other than time and cost are considered.

3.2.3 Operating Procedure of the AHP

To make a decision in an organized way to generate priorities it need to decompose the decision into the following steps.

Identify the problem and find out the kind of knowledge sought

↓

Structure the decision hierarchy from the top with the goal of the decision

↓

Construct a set of pair wise comparison matrices. Each element in an upper level is used to compare the elements in the level immediately below with respect to it.

↓

Use the priorities obtained from the comparisons to weigh the priorities in the level immediately below. Do this for every element.

↓

Then for each element in the level below add its weighed values and obtain its overall or global priority. Continue this process of weighing and adding until the final priorities of the alternatives in the bottom most level are obtained.

Fig1: Different steps of AHP to make a decision, Saaty, T. L. (2008).

Saaty (2008), formulate a scale of numbers (1-9) to make comparisons that indicates how many times more important or dominant one element is over another element with respect to the criterion or property with respect to which they are compared. Table 1 shows the scale.

Table1: The fundamental scale (1-9) of absolute numbers

Importance	Definition	Explanation
1	Equal Importance	Two elements contribute identically to the objective
2	Weak or slight weak	
3	Weak dominance	Experience or judgment slightly favors one element over another
4	Moderate plus	
5	Strong importance	Experience and judgment strongly favor one activity over another
6	Strong plus	
7	Very strong or demonstrated importance	An activity is favored very strongly over another; its dominance demonstrated in practice
8	Very, very strong	
9	Extreme importance	The evidence favoring one activity over another is of the highest possible order of affirmation

Saaty, T.L, (2008).

3.2.4 Fire Vulnerability Map

To make a fire vulnerability map at first attribute maps (five maps) as layer maps has prepared considering five attributes (accessibility, open space in between two apartments, transformer and power line, emergency exit , fire alarm). Those five maps are integrated into a single map to analysis the vulnerability using the Analytical Hierarchy Process (AHP). Weights which chosen depending on the expert opinion for each attribute were used as input in this process. Output of this process defined which attributes get more priority. More priority attribute gained more weight and the score was calculated for each apartment using this weight.

The score calculation equation is given below:

Score for apartment = Weight for accessibility × (1 for No access or 0 for Access) + Weight for open space in between two apartments × (0 for Not vulnerable or 1 for vulnerable for narrow space) + Weight for transformer and power line × (0 for Not vulnerable or 1 for Vulnerable considering power line or transformer) + Weight for emergency exit × (0 for Having emergency exit or 1 for Without emergency exit) + Weight for fire alarm × (0 for Having fire alarm or 1 for Without fire alarm). Islam, et al. (2008).

And after then fire vulnerability map has prepared on the basis of this score. Low score apartments are less vulnerable and higher score apartments are more vulnerable. All the map work was done by using ARC VIEW software and GPS value.

In this research paper, five attributes are road accessibility (A 1), open space in between two apartments (A 2), transformer and former line (A 3), emergency exit (A 4), and fire alarm (A 5). A normalized set of weights is established to be used when comparing alternatives using these attributes. A pair wise comparison matrix M is formed where the number in the ith row and jth column gives the relative importance of Ai as compared with Aj. 1-5 scale is selected from the four expert's opinion. aij = 1 if the two objectives are equal in importance and aij = 2 if Ai is more important than Aj.

Matrix M which is made by expert opinion is given in the following:

$$M = \begin{bmatrix} 1 & 2 & 3 & 2 & 1/2 \\ 1/2 & 1 & ½ & 3 & 1/3 \\ 1/3 & 2 & 1 & 1/3 & 1/2 \\ ½ & 1/3 & 3 & 1 & 1/3 \\ 2 & 3 & 2 & 3 & 1 \end{bmatrix}$$

To normalize the weights, the sum of each column is computed and then each column is divided by the corresponding sum. Thus N is used to denote normalization.

$$N = \begin{bmatrix} 0.231 & 0.240 & 0.316 & 0.214 & 0.188 \\ 0.115 & 0.120 & 0.053 & 0.322 & 0.124 \\ 0.077 & 0.240 & 0.105 & 0.035 & 0.188 \\ 0.115 & 0.040 & 0.316 & 0.107 & 0.124 \\ 0.462 & 0.360 & 0.211 & 0.322 & 0.376 \end{bmatrix}$$

The next step is to compute average values of each row and used these as weights in the attribute Hierarchy. The weights are shown in the table:

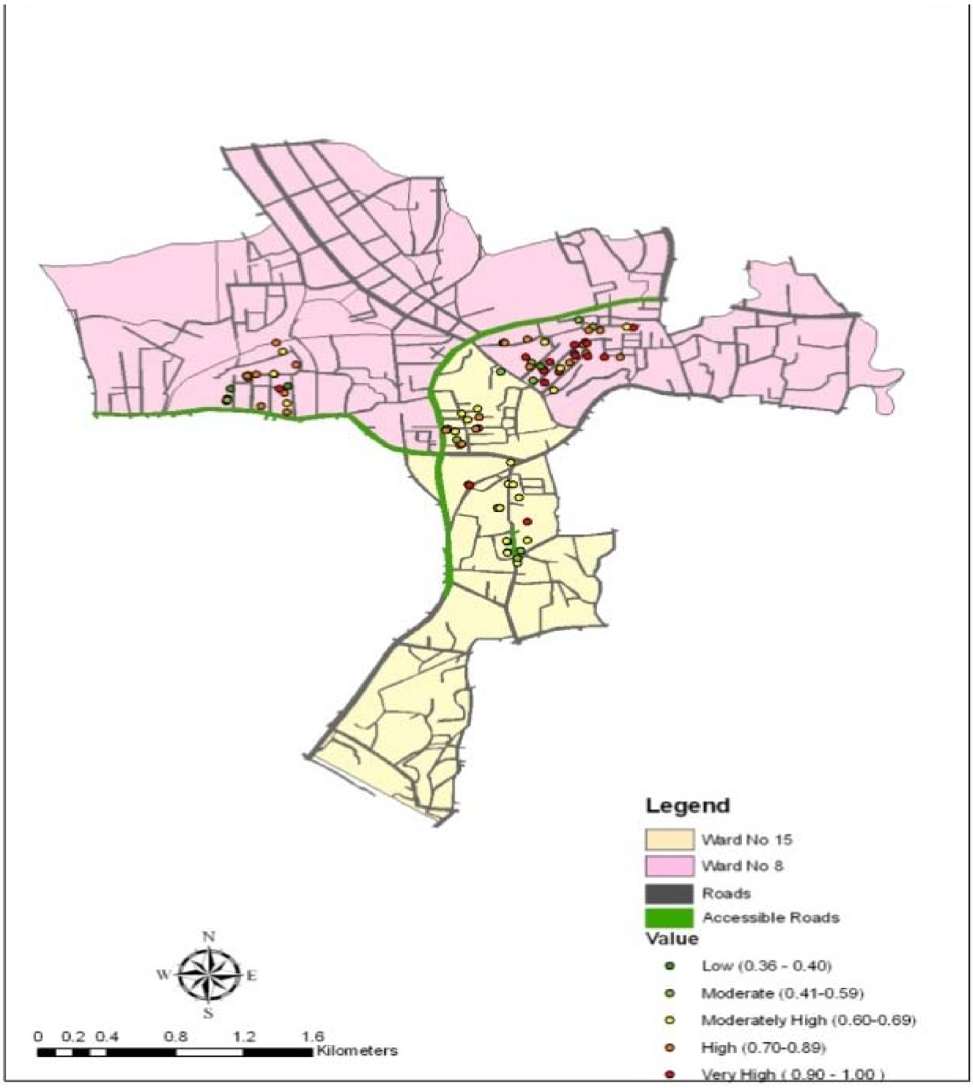
Table 2: Weight of the attributes

Attributes	Weights of Attributes
Accessibility	0.238
Open space in between two apartments	0.147
Transformer and power line	0.125
Emergency exit	0.140
Fire alarm	0.346

Field Survey, July 2010.

Vulnerability score for each building is calculated by the above said equation using the derived weight. On the basis of this vulnerability score all the buildings are classified and it is shown in the apartment vulnerability map. The map is classified into five classes: Low, moderate, moderately high, high and very high.

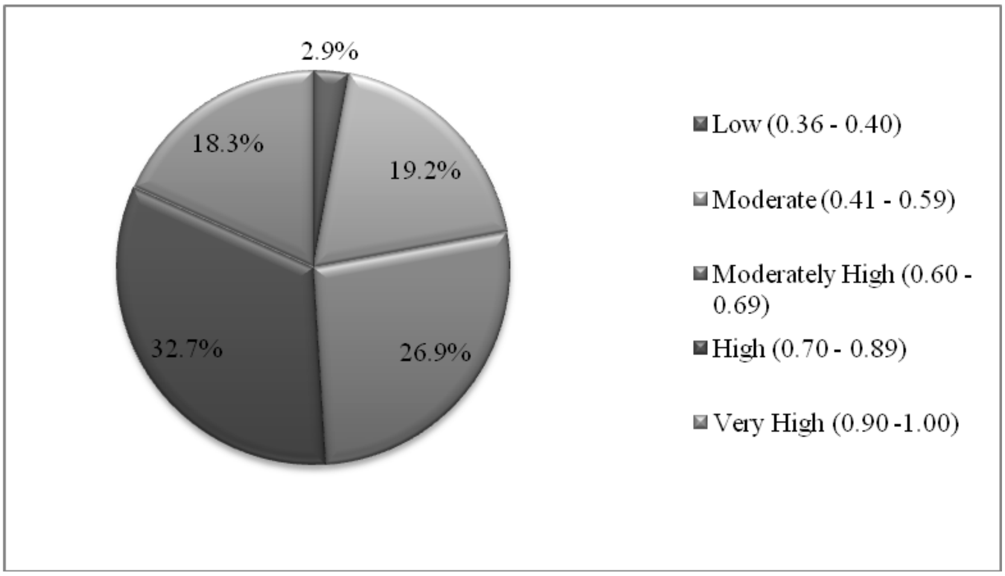
Map 1: Vulnerability map of High-rise Apartments considering Fire Hazard



Base Map Collected from CDA, 2010

Percentage of apartments in different categories is shown in Fig.2. From the map it is found that the apartments are vulnerable in different levels considering fire hazard. The vulnerability score of apartments of the study area range from 0.36 to 1.00 out of 1.

Fig 2: Different types of vulnerable apartments considering fire hazard (in percentage)



Source: Field Survey, July 2010.

In the study area it is found that around 32.7% apartments are highly vulnerable and 26.9% are moderately high vulnerable where is only 2.9% are low vulnerable considering fire hazard.

4. CONCLUSION

From the discussion, it can be said that the high-rise buildings of the study area are vulnerable considering fire hazard. To estimate the vulnerability of the apartments, Analytical Hierarchy Process (AHP) has been applied. This process represents the vulnerability feature of the apartments. If any fire hazards occur in these apartments, it has great chance to take place a large scale of human casualty and economic loss. Finally, considering this situation of fire accident vulnerability of the apartments should strictly follow the fire safety code.

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WATER FOR ENVIRONMENTAL SANITATION IN THE SLUMS OF KHULNA CITY

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ABSTRACT

Water, sanitation and hygiene have the potential to prevent at least 9.1% of the global disease burden and 6.3% of all deaths. If the cost of water for sanitation is high, people become more conservative in water use and the closer the source of water is to the home, the greater the use for sanitation and hygiene. In the slums of Khulna City, normally deep and shallow tubewells in inadequate numbers are installed near the community latrines used by 15 to 30 families. In few cases, tubewells are installed for single and twin pit latrines used by 2 to 3 families. Water of the derelict ponds and ditches, which is contaminated and harmful for health, is used by many households. Some of the tubewells adjacent to the latrines are found out of order for a longer period and the slum dwellers are to use little amount of water carrying from a long distance. This makes the latrines dirty, unhygienic and unused for a long time. The paper aims to highlight availability, demand, consumption and quality of water for sanitation in the slums of Khulna city; sufferings of slum people for using inadequate and contaminated water; and recommendations for increasing water availability for sanitation and hygiene for the slums of Khulna City.

Key Words: *Environmental sanitation, hygiene, safe water, community latrine, pit latrine*

1. INTRODUCTION

Safe water is very important for environmental sanitation, hygiene and better health of any community. But, water demand for environmental sanitation and hygiene with adequate safe water sources is generally not assessed and considered while designing and implementing sanitation and hygiene projects for the urban slums of Bangladesh. Adequate sanitation is the first mean to prevent spreading of excreta related diseases and spreading of pathogens in the residential environment. The second mean is washing hands which prevents pathogen transmission to food and water and further on to other people. Adequate storage of clean water near the toilets is important for improved sanitation. Despite huge investments in sanitation during the United Nations International Drinking Water Supply and Sanitation Decade (1981-90), over 2.4 billion people still lack access to adequate sanitation services and an estimated 3 million children die each year of dehydration related to diarrhoea (WHO, 2000). Sanitation reduces or prevents human faecal pollution of the environment, thereby reducing or eliminating transmission of diseases from that source (although other sources, such as animal excreta, may remain important). Effective sanitation isolates excreta and/or inactivates the pathogens within faeces. There has been increasing recent interest in “total sanitation” i.e. achieving a level of overall sanitation in a community that will significantly reduce disease. The importance of sanitation extends to aspects of privacy, dignity and school attendance (WHO, 2008). If the cost of water for sanitation is high, people become more conservative in water use and the closer the source of water is to the home, the greater the use for sanitation and hygiene. Keeping a source of safe water near the home makes sanitation and hygiene practices much easier and so, it is important to keep water near the toilet. Life on earth is based on water. The quality of life directly depends on water quality. Healthy ecosystems are sustained by good water quality, which leads to improved human well-being. On the contrary, poor water quality affects the environment and human well-being. In any community water demand varies on seasonal, daily and hourly basis. Water use or demand is expressed numerically by average daily consumption per capita (per person). The quantity of water delivered and used for households is an important aspect of domestic water supplies, which influences hygiene and therefore public health. Water consumption generally increases the nearer the water source is to the dwelling. Water availability indicates all people have safe and equitable access to a sufficient quantity of water for drinking, cooking and personal and domestic hygiene. Average water use for drinking, cooking and personal hygiene in any household is at least 20 liters per person per day.

2. OBJECTIVES OF THE STUDY

Objectives of the study are:

- To study the availability, demand, consumption and quality of water for sanitation in the slums of Khulna city
- To find out the sufferings of slum people for using inadequate and contaminated water
- To draw some recommendations for increasing water availability for sanitation and hygiene for the slums of Khulna City.

3. METHODOLOGY

The study was conducted during 2008-2009 in three slums of Khulna City, namely Rupsha slum on Government khas land and Christian community land; Montu Colony slum on Bangladesh Railway land; and Kulibagan slum on Ispahani Company private land. Primary data for the study was collected through structured household questionnaire survey, interview with group and committee members of sanitation and related projects, and concerned GO-NGO staffs. A total of 384 households (22%) were randomly selected and surveyed from the slums, where 215 households were from Rupsha slum, 75 households were from Montu Colony slum and 94 households were from Kulibagan slum. Books, project reports and implementation guidelines, journals, newspapers etc. were reviewed as secondary data sources.

4. KHULNA CITY AND STUDY SLUMS

Khulna is the 3rd largest industrial city of Bangladesh. It is a divisional city and acts as regional hub of administrative, institutional, commercial and academic affairs. It is located on the banks of the Rupsha and the Bhairab rivers. The city covers an area of 45.65 square kilometers with a population of near about 1.5 million. There are 520 slums in Khulna city. Most of the slums of Khulna City have established after 1971, the year of independence of Bangladesh. About 73% slums have been established after 1971. Among 73% slums, 15.2% in 1971-75, 15.8% in 1976-80, 12.9% in 1981-85, 10.2% in 1986-90 and the rest 18% in 1991-2005. In the slums of Khulna city 188,442 poor and landless people live. Density of population per acre and per kilometer in the slum area is 538 and 132,988 respectively (CUS, 2006). Following three slums, located in three diverse locations of Khulna city are the study slums:

Rupsha Slum

Rupsha Slum is situated in Ward 22 of Khulna City Corporation. It is 2.5 kilometers on the north-east side from the Central Business District (CBD). It is on the west side of Rupsha embankment of BWDB which is along the western bank of River Rupsha. Total land area of Char Rupsha Slum is 4.0 acre. The strip of land just along the embankment is owned by the Government and the rest area after along the western side is owned by the Christian Baptist Church. The slum was established nearly in 1960s for settling the poor workers of nearby Dada Match Factory, Rupsha; Khulna Shipyard, Labonchara; and water transports namely boats, launches and steamers. Around 5,500 people live in around 1,000 households. The slum is bounded by BWDB embankment and Rupsha River on the east; Caritas and LGED office premises on the west; Rupsha Natun Bazar on the north; and Rupsha bus stand on the south.

Montu Colony Slum

Joragate Montu Colony Slum is situated in 21 Ward of Khulna City Corporation. The land is owned by the Railway Department of the Government of Bangladesh. It is established immediate after the liberation war of 1971. First time a population of around 75,000 lived here but it reduced at around 35,000 after an eviction during the Mujib regime. The slum also faced an eviction during the Zia regime. As the land is taken lease from the Department of Railway by Mr. Montu, a contractor of the area of that time, the area is named as Montu Colony. There are 350 households in the slum with a population of around 1,925. The 5 no. ghat (landing point at river bank) of Bhairab river was famous and busy for shipment of goods from cargo, launches, trawlers and boats to Khulna city and adjacent industrial area. In past, transport, goods landing and industrial laborers lived in this slum. The Montu Colony slum is bounded by Joragate Rail crossing on the west, Koiladepo slum and Bhairab river on the east, KCC Kitchen market (Kutcha bazar) and the road of 7 no. ghat on the north and Railway vacant land and *Natun Bastee* on the south.

Kulibagan Slum

Kulibagan slum is in Ward 3 of Khulna City Corporation. The Slum is located about one kilometer away from Daulatpur traffic island towards north at the western side of Khulna-Jessore highway. Area of the slum is 2.5 acre. Kulibagan is at the southern side of Moheswarpasa Modhodanga road which is originated from the Khulna-Jessore highway towards west. The slums are bounded by Shashibhusan High School at the south, Sadhubagan slum and Urban Primary Health Care Center of KCC at the north, Khulna-Jessore Highway on the east and Daulatpur Police Station on the west. Around 2,390 people live in around 435 households and workers mess of the slum. The land of the slum is owned by Ispahani Group of Industries.

5. RESULTS AND FINDINGS

5.1 Availability, Demand, Consumption and Quality of Water for Sanitation in the Slums

5.1.1 Availability of Water for Sanitation in the Slums

Tubewells and ponds are the sources of water in the study slums. Deep tubewell water is mainly used for drinking, whereas slum dwellers use shallow tubewell and pond water for bathing, washing utensils and clothes, and in latrines in general. In the study slums, water sources i.e. tubewells, ponds and ditches were found very close to the households.

Table 1: Distance of slum households from water sources

Name of slums	**Percentage of households with distance (Meter)**					**Total**
	Less than 25	**25-50**	**51-75**	**76-100**	**More than 100**	
Rupsha slum	191	18	0	6	0	215
	88.9	8.3	0.0	2.8	0.0	100.0
Montu Colony slum	30	15	4	11	15	75
	40.0	20.0	5.7	14.3	20.0	100.0
Kulibagan slum	65	22	0	6	0	94
	69.2	23.8	0.0	6.9	0.0	100.0
Total	286	55	4	23	15	384
	70.9	18.6	0.8	6.8	3.0	100.0

Source: Household survey, 2008-2009

In the study slums, water sources were found very close to the households. Table 1 reveals that 70.9% households had their water sources in less than 25 meter distance. Households of 88.9%, 69.2% and 40% of Rupsha, Kulibagan and Montu Colony slums mentioned their water sources in less than 25 meter distance respectively. Numbers of tubewells in Montu Colony slum were less than Rupsha and Kulibagan slums and as such water sources of the households of Montu Colony slum were not so close. About 34.3% households of Montu Colony slum reported their water sources in more than 75 meters.

5.1.2 Demand and Consumption of Water for Sanitation in the Slums

Basic survival water needs for personal hygiene practices depend on social and cultural norms and in general it is 6-7 liter per capita per day. In the slums, a person is found to use about 4-5 liter water in latrine for sanitation. It is also lower compared to the basic survival water need for sanitation. Generally one *Badna* contains 2 (two) liters water.

Table 2: Number of water *Badna* (Pot with spout) used at every use of latrine

Name of slums	**Number water *Badnas***			**Total**
	1	**2**	**More than 2**	
Rupsha slum	0	0	215	215
	0.0	0.0	100.0	100.0
Montu Colony slum	0	0	75	75
	0.0	0.0	100.0	100.0
Kulibagan slum	12	20	62	94
	12.3	21.5	66.2	100.0
Total	12	20	352	384
	6.8	11.8	81.4	100.0

Source: Household survey, 2008-2009.

Table 2 reveals that most of the household members i.e. 81.4% households of the study slums used about 2 liters water (1 Ba*dna* water) at every use of latrine for cleaning the excreta and latrine. All the households of Rupsha and Montu Colony slums reported to use about 4 liters water (2 *Badna* water) at every use of latrine as they use pond, ditch and shallow tubewell water in latrines. On the other hand about 33% households of Kulibagan slum reported to use about 2 liters water (1 *Badna* water) at every use of their latrine.

5.1.3 Quality of Water for Sanitation in the Slums

Pond and ditch water is available in Rupsha and Montu Colony slums and as such household members of the two slums use much more water than Kulibagan slum at every use of latrines. But, water of these ponds and ditches are polluted and unhygienic due to presence of hanging latrines on these. Health and environmental condition of the slums is good where water, sanitation and drainage facilities are in good condition and slum dwellers are habituated in hygiene sanitation practice.

Table 3: Access of households to drainage facility

Name of slum	**Access to drainage facility**		**Total**
	Have access	**No access**	
Rupsha slum	104	111	215
	48.6	51.4	100.0
Montu Colony slum	9	66	75
	11.4	88.6	100.0
Kulibagan slum	93	1	94
	99.2	0.8	100.0
Total	206	178	384
	53.6	46.4	100.0

Source: Household survey, 2008-2009.

Table 3 reveals that about 53.6% households of the study area had access to drains. Drainage condition of Kulibagan slum is quite good (99.2% households have access to drains). Drainage condition of Montu Colony slum is very poor (88.6% households have no access to drains and only 11.4% households have access to drains). Drainage condition of Rupsha slum is moderate (around 48.6% households have access to drains).

Table 4: Water logging problem in the slum area

Name of slum	**Water logging problem**		**Total**
	Facing problems	**Facing no problem**	
Rupsha slum	111	104	215
	51.4	48.6	100.0
Montu Colony slum	39	36	75
	51.4	48.6	100.0
Kulibagan slum	5	89	94
	5.4	94.6	100.0
Total	155	229	384
	40.4	59.6	100.0

Source: Household survey, 2008-2009.

Figure 1: Polluted pond water by hanging latrine (2009) of Rupsha slum is used in latrines.

Figure 2: Tubewell close to latrine (2007) in Rupsha slum remained out of order long time.

Figure 3: Community latrine (2007) in Kulibagan slum was unhygienic due to inadequate water.

Table 4 reveals that about 51.4% households of both Rupsha and Montu Colony slum faced water logging problem. 94.6% households of Kulibagan slum mentioned not to face water logging problem. As rain water is not easily drained out and remains stagnant often in Montu Colony slum and in Rupsha slum, the slum area becomes unhygienic with dispersal of excreta over the entire slum area from the submergence of open and ground level latrines.

5.2 Sufferings of Slum People for Using Inadequate and Contaminated Water for Sanitation

Diarrohea and cholera, dysentery, skin disease, cold and coughing are the sanitation and water related diseases outbreak in the study slums.

Table 5: Diseases in slums related with water and sanitation

Name of slums	Percentage of households with disease						Total
	Diarrhoea	**Cholera**	**Dysentery**	**Skin diseases**	**Cold**	**Coughing**	
Rupsha slum	92	20	51	20	31	0	215
	42.9	9.5	23.8	9.5	14.3	0.0	100.0
Montu Colony slum	8	26	11	11	8	11	75
	10.0	35.0	15.0	15.0	10.0	15.0	100.0
Kulibagan slum	15	2	38	9	26	4	94
	15.9	2.3	40.9	9.1	27.3	4.5	100.0
Total	115	49	101	40	64	15	384
	29.9	12.7	26.3	10.5	16.6	4.0	100.0

Source: Household survey, 2008.

Table 5 reveals that 42.6%, 26.3%, 16.6% and 10.5% households of the study slums suffered from diarrohea and cholera, dysentery, skin disease and cold respectively.

Unhygienic latrine, unhygienic surroundings, unhygienic food and weather change are the causes of diseases in the slums.

Table 6: Causes of diseases outbreak in the slums

Name of slums	Causes of disease					Total
	Weather change	**Unhygienic latrine**	**Unhygienic surroundings**	**Unhygienic food**	**Don't know**	
Rupsha slum	61	41	51	20	31	215
	28.6	19.0	23.8	9.5	14.3	100.0
Montu Colony slum	30	34	11	0	0	75
	40.0	45.0	15.0	0.0	0.0	100.0
Kulibagan slum	45	28	21	0	0	94
	47.7	29.5	22.7	0.0	0.0	100.0
Total	136	102	84	20	31	384
	41.2	30.6	21.2	2.4	3.5	100.0

Source: Household survey, 2008-2009.

Table 6 reveals that most of the households i.e. 51.8% households have reported unhygienic latrines and unhygienic surroundings are the major causes of out breaking diseases in the slums.

Absence in work, absence in school, physical weakness, financial loss, sufferings of family members and relatives etc. are the major consequences have to face by slum households for the water and sanitation related diseases.

Table 7: Consequences of water and sanitation related diseases

Name of slums	Consequences of diseases				Total
	Absence in work	Absence in school	Physical weakness	Financial loss	
Rupsha slum	41	20	133	20	215
	19.0	9.5	61.9	9.5	100.0
Montu Colony slum	35	13	0	26	75
	47.1	17.6	0.0	35.3	100.0
Kulibagan slum	43	17	19	15	94
	45.5	18.2	20.5	15.9	100.0
Total	119	51	152	62	384
	39.0	15.9	26.8	18.3	100.0

Source: Household survey, 2008.

Table 7 reveals that most of the households i.e. 39% households reported absence in work as the major consequence of water and sanitation related diseases. Households of 26.8%, 18.3% and 15.9% respectively mentioned physical weakness, financial loss and absence in school are the consequences of water and sanitation related diseases.

Duration of sufferings at a time of most of the family members of the study slums due to Water and Sanitation related diseases is 4-7 days. Table 8 reveals that households of 70.5% reported the duration of sufferings 4-7 days next to 20% households with more than 7 days.

Table 8: Duration of sufferings from water and sanitation related diseases

Name of slums	Duration of sufferings (in days)			Total
	1-3	4-7	>7	
Rupsha slum	41	133	41	215
	19.1	61.9	19.0	100.0
Montu Colony slum	8	49	19	75
	10.0	65.0	25.0	100.0
Kulibagan slum	4	72	17	94
	4.6	77.0	18.2	100.0
Total	53	254	77	384
	9.5	70.5	20.0	100.0

Source: Household survey, 2008-2009.

Most of the slum dwellers of the study slums receive allopathic treatment during their ailment. Table 9 reveals that about 84.8% and 15.2% households received allopathic and homeopathic treatments respectively. Slum people generally receive allopathic treatment from the nearby local medical practitioners who run small dispensaries and medicine shops. Slum dwellers of Rupsha slum and Montu Colony slums receive treatments from Khulna Sadar (General) hospital and Khulna 250 bed hospital respectively.

Table 9: Treatment types received by households for WatSan related diseases

Name of slums	Treatment types		Total
	Allopathic	Homeopathic	
Rupsha slum	185	30	215
	86.2	13.8	100.0
Montu Colony slum	54	21	75
	72.0	28.0	100.0
Kulibagan slum	86	8	94
	92.0	8.0	100.0
Total	326	58	384
	84.8	15.2	100.0

Source: Household survey, 2008.

Slum families have to spend a significant amount for the treatment of water and sanitation related diseases.

Table 10: Treatment cost pattern for WatSan related diseases in six months

Name of slums	Percentage of households with treatment cost (In Tk.)				Total
	100<	101-200	201-400	>400	
Rupsha slum	51	61	21	82	215
	23.9	28.5	9.6	38.0	100.0
Montu Colony slum	4	25	13	33	75
	5.5	33.3	16.7	44.5	100.0
Kulibagan slum	13	68	4	9	94
	13.6	72.8	4.5	9.1	100.0
Total	68	155	37	124	384
	14.4	53.2	8.4	24.0	100.0

Source: Household survey, 2008-2009.

Table 10 shows that maximum i.e. 53% households of the study slums spent Tk. 100-Tk. 200 for treatment of water and sanitation related diseases in six months in 2008 next to 24% households more than Tk.400.

6. RECOMMENDATIONS FOR INCREASING WATER AVAILABILITY FOR HYGIENIC SANITATION IN THE SLUMS

Following recommendations are made for increasing water availability for hygienic sanitation in the slums of Khulna City:

- Concerned organizations i.e. KCC, DPHE, KWASA, NGOs and CBOs should assess the demand of hygienic water for sanitation for the slum people and should take necessary steps to create more water sources for sanitation.
- Existing ponds and ditches as surface water sources can be protected and preserved ensuring environmental quality.
- Steps can be taken by the concerned organizations to make all the slum people more aware about the importance of using adequate hygienic water for sanitation.
- Community people can be made aware also on causes, consequences and remedial measures of deterioration of surface water quality with respect to use in sanitation.
- More researches can be conducted on the parameters and factors of surface and ground water quality for sanitation.

7. CONCLUSION

Availability of adequate hygienic water is very important for flushing, washing and keeping the latrines neat and clean after their every use. Keeping a source of safe water near the home makes sanitation and hygiene practices much easier and so, it is important to keep water near the toilet. Water consumption generally increases the nearer the water source is to the dwelling. Some of the tubewells adjacent to the latrines in study slums were found out of order for a longer period and the slum dwellers had to use little amount of water carrying from a long distance. This makes the latrines dirty, unhygienic and unused for a long time.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STUDY ON BIOGAS GENERATION FROM WEEDS

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ABSTRACT

Bangladesh is a densely populated country where overall energy consumption is still very low. But in near future the energy consumption will be huge and the energy crisis will become worst because the natural source of energy is limited and is decreasing for the continual use. So now a day we must rely on the alternative source of the production of the biogas. The main aim of this study is to identifying the simple way from which gas can be produced with a very low expanse. Pterislongifolia (DhekiShak) and Water Hyacinths are very common weeds in our country. They grow profoundly in our country and regarded as useless weeds. So if these weeds can be used it would be a great source of Biogas. Mainly two processes have been used for the gas production (Batch Process and Continuous process). In batch Process we can get 2018cm^3 of biogas from 8 days from the mixture of these weeds while 1375cm^3 of biogas with Cow Dung. In Continuous process we can get 5000cm^3 of biogas from the mixture stated above which should be fed at 7-8 days intervals. The study also shows that total gas yield from mixture is 2 times greater than individually DhekiShak and 6 times greater than Water Hyacinths. If this method is practiced practically in larger set up it would be a very useful source of alternative biogas generation in our country which will reduce the energy crisis.

Keywords: *Energy consumption, Biogas, Weeds, Lower cost, Batch process, Continuous Process.*

1. INTRODUCTION

By all measures, energy is a key determinant of socio economic development of any country. Bangladesh, a small country of 55,000 square miles having more than 143.8 million people (fao yearbook livestock bangladesh, 2005) would require gigantic amount of energy for its development. But the matter of fact is that in bangladesh the scarcity of energy is already acute and bears little sign of improvement. The challenges include a very limited choice of energy sources, shortage of funds for research purposes and overall development in energy sector, technical expertise etc. these challenges are more enhanced by the pressures put on decision makers by various lobbies to go the decision in their favorable positions, an alleged lack of transparency in the decision making process and a shortage of technical experts (Jamil, 2007). Only 4% people are connected in a gas pipelined network and in villages there is almost no gas network at all (Chowdhury, 2003). Access to electricity in Bangladesh is one of the lowest in the world. In rural areas, where 80% of the total population lives, the rate is really diminutive. Only about 10% households in the rural areas have electricity access. More than 60% of total energy consumption of the country is being met from biomass (Islam, 2000). More than 39 million tons of the traditional fuel such as, agricultural residue, tree twigs, jute stick, leaves and cow dung are used as fuel in Bangladesh each year (BBS, 1991). Agricultural residue and animal dung comprised 84.68% of the total fuel used in 1995-96 (BBS, 1998). Wood accounted for 16.2% and commercial energy sources contributed 26.9% (BBS, 1991) of the household fuel demand, as revealed by 1991 Population Census. Biomass is cheap but extensive and unplanned consumption pattern of biomass fuels led the consumption beyond regenerative limits with serious environmental consequences (Islam, 1990). Continuous use of leaves, twigs, agricultural and animal residues as fuel deprive soil from valuable nutrients and causes soil erosion, especially in the coastal region of Bangladesh (Islam, 1993). Apart from these, many adverse health problems are also occurring mainly among women. It is apprehended that, with population growth, the energy crisis, environmental degradation, deforestation, declining of soil fertility, etc. will sharpen further if the things move as usual and no alternative measures are undertaken. The current situation in the energy sector leads the policy makers to think about the alternative sources of energy, which will be renewable in nature and can be adopted throughout the country Biogas offers a sustainable solution, at least in part, to all these problems Bangladesh is currently facing. There has been a renewed interest in biogas owing to rising concerns over the greenhouse effect, high price of fossil fuels, and other environmental and health concerns. This work is a comparative study on the production of biogas from non conventional weeds (Dheki Shak and Water Hyacinths) along with selected ingredients by batch and continuous process. The main objective of this research is to identify the simple way from which gas can be produced with a very low expanse. This publication will be useful for policy- and decision-makers in governments, the donor community, and the private sector dealing with biogas programmes.

Biogas is basically a mixture of methane and carbon dioxide. It is produced by the anaerobic decomposition of organic matter, both animal and vegetable.

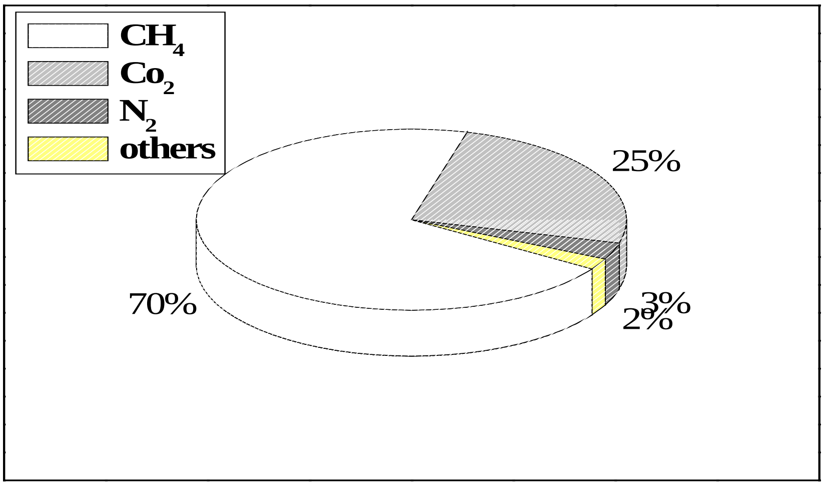

Figure 1: Composition of all the materials of Biogas (in %)

The whole biogas-process can be devided into three steps: hydrolysis, acidification, and methane formation (Figure 2). Three types of bacteria are involved (Amon & Boxberger, 2001).

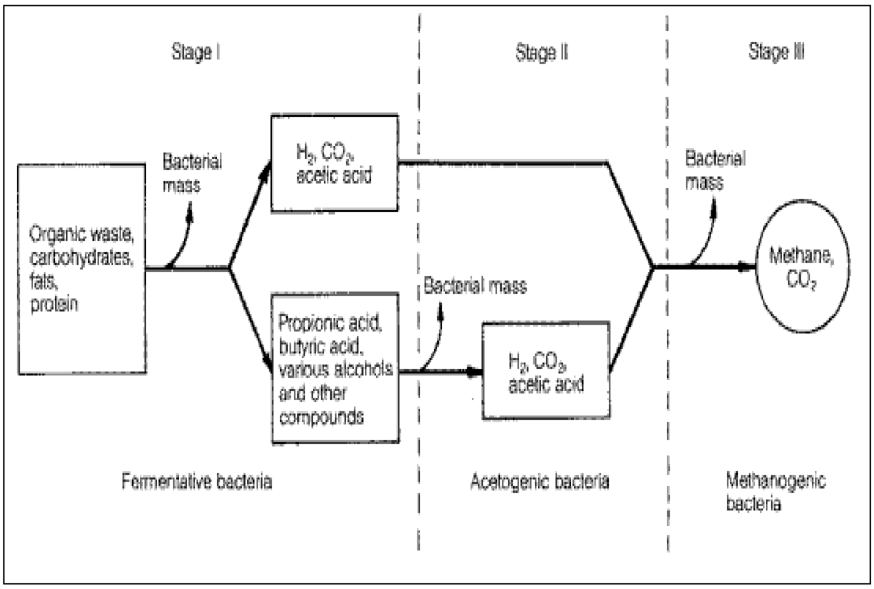

Figure 2: The three-stage anaerobic fermentation of biomas

Anaerobic digestion is a three stage process.The first stage consists of micro organisms attacking the organic matter where complex organic compounds such as cellulose and starch are converted to less complex soluble organic compounds. Polymers are transformed into soluble monomers through enzymatic hydrolysis.

$$n(C_6H_{10}O_5) + nH_2O \xrightarrow{\text{hydrolysis}} n(C_6H_{12}O_6) \quad (1)$$

These monomers become substrates for the microorganisms in the second stage where they are converted into organic acids by a group of bacteria.

$$n(C_6H_{12}O_6) \xrightarrow{\text{acid forming bacteria}} 3n\ CH_3COOH \quad (2)$$

These organic acids primarily acetic acid form the substrate for the third-stage.

$$CH_3COOH \xrightarrow{\text{methane forming bacteria}} CH_4+CO_2 \quad (3)$$

In the third step, methanogenic bacteria generate methane by two routes, by fermenting acetic acid to methane (CH_4) and CO_2 and by reducing CO_2 via hydrogen gas or formate generated by other bacterial species.

$$CO_2+4H_2 \xrightarrow{\text{reduction}} CH_4+2H_2O \quad (4)$$

Similarly CO_2 can be hydrolysed to carbonic acid and to methane as in equations (5) and (6)

$$CO_2+H_2O \xrightarrow{\text{Hydrolysis}} H_2CO_3 \quad (5)$$
$$4H_2+H_2CO_3 \xrightarrow{\text{reduction}} CH_4+3H_2O \quad (6)$$

The carbon dioxide and hydrogen sulphide in the biogas are undesirable. They are removed for optimum performance of biogas as fuel. Carbon dioxide is removed by passing the gas into lime water which turns milky due to formation of calcium carbonate.

$$Ca(OH)_2(aq)+CO_2(g) \longrightarrow CaCO_3+H_2O \quad (7)$$

H_2S is removed by passing the gas through a lead acetate solution.

$$(CH3COO)_2Pb(aq)+H_2S(g) \longrightarrow 2CH_3COOH(aq)+PbS(s) \quad (8)$$

In addition to temperature other parameters are controlled to ensure proper operation. Microorganisms are sensitive to pH changes. Buffering is necessary for pH. The rate of biogas production depends: the nature of the substrate, temperature, pH, loading rate, toxicity,nutrients and, slurry concentration. (Guruswamy & Kumar, 2003)

2. MATERIALS AND METHODOLOGY

Mainly four items has been used in this study of Biogas generation. The items are as follows:

1) Pteris longifolia
2) Water Hyacinths
3) Cow dung
4) Bran

A gas production set up is first made for the gas production. Ingredients are first cut into a small pieces say ¼”. Then they are kept in the reactor and the reactor is kept water tight. A small hole is made in the cork of the reactor and a small pipe is inserted into it and another point of the pipe is kept to a bottle in which another plastic jar full of water is kept invertly to the bottle. The generated gas flows through the pipe and the water level gradually decrease for the gas. The decreasing amount of water indicates the amount of generated gas in volume.

In this study mainly continuous process is used. In a continuous process, initially a certain amount of ingredients are provided. The whole reactor is then water tightened. The ingredients then started to digested anaerobically. The initial rate of digestion is very high. Within a few days the digestion rate become slower so as the gas generation. When this phenomenon is occurred another layer of ingredients is provided to the reactor. And it started to digest again. Thus the process is run continuously by the continuous feeding of the ingredients. The main advantage of the process is that it provides a continuous gas generation. So this process is termed as continuous process for gas generation.

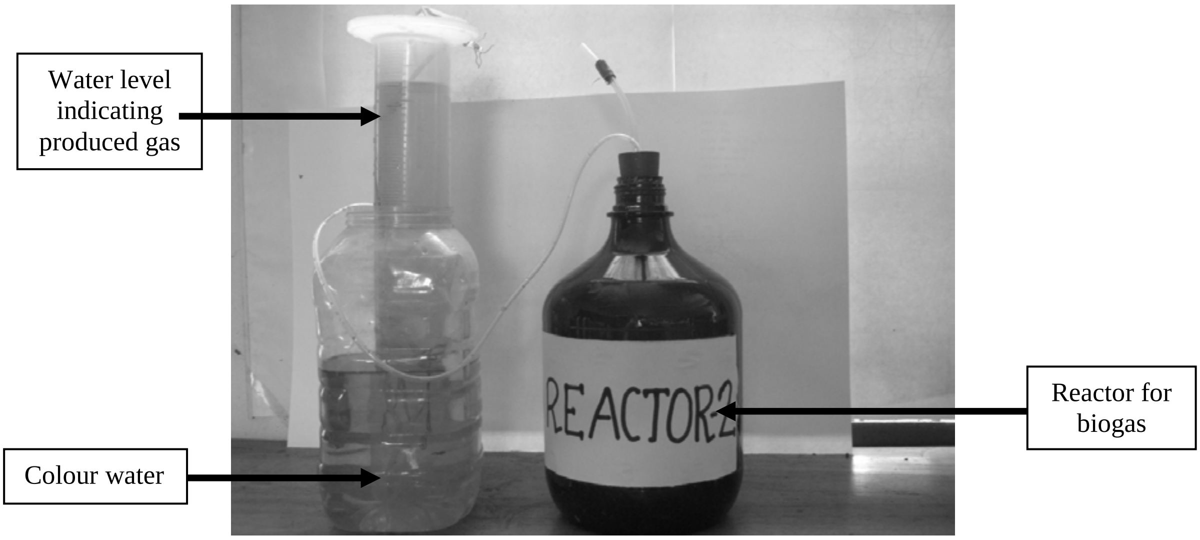

Figure 3: Set up for gas production

2.1 Preparation Of The Ingredients

In this study different phases has been used with different forms of ingredients. The equipments selected previously, collected locally around from the KUET campus. Four forms of experiments have conducted in the study.

1st Run The experiment was conducted with the combination of the all the ingredients. It included Water Hyacinths, Dheki Shak, and Cow dung. A sufficient amount of Bran was also used for the digestion of the elements. The method of the digestion was **Batch Process.** The total weight of the sample was **1 kg (1000g).**

Table 1: Proportion of the ingredients for the first set up.

Name	Dheki Shak	Water Hyacinths	Cow Dung	Bran
Weight (gm)	400	300	250	50
Ratio	4	3	2.5	0.5

2nd Run The experiment was conducted by using only cow dung. This was conducted for determining the amount of gas production for comparing the result among the cow dung and the combination of all ingredients. This was also the **Batch Process.** The total amount of cow dung used was **1 kg (1000g**)

3rd Run In this set up only Water Hyacinths was used with a smaller proportion of Cow dung and Bran. The main difference was that the process was the continuous one. Experimentally two feeding were provided in the study. Same amount of ingredients were used in two set up simultaneously for monitoring the variations.
Initial total weight: 400g.

Table 2: Proportion of the ingredients for third set up

Name	Water Hyacinths	Cow dung	Bran
Weight	250	100	50
Ratio	5	2	1

The next feeding was given after seven days. The weight of the next feedings was equal to the initial weight.
4th Run This is the set up which is completely identical with the third one. The difference was only that Pteris longifolia was used instead of Water Hyacinths. The procedure was same as the third one.

5th Run Combination of all the ingredients was used for the gas production by the **continuous process.**

Table 3: Proportion of ingredients for fifth run

Name	Dheki Shak	Water Hyacinths	Cow Dung	Bran
Ratio	5	3	2	1

Initial weight: - 500g
Weight of each feeding was 500g. Two feeding were provided experimentally.

Figure-4 describes the sequential order of the gas production process in the study.

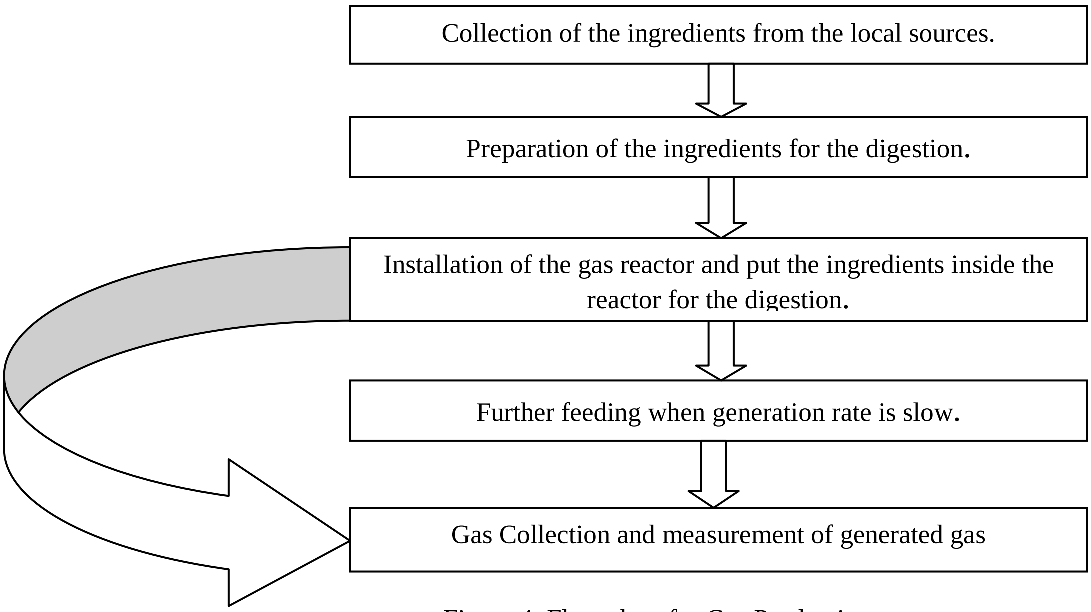

Figure 4: Flow chart for Gas Production

2.2 Gas collection and measurement process

Figure 1.3 shows the gas production setup . Gas production, collection and measurement process are described in details below

- Two glass containers of 2.75 L were selected for the reactor plate.(Figure-3)
- The selected items were collected locally. The weeds were cut into smaller pieces (approximate size 0.25 in). Cow Dung and Bran were mixed homogeneously with the weeds in proportion stated in the previous paragraph.
- The mixture was then fed into the reactor (glass cylinder) through the inlet.
- The inlet of the reactor was made airtight by using rubber cork for anaerobic digestion.
- A small hole was cut through the rubber cork. The diameter of the hole was 6.25mm. A small dia (6.25mm) pipe was inserted through the hole cut in the rubber cork.(Figure-3)
- A plastic bottle and a cylindrical standard plastic jar (1000ml) was used for the gas measurement set up.
- Two of the individual set up was used for the two reactors.
- The plastic bottle was first filled with colored water. The color was prepared by mixing Potassium per Menganate ($KMnO_4$) with the clear water. The cylindrical plastic jar was also filled with the colored water to the top of this jar.
- The cylindrical jar was set invertly with full water to the plastic bottle.
- The reason of the colored water use was that it will help to mark the water level from the transparent plastic jar.(Figure-3)
- The point of the plastic pipe (6.25mm) was set to the upper point of the inverted cylindrical plastic jar.
- When the gas is generated it flows through the pipe from the reactor to the cylindrical jar.
- The accumulated gas in the cylindrical jar forces the water level to lower from the initial height.
- The total lowering (ml) from the initial height of the jar represent the amount of generated gas.
- This is the procedure of gas measurement.

3. RESULTS AND DISCUSSION:

3.1 Data collection

It is the experimental value for the gas production from the mixture of all the ingredients. The ratio of the each of the component has been described at the previous tables.

Table 4: Gas production from the mixture of Dheki Shak, Water Hyacinths, Cow Dung, Bran(Batch Process)

Day	Gas generation (ml)	Cumulative gas generation. (ml)
0	-	-
1	232	232
2	326	558
3	425	983
4	478	799
5	391	1461
6	287	1748
7	192	1940
8	78	2018

Total produced gas in eight days= 2018ml

Table 5: Gas generation from Cow Dung (Batch Process)

Day	Generated gas (ml)	Cumulative generated gas (ml)
0	-	-
1	174	174
2	294	468
3	372	840
4	211	1051
5	135	1186
6	97	1283
7	69	1352
8	23	1375

Total Generated gas in nine days =1375ml

Table 6: Gas generation from Water Hyacinths by Continuous process

Time (Day)	Reactor-1		Reactor-2		Variation in Gas Production (%)	Average Variation %
	Generated Gas (ml)	Cumulative Gas (ml)	Generated Gas (ml)	Cumulative Gas (ml)		
1	-	-	-	-	-	18.5
2	25	57	21	21	16	
3	32	89	37	58	15.625	
4	78	167	61	119	21.8	
7	122	289	101	220	17.22	
8	61	350	81	301	24	
9	48	398	66	367	27	
10	35	433	46	413	24	
11	95	528	65	478	31.5	
14	108	636	97	575	10	
15	121	757	135	710	10	
16	85	842	89	799	4.5	
17	68	910	57	856	16.12	
18	36	946	28	884	22.22	

Table 7: Gas generation from Pteris longifolia (Dheki Shak) by continuous Process

Time (Days)	Reactor-1		Reactor-2		Variation in Gas Production (%)	Average Variation (%)
	Gas Generated (ml)	Cumulative Gas (ml)	Gas Generated (ml)	Cumulative Gas (ml)		
1	-	-	-	-	-	23%
4	77	77	68	68	11.70	
5	120	197	98	166	18.3	
6	135	332	126	292	6	
7	155	487	121	413	22	
8	245	732	263	676	6	
11	185	917	214	890	13.5	
12	127	1044	135	1025	5	
13	92	1136	140	1165	34	
14	140	1276	254	1419	40	
15	155	1431	284	1703	42	
18	209	1640	364	2067	42	
19	327	1967	239	2306	27	
20	169	2136	142	2448	15	
21	89	2225	47	2495	47	
22	29	2254	12	2507	55	

Table 8: Gas production data from the combination of Pteris longifolia, Water Hyacinths, Cow Dung and Bran

Time (Day)	Reactor-1		Reactor-2		Variation in Gas Production (%)	Average Variation (%)
	Generated Gas (ml)	Cumulative Gas (ml)	Generated Gas (ml)	Cumulative Gas (ml)		
1	-	-	-	-	-	20
2	128	128	92	92	28	
3	328	456	231	323	28	
6	632	1088	439	762	30	
7	652	1740	527	1289	19	
8	448	2188	463	1752	4	
9	162	2350	281	2033	22	
10(new feeding)	311	2661	296	2329	5	
13	453	3114	361	2690	20	
14	501	3615	432	3122	13	
15	568	4183	621	3743	9	
16	611	4794	555	4298	9	
17	468	5262	398	4696	15	
20	311	5573	294	4990	6	
21	265	5838	164	5154	38	

3.2 Data Analysis

3.2.1 Gas production from mixture (batch process)

Figure-5 represents the data of the gas production from the combination of Pteris longifolia, Water Hyacinths, Cow Dung and Bran. Total weight of the sample was 1 kg (1000g). The graph explains that the highest gas is produced at the fourth day and the amount is 478ml. After the fourth day the amount of gas production reduced gradually. The total amount of the gas is 2018ml for eight day.

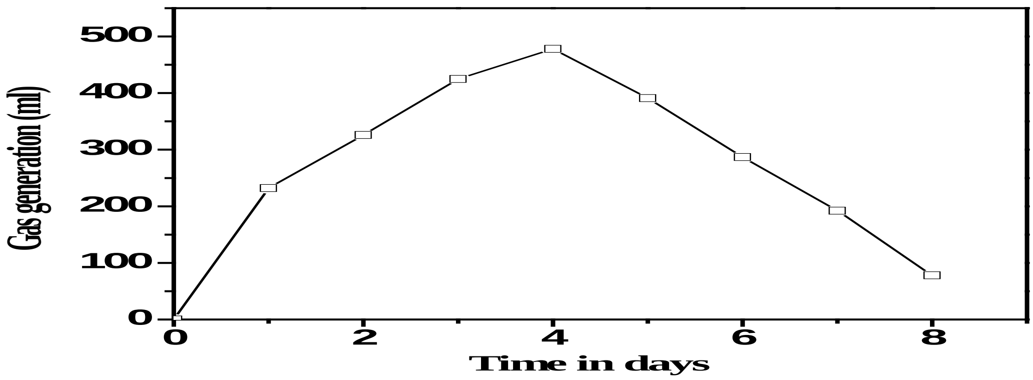

Figure 5: Gas production from combination of the selected item

Figure-6 describes the total cumulative gas production from eight days. In day 4 there is a drop in gas production because feedings for gas production is deacreased. After the 4th day gas production increases again.

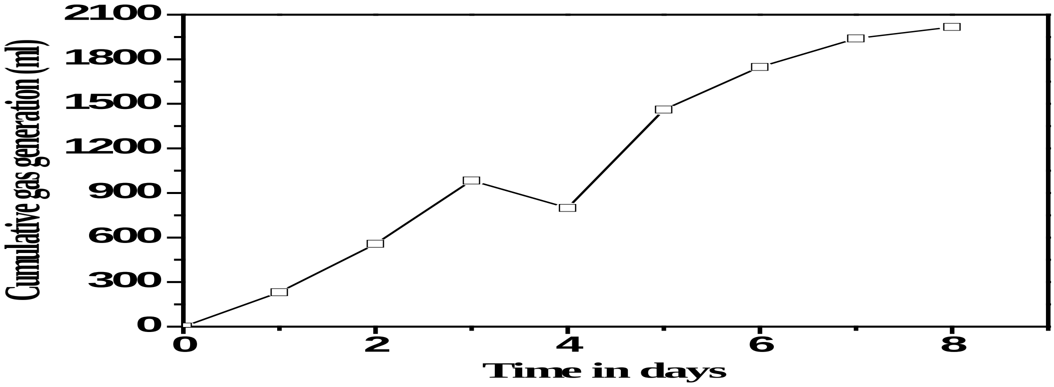

Figure 6: Cumulative Gas Productions

3.2.2 Gas production from cow dung(batch process)

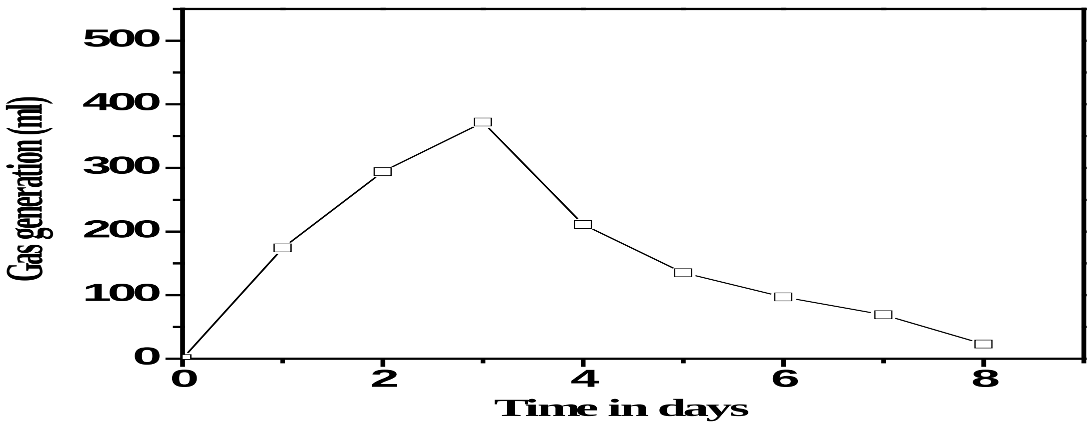

Figure 7: Gas produced from Cow Dung

Figure-7 represents the gas production from the Cow Dung. The weight of the fresh Cow Dung was 1 kg. The graph shows that the gas production in initial days is high. The maximum gas production is in the fourth days which are 372 ml. After that the production of gas decreases gradually.

Figure-8 shows that the total cumulative produced gas was 1375 ml for the nine days of being fed. It shows a gradual increase in day to day basis. From day 1 to day 3 the production rate is higher, then it increases slowly.

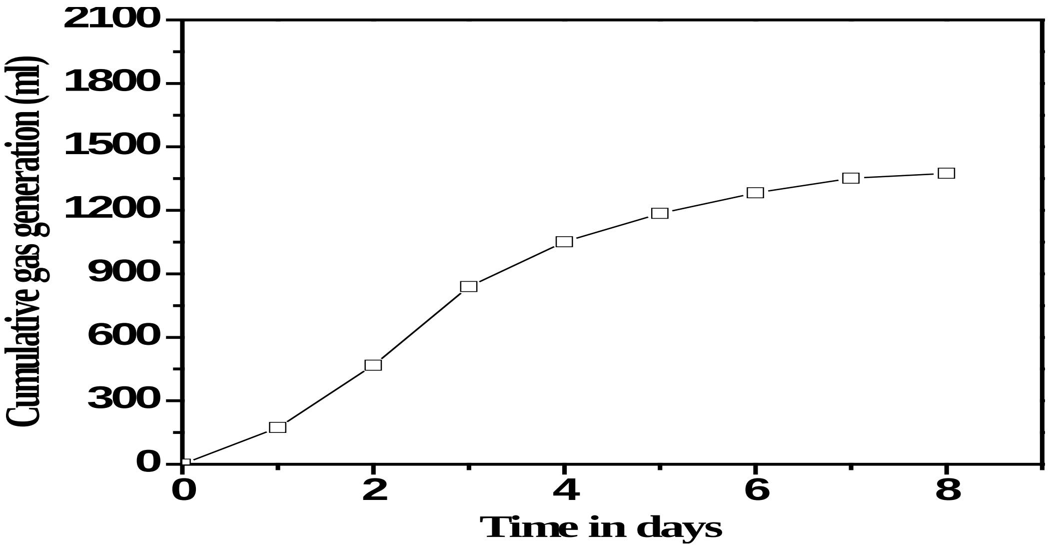

Figure 8: Cumulative gas productions from Cow dung.

3.2.3 Gas Production from Water Hyacinths (Continuous Process)

The following figure-9 represents the gas production from Water Hyacinths from two identical reactors. In the figure (R-1means Reactor 1; R-2 means Reactor 2; and Gas P. means Gas Production)

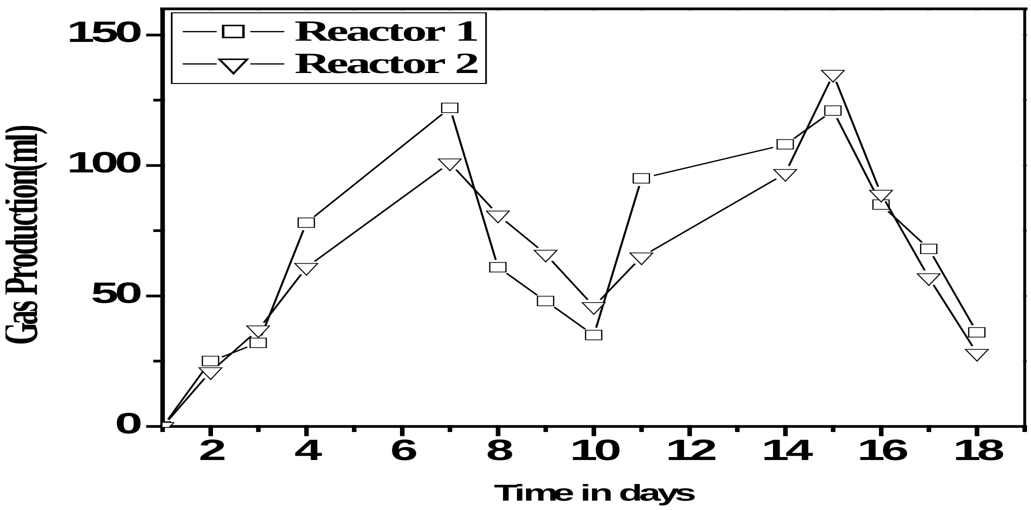

Figure 9: Gas Production from Water Hyacinths.

Figure-9 explains that in the initial days gas production rate is low. Gas production is peak, 122ml at 7th days in the Reactor 1 and 101ml for Reactor 2.After that the generation rate slows down. Then new feeding was provided in both the reactor. For the new feeding anaerobic digestion rate increases slowly so as the gas production. For the 2nd feeding the peak gas generation is 121ml for Reactor1 and 135ml for Reactor 2.
The graph shows the average daily variation in two reactors is 18.5%.

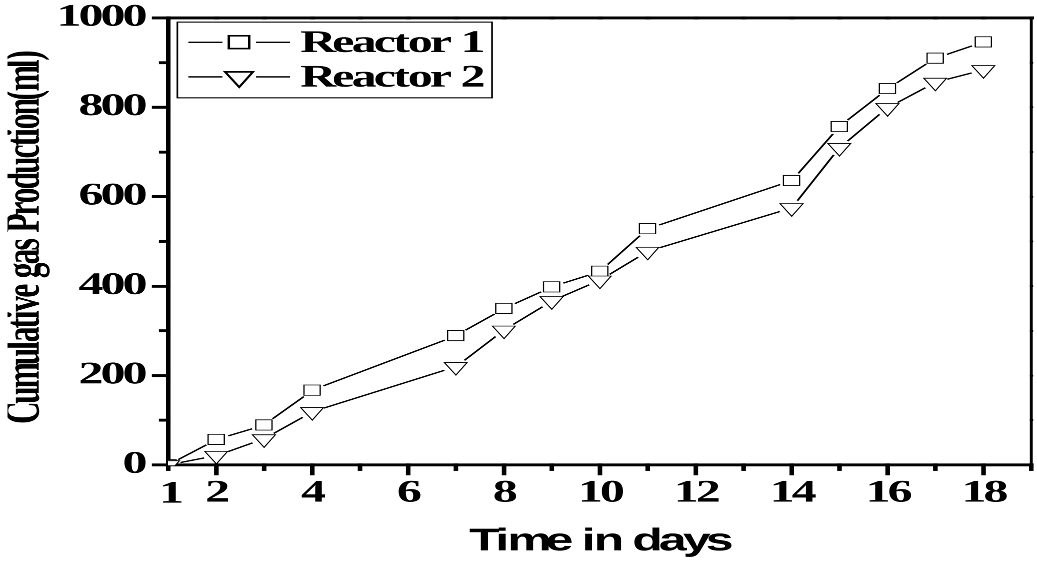

Figure 10: Cumulative Gas production by Water Hyacinths from two reactors

Figure-10 shows the cumulative gas production in two reactors from water hyacinths. The graph describes the cumulative gas production from the two reactors. It shows that the total gas production from Reactor 1 is 946ml and from Reactor 2 is 884ml. The variation in cumulative gas production from two reactor is 6.5%. The figure shows that gas production in the reactors are almst parallel.

3.2.4 Gas Production from Pteris longifolia (Dheki Shak)

The Figure-11 below shows the comparative gas production from Dheki Shak from two reactors. Graph shows that the highest gas produced in a single day in the Reactor-1 and Reactor-2 was relatively 327ml and 237ml. Reactor-1 shows a highest intermidiate drop in day-12 while for reactor-2 it is day-13. Day-18 shows maximum difference in gas production for reactor-1 and reactor-2 and it is 155 ml. The average variation in gas production between the reactors is 23%.

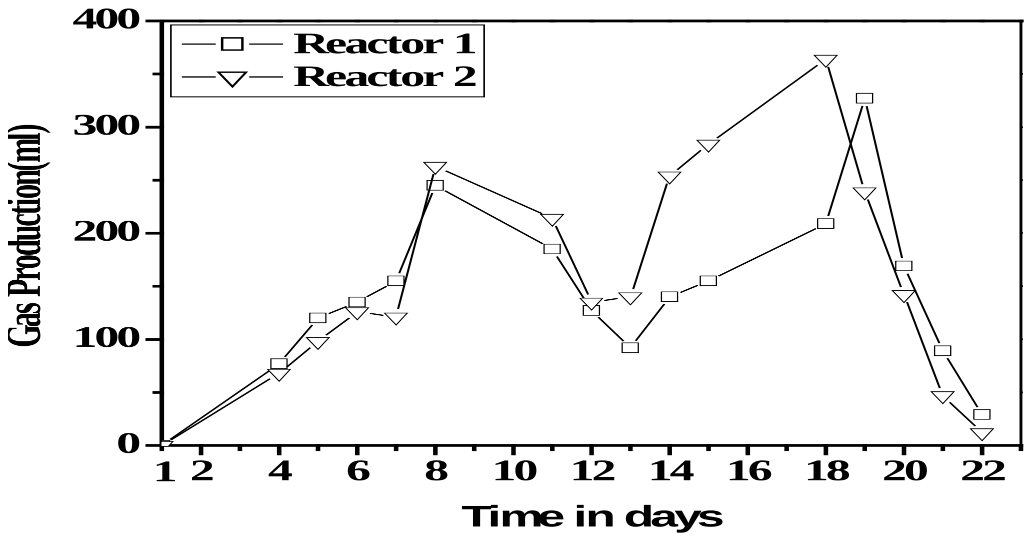

Figure 11: Gas Production from Dheki Shak in two simultaneous reactors

The following figure-12 shows the cumulative gas production from Dheki Shak. Graph shows that the total amount of gas produced in two reactors are 2254ml and 2507ml in 22 days. The figure shows that initially the variation between cumulative gas production for reactor-1 and 2 is very low, but as day passes the variation increases.

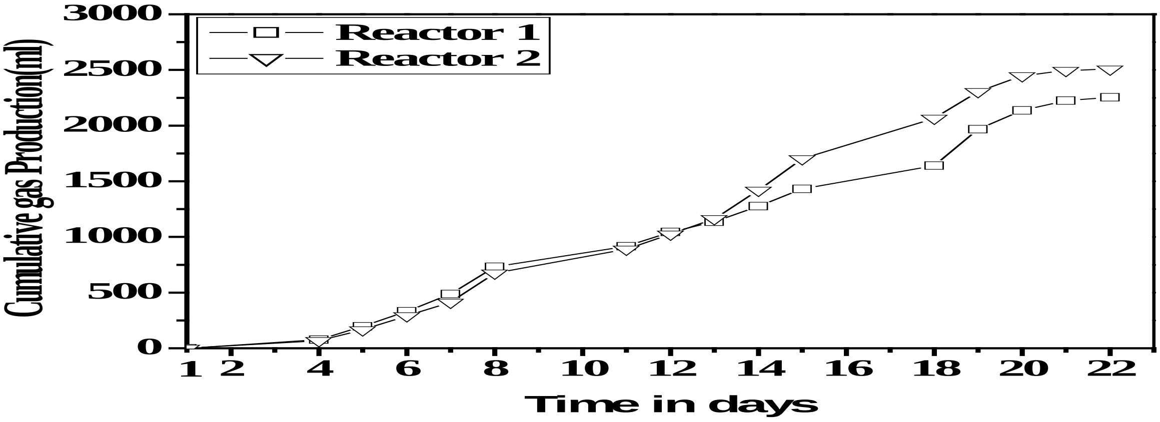

Figure 12: Cumulative gas Production from Dheki Shak

3.2.5 Gas Production from mixture of all items by Continuous Process

Figure-13 represents the gas production from the mixture of Pteris longifolia, Water Hyacinths, Cow Dung. The highest gas production in a single day for reactor-1 and reactor-2 is 652ml and 621 ml respectively. After day-9 both the reactors shows huge drop in gas production, so new feeding is done and gas production increases in both cases. Average variation in gas production for the reactors is 20%.

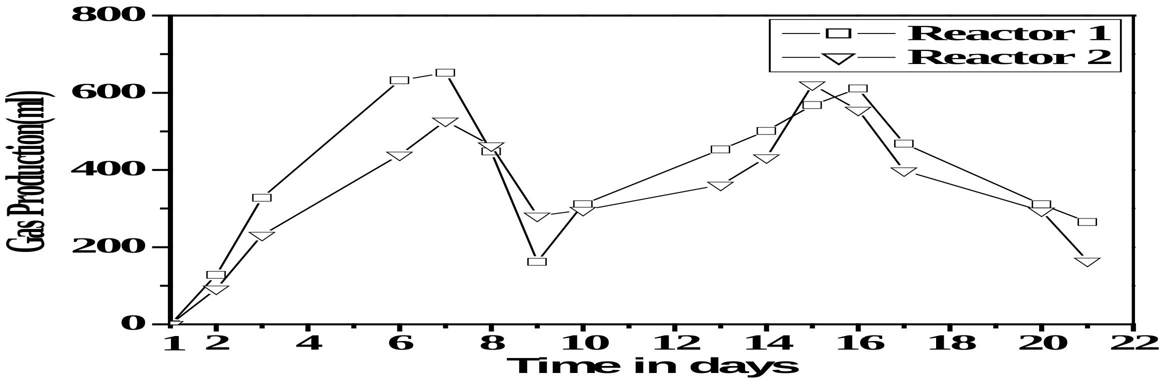

Figure 13: Gas production by continuous process

The following figure 14 represents the Cumulative gas production in two reactors. The amount of gas produced in two reactors is 5838ml and 5154ml in 21 days. The figure shows that the variation in cumulative gas production for the reactors is lowest in the begining and highest in the last day(day-21). The variation is about 11.7% for day-21.

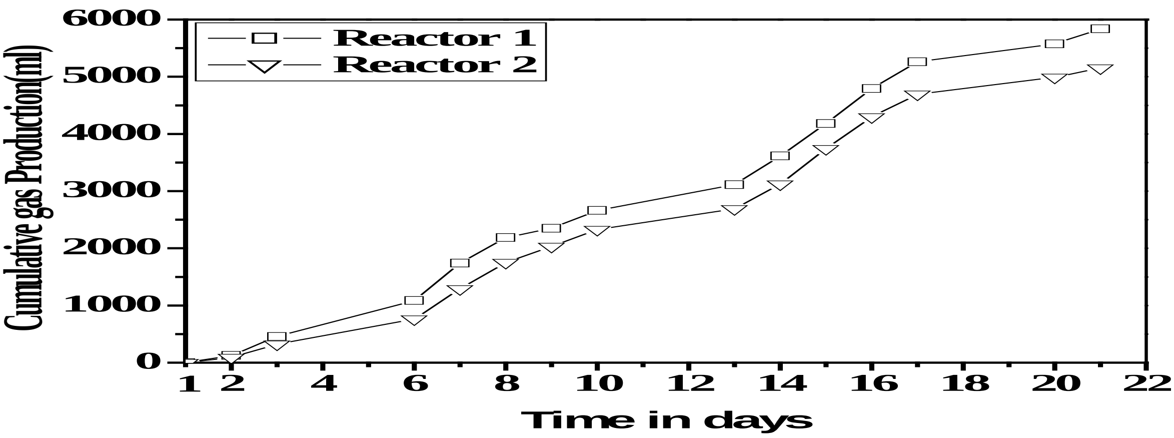

Figure 14: Cumulative Gas productions

4. CONCLUSIONS

Introduction of any new technology is always a difficult task. It involves a tidious and lengthy endeavour of persuasion and motivation. In batch process mixture of the weeds yields 2018ml while cow dung yields 1375ml. It indicates that the mixture yields 1.5 times more than only cow dung. In continuous process Water Hyacinths yields 946ml and 884ml respectively in reactor 1 and reactor 2.It indicates the variation in two reactors is 6.5%. The average gas production is 915ml. In continuous process Dheki Shak yields 2254ml and 2507ml in two reactors. The variation is 10% and the average gas production is 2380ml. In continuous process mixture of the weeds yields 5973ml and 5223ml in two reactor showing the variation 12.5% and average yield 5600ml. Above Discussion shows that total gas yield from mixture is 2 times greater than individually Dheki Shak and 6 times greater than Water Hyacinths. From the results of this work, it can be concluded that biogas production from weeds is an ecologically and economically effective technology.

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WATER BASED URBANIZATION: AN ANALYTICAL STUDY ON DHAKA

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ABSTRACT

Dhaka was once crisscrossed by numerous water channels draining the city as well as serving as a main source of service and communication line and naturally water bodies and Rivers had played an important role in the spatial development, life and liveability of Dhaka. The physiographic effect of water and land can be seen on the settlement pattern, which sited on available high ground or on artificial mounds created on the flood plains. Contemporary urban planning and design in Dhaka has its roots in the colonial period which ignored the traditional pattern resulting in the settlements to turn its back to the water bodies whereby causing health, sanitation and water logging problems. This calls for an urgent attention to integrate the water bodies with the existing city fabric. The study is an attempt to explore the physiographic dynamics of Dhaka along with its socio spatial context and propose guide lines which would respond well to the water based context of Dhaka. The number of key words should be limited to 6 and be formatted in a way similar to this template.

Keywords: *Geo-morphology; Water bodies; Urbanization; Historical Context; Dhaka; Urban Fabric*

1. INTRODUCTION

Dhaka has grown from a small settlement within the confines of River the Bouriganga and the Dholai *Khal* (canal) to a sprawling metropolis of about fourteen million people. Dhaka is encircled by Bouriganga River on its south and south-west, Turag River on its west-northwest and Balu River on its east connected to Turag by Tongi (khal) River to the North. The spatial development followed the prong of flood free terrace originating from the old nucleus along Bouriganga River towards north as a part of Madhupur terrace (Dhaka Terrace) of pre-ostacian age. The Dhaka Terrace sloped towards eastern and western flood plains, marshes and Rivers. Water bodies and Rivers have historically played an important role in the spatial development, life and liveability of Dhaka.

Urbanization, without considering the geo-morphology of Dhaka during recent times has left a deep scar in the city's environment. Dhaka is now at crossroads. It needs some strategic decisions and quick actions to remain liveable. Water logging, pollution, changes in hydro-geological system, land subsidence and building collapse are some of the severe consequences of these environmental changes. In this paper, geo-morphological and hydrological features were explored to ascertain an appropriate role for them for their rejuvenation and integration into the city fabric. This study also attempts to trace the past and present settlement pattern vis-à-vis natural and manmade water bodies in Dhaka and their generic nature.

2. UNDERSTANDING THE CONTEXT

In 1765, James Rennel,(1792) the English Surveyor, wrote, "*the Kingdom of Bengal, particularly the eastern part* (Bangladesh), *is naturally the most convenient for trade within itself of any country in the world; for the rivers divide into such a number of branches that the people have the convenience of water carriage to and from any principal place*." Situated at the centre, Dhaka was able to command all these great water routes. This locational advantage gave rise to various urban settlements during various points in history. *"...the largest town being Dacca city (90,542)...* "(IGI,1908). Dhaka was the seat of provincial Mughal administration for about one hundred years from 1610 AD, and later the capital of the newly formed East Bengal-Assam province (1905-11), during the British colonial period, for a couple of years. In 1947 Bengal was partitioned between India and Pakistan and subsequently Pakistan's Bengal became independent in 1971 under the name of Bangladesh. Dhaka, being the major city in the area was chosen as the capital of the province and then the independent nation. There were political up and downs, affecting the city's size and morphology. But it never ceased to be an important urban centre in this region because of its location and physiography (Fig. 01).

Study of Pre-Mughal settlements in Dhaka and its environs show that those settlements were in response to the need to control the trade and commerce of the region and their locations were determined by the areas from where they could command major (water ways) trade routes (Ahmed, 1960). Banglabazar- Sutrapur area of Dhaka within the confines of River Bouriganga and the Dholai *Khal* being natural harbour became the trade outpost for Sonargaon – the pre-Mughal capital.

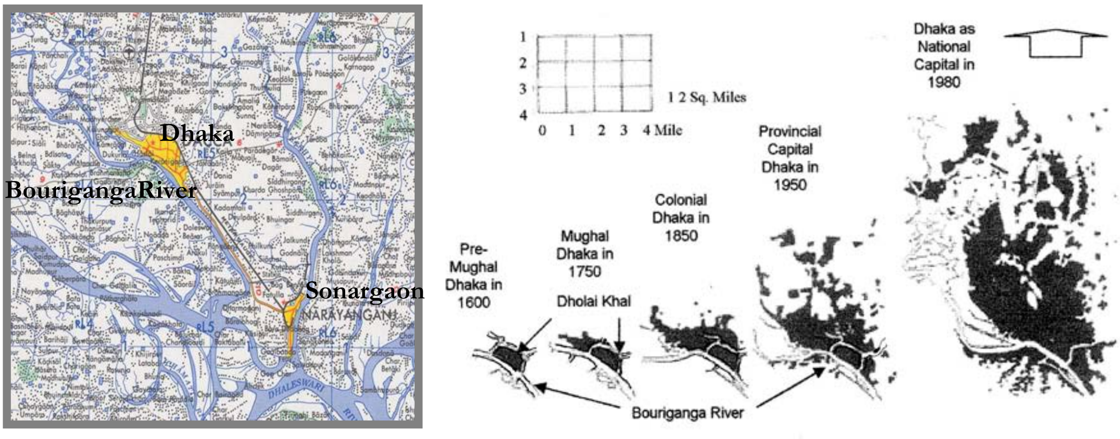

Figure-01: Physiographic Setting of Dhaka and its growth

Contrary to the pre-Mughals, the considerations for Mughals was mainly to guard against the incursion of independent and semi-independent 'Bhuyan' chiefs or to keep out Magh and Portuguese raiders coming from south and east who use to plunder this land frequently (Karim, 1991). Hydrography of Dhaka shows that it is strategically as well as commercially important centre and was a watch post to protect the eastern boarders/lands of the vast Mughal Empire which was upgraded to the position of 'Capital of Suba-e-Bengal' in 1610 AD during the reign of Emperor Jahangir. Since their agenda was not defence or retreat but rather offensive, Lalbagh area with an older Afghan fort was strategically and geo-morphologically more suited to them. They made it the centre of their administration. The city was well protected by a natural system of rivers and a network of canals and low lands. The Mughals had a number of river forts strategically located to control the water ways down streams i.e. Idrakpur fort, Sonakanda fort, Hajiganj forts etc. The city could be approached from the south-east by river, hence a series of forts were built on the way and two forts (Beg Murad fort 1&2) were built on either sides of Dholai *Khal* at Postogola in 16th century, at the entry point of the city. The Postogola watch post were to oversee entries from the south, second line of defence being the older settlement within the Dholai Khal confines.

The city was entered from Bouriganga River in the south from Chandnighat and Swarighat, the path leading to the *Chauk* (the city square) from the river *ghat* (landing piers) had two caravansaries on its two sides i.e. Bara and Chotta Katra. Two principal streets crossed each other at Chouk, one east-west running parallel to the river and another from the river to the defence outpost in the north to Tongi through Tejgaon. Islam Khan, the Mughal General and the Governor, added a canal in the topography of Dhaka to connect Bouriganga River near Pukur Teer (Pakurtali) with Dholai. It was to protect the Mughal sector as well as to create a short cut waterway to Lakhya from Chandnighat Navy base (Karim, 1991). The Dholai taking off from the Balu near Demra flowed south-westward through Dhaka to join Bouriganga near the Modern Mill Barrack area. The Balu rises from the Lakhya near Kapasia and joins the same river at Demra. These water bodies had shaped the Mughal Dhaka.

3. WATER BASED URBANIZATION IN DHAKA – A TRADITIONAL RESPONSE

The Dhaka terrace was crisscrossed by numerous water channels that drained the city as well as served as a main source of service, water supply and communication line. The physiographic effect of water and land can be seen on the settlement pattern, which sited on available ground or on mounds created on the flood plains (Mowla,2005). Due to this local geo-climate and water based development, Dhaka was once befittingly called the Venice of the east (Taylor, 1840).

The settlement pattern of the pre-partition (1947) period in comparison to the contemporary one presents a richly woven urban fabric at the human scale. The rivers spilling over the flood plains and into canals / *khals* or connecting inland depressions or lakes together provided a hierarchy and network of water bodies and navigation routes giving rise to settlements alongside. There were flights of steps, locally known as *ghats* at intervals rising up to the lanes or community spaces (Fig.02). The *ghats* (landing piers) on the bigger water bodies or rivers were major community spaces where the daily activities take place such as bathing, washing or religious or commercial activities. Historic pattern reveal that the relationship of urbanization and water bodies was positive and complementary. *Ghats* were community spaces and urban nodes as well besides providing a hydraulic character to the settlements in the area.

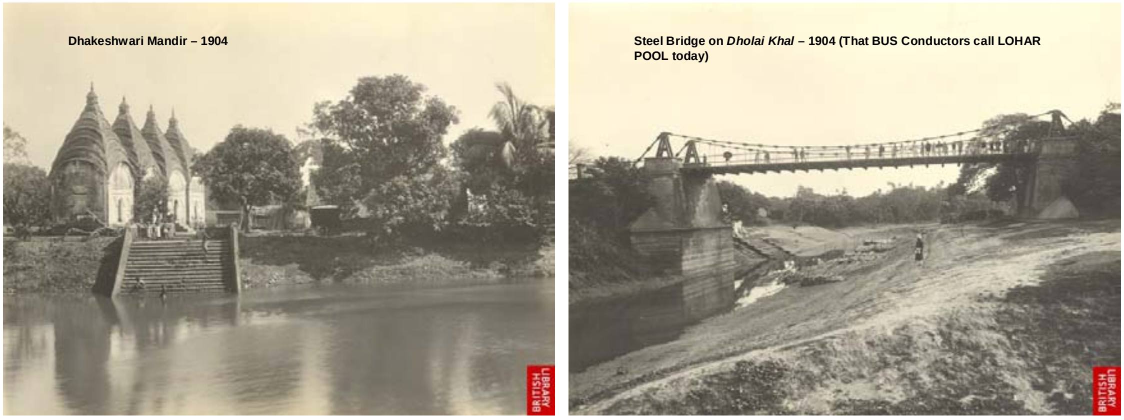

Figure- 02: Glimpses of historic development responding to water.

Figure-03: Settlement Pattern Evolving out of traditional needs in response to water bodies.

The traditional system was of dig-elevate-dwell whereby ensuring water retention during floods and reservoir for lean period. The system protecting the homesteads above flood levels with crop fields around helped create adequate biomass was a time tested approach of the settlement pattern for sustainability in the area (Mowla,2005&2008). In Dhaka traditionally the settlement structure evolved in consonance with water bodies (Fig.03). The water bodies were the main transport corridors, streets being secondary to it. Contemporary urban planning and design in Dhaka has its roots in the colonial period which ignored the traditional pattern resulting in the settlements to turn its back to the water bodies whereby causing hydro-geological as well as health, sanitation and water logging problems.

4. WATER URBANISM – THE EVOLVING PATTERN

Historically Dhaka's urban life and living was interwoven with the system of rivers, canals, lakes and ponds scattered and crisscrossing the city (Mowla, 2008). Water channels like the Dholai Khal, the Gerani Khal, the Segunbagicha Khal or the Begun Bari Khal played an important role in the indigenous city life. Most of these *Khals* lie east-west, that used to serve an important purpose of intracity communication, besides other needs. Boats were the primary means of communication, through numerous *Khals* within Dhaka until the Mughal period when narrow alleys and roads started to appear in the scene to give way to carts (Taylor, 1840). The effect of physiography can be seen on the settlement pattern rather than on building structures themselves. Settlements are sited on available ground or on artificial mounds (Muktadir & Hasan, 1985). Before the contemporary era network of natural canals within the city served as the means of drainage of the rain run off and water during the events of flood, besides these canals served as a good means of transportation.

At the time of partition of 1947, about 50% of present Dhaka was low lying flood plain and water bodies. Dhaka relied on the gravity drainage system based on 'khals' and 'wetlands'. Historically, wetlands and water bodies in Dhaka were the main source of water. Water bodies also offered highly valuable environmental and recreational asset for the area. But gradually in course of time this natural water supply and drainage system is being almost destroyed. Many of the roads in Dhaka are developed by filling the water bodies or by making box culverts, thus shrinking the water carrying capacity as well as reducing ground water replenishment (eg Dholai Khal and Panthapath). Destruction of these water channels and depressions has resulted in the disruption and alteration of the natural process of land accretion, land formation and ecosystems. Traditional Architecture, Urban Design and planning in this region offers the best and integrated solutions towards human needs, in their relation with the nature, ecosystems and the community but contemporary development ignored living with nature. Water logging is an inherent problem associated with uncontrolled urbanization and lack of holistic planning in Dhaka.

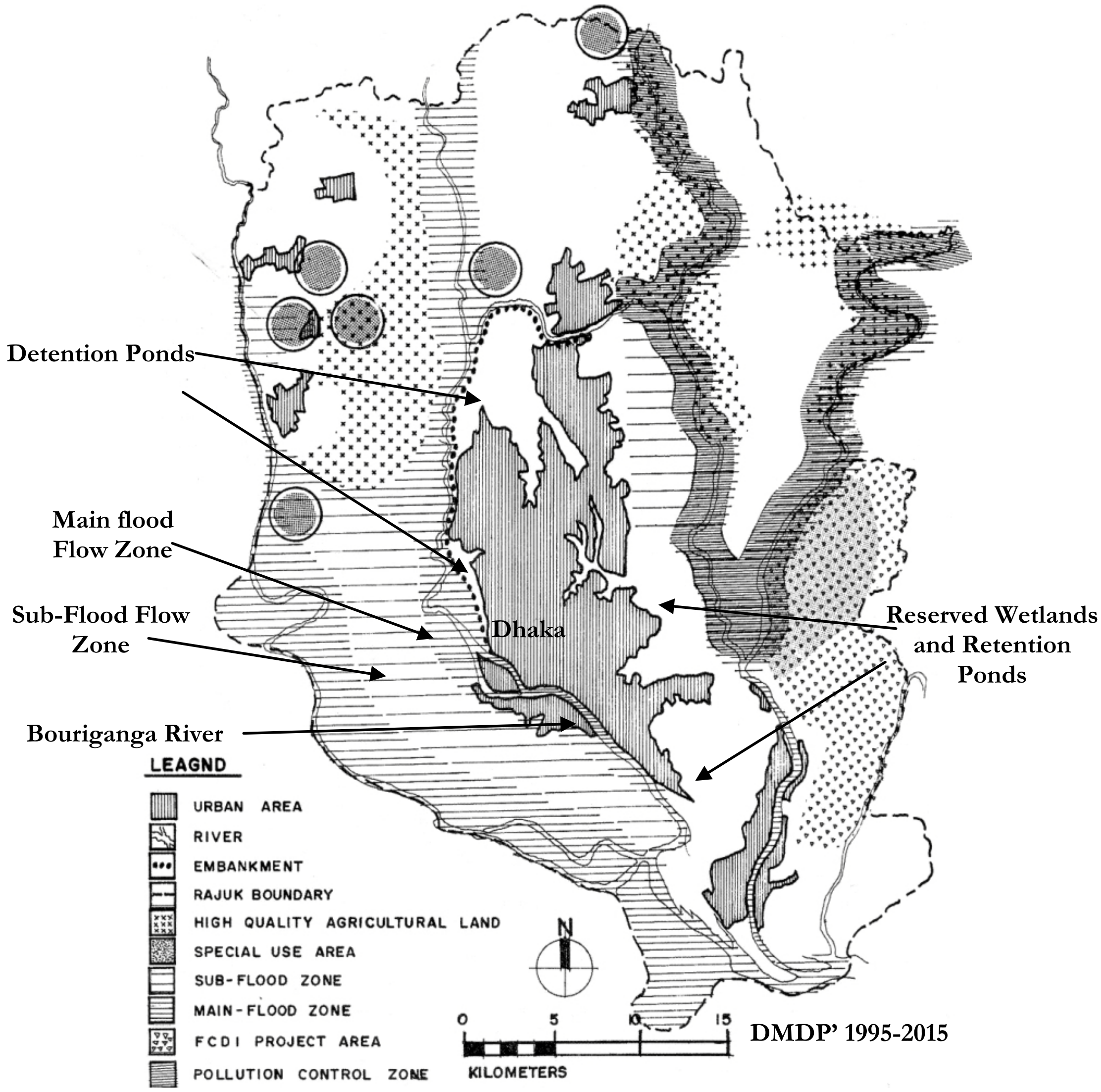

Figure-04: DMDP' 95 - Proposed Plan Policy Areas ie built and natural areas for Dhaka.

5. WATER SCENARIO IN CONTEMPORARY DHAKA

It is said that Dhaka must have at least 20 retention ponds of Hatirjheel size to tackle the storm water (Bangladeshnews,2009a). Dhaka Metropolitan Development Plan (DMDP, 1995)(Fig.04) considered retaining at least eight flood-flow zones undisturbed – Dhaka West; DND Triangle; Eastern Fringe; Narayanganj West; Dhaka NW; and Narayanganj East (Nagario Prokoton, 1995). Transparency International Bangladesh-TIB informed that around 1,000 ponds, which were in the city, have now been totally destroyed. TIB is trying to recover 13 out of 52 canals but could not succeed so far (Bangladeshnews,2009b). TIB reports that 800 acres of land in 5 rivers including Bouriganga and Sitalakkhya were illegally grabbed violating the Wetland Protection Act, 2000. The fact is that the contemporary planning process never took water systems as the driving force in any physical planning in this delta. IRS images of 1996 and 2000 clearly shows that water bodies measuring about 2300sq.m were filled in by the Bashundhara and Bashumoti housing estates and 66 acres filled by Aftabnagar Housing. In 1996 there were 211 acres of water bodies in the Mohamadia Housing Estate and Adabor area of which 91 acres disappeared between 1996 and 2006 and 68 acres from 2006 to 2009 (Rahman, 2010). RAJUK, the main planning body for Dhaka is the major violator of its own plans. RAJUK initiated a residential project in the southern part of Bouriganga River covering an area of 381 acres – over the last two years 3000sq.m of natural water bodies has already disappeared. Purbachal housing is another recent example of many such violations of DMDP,95.

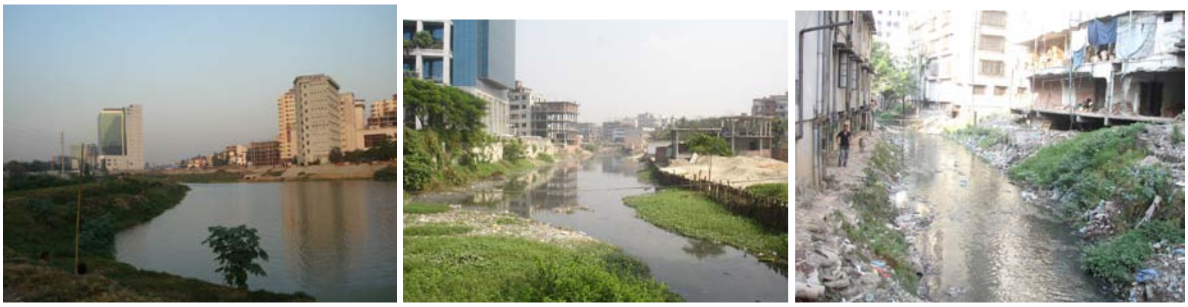

Figure-05: Contemporary Planning with water bodies was never in the development agenda: See the Bagunbari Canal and Kalyanpur canal being encroached indiscriminately.

Studies show that around 40% of the wetlands of Dhaka city has disappeared in 20 years due to indiscriminate filling up of lowland and flood flow zones (both by public and private agencies) that also has reduces its drainage capacity (Fig.05). It says that the temporary wetland area in DMDP,1995 was 1,528 sq.km. which was 40,765 hectare in 1989 and came down to 35,740 hectare in 1999 and 24,208 hectare in 2005 (Bangladeshnews, 2010). Same study informs that to protect eastern Dhaka from floods, at least 40% (66 sq.km) of the drainage catchment area must be delineated and protected as wetlands and water bodies under Wetland Protection Act, 2000, considering this as an 'ecologically critical area'. Of the recommended 40% drainage area, a minimum of 12% (about 20 sq.km) can be made available as reserved ponds or lakes and another 38% (about 46 sq.km) protected as natural wet lands for retention of storm water. A Dhaka University study reveals that the government owns about 33 lakh acres of khas (Government) land of which about a quarter is water bodies. DAP (2007) covers the area of DMDP,1995 but did not follow the planning guidelines provided in it. Against the 40% requirement, DAP,2007 recommends 21% of Dhaka's land as water bodies where no development would be permitted. DAP recommends 50m land from riverbanks to be earmarked for walkway or driveway; enlisting parks, playgrounds and open spaces; and marking existing canals in the CS and RS maps (DAP,2007). No basis of this recommendation is given.

Surface water (River, Canals, Wetlands, Ponds etc) compliment and supplement underground water. There is a continuous tendency in contemporary times to destroy the surface water channels but underground water is also not spared. The ground water replenishment rate is much slower than the extraction rate. In 2001, underground water table in Dhaka was at a depth of between 200-300 feet. In 2010, this has deepened to 1,000 -1200 feet. This is not a matter of concern only with respect to water supply but because a large vacuum between surface and under-groundwater has developed which might trigger land slide or subsidence particularly in the event of earth quakes.

6. THE CRISIS AND THE RESPONSE

Bangladesh is a deltaic country and has abundant rainfall, but due to the absence of clear policy-guidelines, useable water quantity is declining fast. Most countries have water policy and regulatory laws for water use (particularly ground water). Wetland Protection Act, 2000 in Bangladesh restricts change in the wetland areas but doesn't regulate the use of water. Neither the building construction rules, or building code or any planning regulation nor wetland act calls for integrating natural water bodies or channels into the urban planning and design frame work.

Given the fact that Dhaka is subject to frequent flooding, an appropriate plan for living with flood is needed. A formal holistic policy and plan at national level sets priorities identifying major areas and quantities of uses i.e. Forest, wetlands & water bodies, natural areas, agricultural, rural, urban and industrial areas. Urban structure plan (SP such as DMDP,95) is the second level plan within broad national urbanization policy guidelines of national vision. The level three planning and designing is the detail area plan (DAP) within the planning guidelines set in SP. In the fourth level is small area and plot level designing. Such a sequential plan is not evident in the Detailed Area Plan (DAP, 2007), which rather than setting aside adequate space for water retention ponds and for permeable surfaces (including parks and other unpaved surfaces), focuses instead on roads and buildings, embankment and pump oriented flood control approach which further intensified the suffering of Dhaka residents due to flooding. Considering the geo-morphology of Dhaka, contrary to the embankment- and pump-oriented flood control and drainage management approach of DAP, 2007, flood management and a detention reservoir-based gravity drainage system is expected to be more reliable and appropriate for storm water drainage system in a floodplain landscape like Dhaka with rivers encircling the city (Mowla,2010). Some sporadic attempts to integrate water bodies with the settlement pattern are observed in Dhaka without any link with upper and lower level planning and design framework (Fig.06&07).

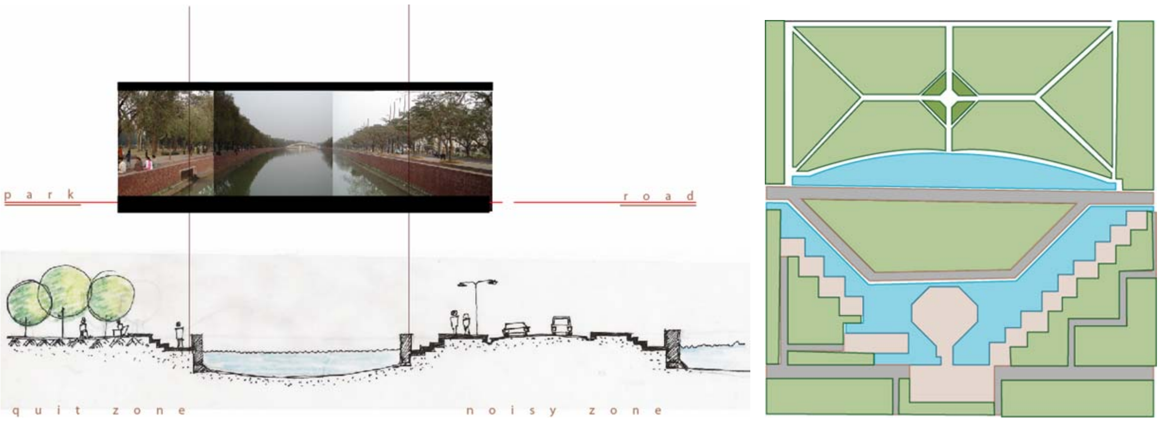

Figure-06: Shangshad Bhaban Precinct design considering water bodies as its integral part.

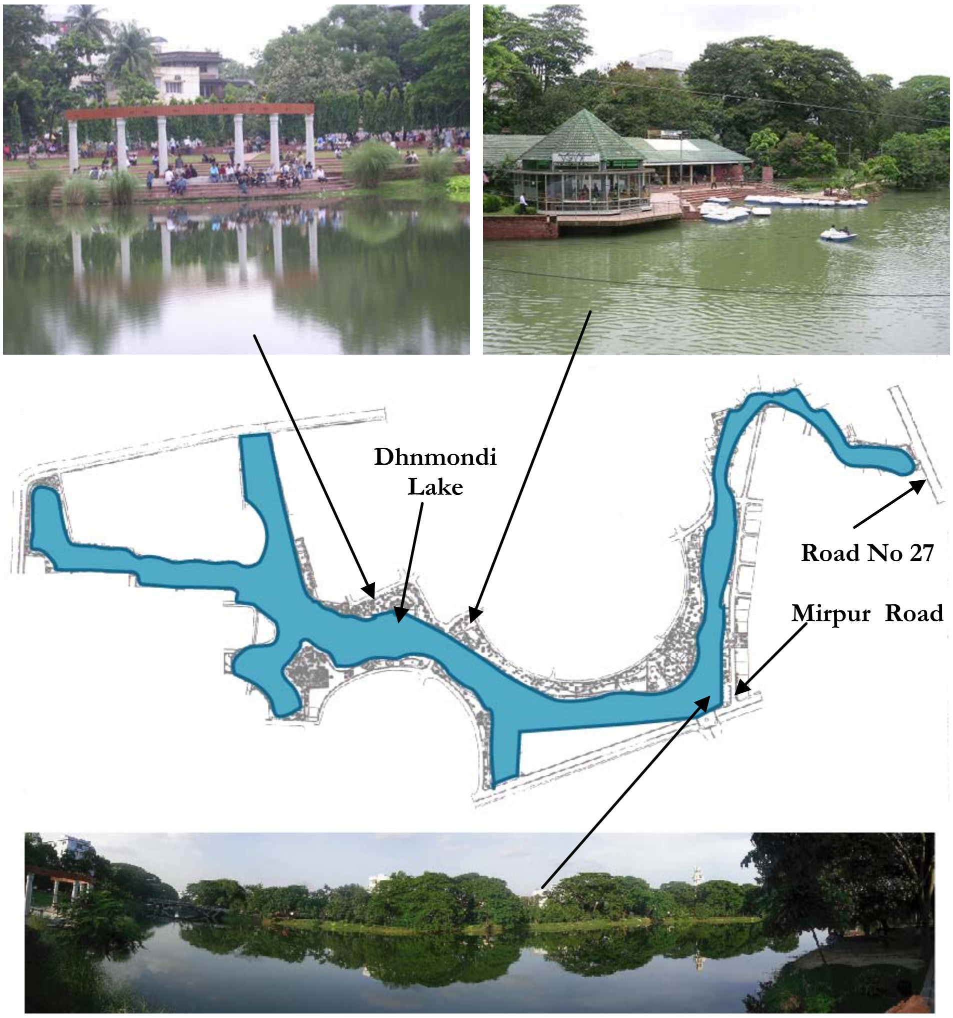

Figure-07: Dhanmondi R/A Designed with water-bodies but not integrated.

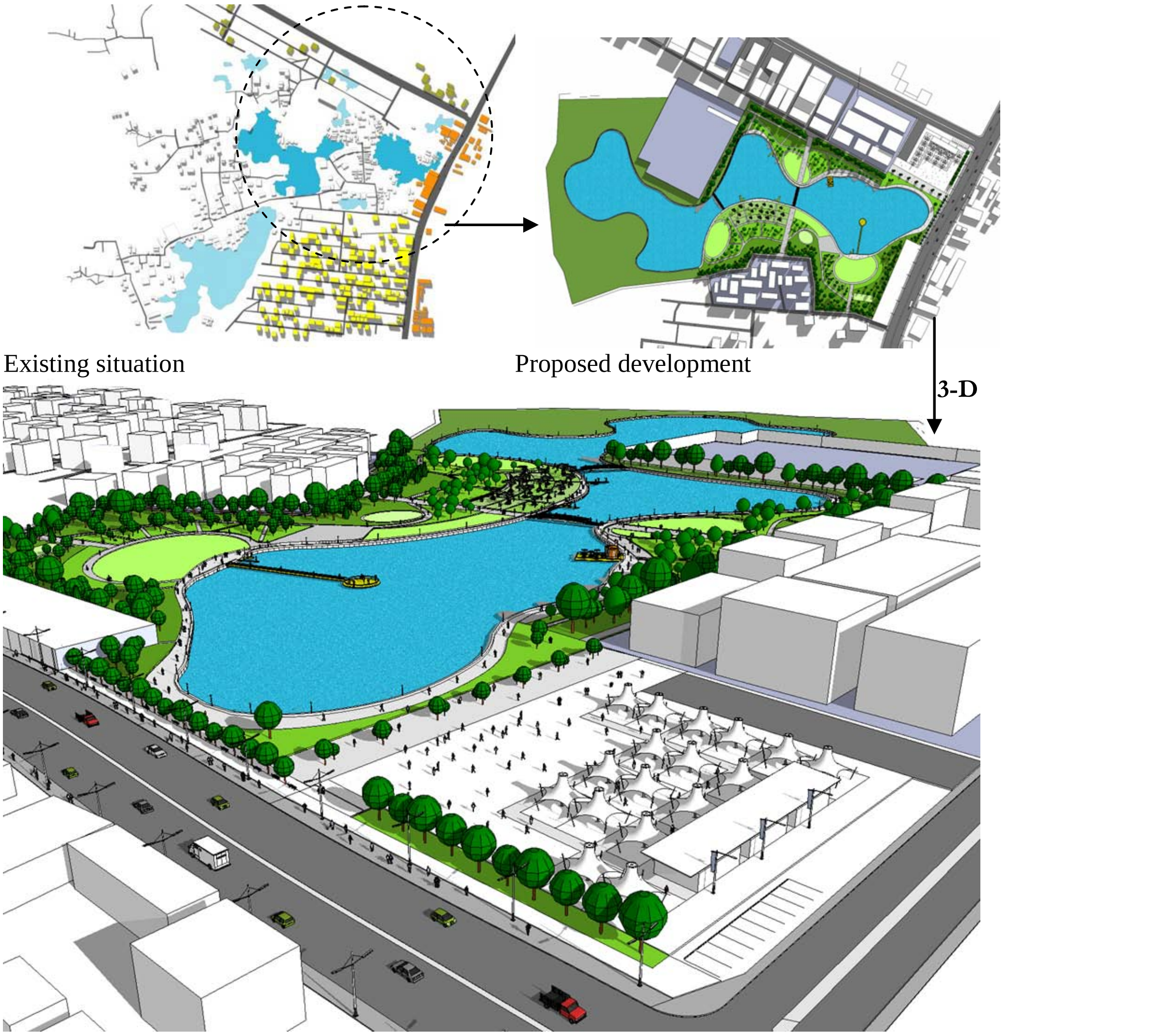

Figure-08: Badda retention pond development area students' vision of development without sacrificing density or contemporary needs but integrating nature with the development.

To assess the sustainability status of integrating water bodies in the urban design under contemporary context without compromising on the social, environmental and economic interests, a hypothetical student exercise was done in the Department of Architecture, BUET and a day long workshop was also organized (BUET,2009) which was participated by the level-4 BArch students of BUET and 5th year architecture students from KUL (Belgium). One of the many proposals submitted by the students (Fig. 08) suggest that traditional approach of planning and design of living with nature is still valid without sacrificing contemporary needs and that natural areas be strictly controlled by regulations. It is also suggested that the Planning and regulatory agencies like RAJUK must not indulge in (real estate) development since monitoring and execution are conflicting roles with conflict of interest (like judiciary and bureaucracy).

Considering the geo-morphology of Dhaka, it seems essential that if and when needed the earth cutting to fill / raise lands must be judiciously planned and invariably done on the channel ward side that would get rapidly filled up by the natural process of siltation / accretion. The western embankment of Dhaka is a response to the floods of 1988. The attitude of zero tolerance for floods amounts to demanding equal protection from floods along different stretches of rivers irrespective of the geographical difference found there, which in practice is not possible. It increases the ferocity of flood flow in the lower reaches. Embankments saves some areas at the expanse of some other areas but have a long term effect of higher river beds than their surrounding, insufficient percolation and diminishing soil fertility. According to noted water resources expert Prof Shahjahan (Mowla,2000) 'such flood control and irrigation projects tend to focus exclusively on engineering aspects, neglecting the people, water, soil, fish, vegetation – many vital concern.' Bernier (an English traveller of 17th century) observed that the overflow irrigation combated malaria, provided an abundant harvest of fish and vegetation, enriched the soil and made congestion of the river impossible.

7. CONCLUSIONS

Historically Dhaka's development responded well with the hydro-geological realities of the place. Traditional Architecture, Urban Design and planning in this region offered the best and integrated solutions towards human needs, in their relation with the nature, ecosystems and the community but contemporary development ignored living with nature. After abusing urban water bodies through the years of hard use and neglect, when it has retaliated with unthinkable magnitude, there is a realization on the stake holders that they are valuable natural assets / resource for the community.

Urban development with water bodies as focus was not given a trial during the rapid urbanization over the last 100 years. Review of Dhaka's geomorphology reveals that for the sake of ecological, hydrological integrity and development sustainability, natural systems must be protected and can be protected. DAP,2007, though quite conservative, if strictly followed is expected to improve this situation. Studies show that much greater environmental as well as socio-economic success or advantages can be achieved through design with ecological principles in mind than without it. Fragmented approach of planning must be avoided.

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CHARACTERIZATION AND CHEMICAL TREATMENT OF LANDFILL LYSIMETER LEACHATE: COAGULATION PROCESS USING FERRIC CHLORIDE AND POLY-ALUMINUM CHLORIDE

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ABSTRACT

This research concentrates on the performance of coagulation process for removing the pollutants in leachate generated from municipal solid waste (MSW) deposited in sanitary landfill lysimeter at KUET campus, Bangladesh. The average characteristics of leachate were: turbidity= 106 NTU; colour= 2190 Pt-Co; TSS= 750 mg/L; COD=1280 mg/L; pH= 7.31; calcium (Ca)= 214 mg/L; potassium (K)=1444 mg/L; sodium (Na)=1489 mg/L; magnesium (Mg)= 227 mg/L; copper (Cu)=0.213 mg/L; zinc (Zn)=0.595 mg/L; cadmium (Cd)=0.043 mg/L; chromium (Cr)=0.06 mg/L; manganese (Mn)= 4.4 mg/L; nickel (Ni)=0.058 mg/L and lead (Pb)= 0.235 mg/L. The coagulation process was accomplished by an integrated technique consisted of ferric chloride ($FeCl_3$) and poly-aluminum chloride (PAC) in various dosages and pH. In the laboratory, standard coagulation-flocculation experiments were performed in a conventional Jar-test apparatus. Result showed that $FeCl_3$ was performed to optimum reduction at optimum coagulant dosage (OpCD) of 1.5g/500mL at pH 7 and 110 rpm mixing speed, while, PAC at OpCD of 2.0g/500mL at pH 7 with the same mixing speed. It was noticed that $FeCl_3$ was found to be effective coagulant in leachate treatment, prior to biological treatment because it was able to give more reduction of pollutant than PAC.

Keywords: *Landfill lysimeter, chemical coagulant, leachate pollutants, optimum, regression coefficient.*

1. INTRODUCTION

Landfilling is the most common technique for MSW disposal. Sanitary landfill leachate is a heavily polluted wastewater which has high potential to pollute subsoil and ground water (Li et al., 1999 and Wichitsathian et al., 2004). Although different natural and synthetic liners are used to prevent the movement of leachate into the ground water, no landfill liner can capture the entire generated leachate (Rafizul et al., 2009). Leachate is one of the major problems to overcome in managing the landfill. The collection and treatment of leachate has become common practice in order to prevent environmental pollution (Rafizul et al., 2012). Therefore, studies for the development of viable technologies for leachate treatment are important. Leachate presents considerable variations in both volumetric flow and chemical composition (Tatsi et al., 2003) due to the variations in the composition of deposited solid wastes, landfill age, hydrogeology of landfill site, specific climate conditions, moisture routing through the landfill (Chu et al., 1994) and design and operation of the landfill (Aziz et al., 2004). Moreover, leachate is generated from MSW due to precipitation, surface run-off, and percolation of water through landfill (Ntampou et al., 2006). It can cause great environmental problems if it gets into underground water, especially because it has organic matter, heavy metals, chlorinated organic and inorganic salts. However, some of these pollutants can be degraded by microorganisms (Wang et al., 2002).

Several wastewater treatment processes have been applied to treat landfill leachate: aerobic and anaerobic biological degradation, chemical oxidation, chemical precipitation, coagulation-flocculation, activated carbon adsorption and membrane processes (Tatsi et al., 2003 and Amokrane et al., 1997). Rivas et al. (2004) reported that, leachate treatment technologies can be classified into three major groups: (a) leachate channeling: recycling, lagooning, etc., (b) biodegradation: aerobic and anaerobic processes and (c) chemical and physical methods: oxidation, adsorption, etc. Biological processes are especially efficient in the treatment of young landfill leachates that are rich in volatile fatty acids (Amokrane et al., 1997 and Silva et al., 2004).
Physico-chemical processes are used for the pretreatment of young leachate to make it amenable to biological treatment, and to hydrolyze some refractory organic compounds in older landfill leachate (Chian, 1977). In practice, a combination of physical, chemical and biological methods are usually used for the treatment of leachate since it is difficult to obtain satisfactory effluent water quality using any one of these methods alone (Kargi and Pamukoglu, 2003).

Coagulation and flocculation is one of the widely used physico-chemical methods of water and wastewater treatment (Imran et al., 2011). Coagulation-flocculation, as a treatment process in leachate treatment, is often recommended for removing pollutant parameters from stabilized leachate (Rivas et al., 2003; Duan and Gregor, 2003 and Beauchesnea et al., 2005). Common coagulants used for leachate treatment include aluminum sulfate (alum), ferrous sulfate, ferric chloride and ferric chloro-sulfate (Amokrane et al., 1997). Iron salts are generally more efficient than aluminum ones. The present study was conducted to evaluate the suitability of coagulation-flocculation as the physico-chemical methods using the chemical coagulants of $FeCl_3$ and PAC for the treatment of leachate which collected from leachate collection chamber of sanitary landfill lysimeter at KUET campus, Khulna, Bangladesh. At the initial stage, the optimum dosage of the used different coagulants was determined for the removal of relevant parameters then these optimum doses were selected for the treatment of leachate at the ranges of pH values of 4 to 9. In the laboratory through standard methods, analysis was performed on the leachate samples and several parameters were measured and monitored. The main objectives of this study as reveals: (i) to determine the characteristics of untreated leachate; (ii) to study the effectiveness of treating leachate using: $FeCl_3$ and PAC and (iii) to compare the efficiencies of the used chemical coagulants to reduce the inorganic compounds and metals concentration.

2. MATERIALS AND METHODOLOGY ADAPTED

2.1 Analytical Methods for the Assessment of Leachate

This research concentrates on the leachate of sanitary landfill lysimeter at KUET campus, Bangladesh. Rafizul et al. (2009) reported that three landfill lysimeters designed as A, B and C were designed and constructed where three different situations were considered. The deposited MSW into each lysimeter mainly consists of 93% organic (food and vegetables), 3% of plastic/polythene and 2% of leather/rubber and 2% of others. The organic and moisture content of MSW was found 52% and 65%, respectively, and the total volume was $2.80m^3$ (height 1.6m) with a manual compaction to achieve the unit weight of 1,064 kg/m^3. Moreover, a leachate collection tank (3.68x1.56x1.64m) accommodating four separate leachate discharge pipes in the temporary collection and storage containers, were constructed using 250mm thick brick.

Raw leachate was taken from the leachate collection chamber for laboratory analysis to establish the mean characteristics. Leachate samples were analyzed for physico-chemical parameters of pH by pH meter (HACH, Model No. Sens ion 156), chloride by potentiometric titration method using silver nitrate solution, conductivity by conductivity meter (HACH, Model No. Sens ion 5), alkalinity by titration method, hardness by EDTA titrimetric method, COD by closed reflexive method, turbidity using turbidity meter and colour using spectrophotometer (HACH; DR/2400) as per the Standard Methods (APHA, 1998). However, the Ca, Na, K and Mg ions were determined using flame atomic absorption spectrophotometer (VARIAN; AA/2400). In addition, Heavy metals viz. Cu, Zn, Cd, Ni and Pb were analyzed using spectrophotometer (HACH; DR/2400) as per the Standard Methods (APHA, 1998). Table 1 shows the leachate characteristics resulting from four samplings at two weeks interval for about 2 months from 1st July-30th August 2010.

2.2 Coagulation and Flocculation

Coagulation-flocculation experiments were performed in a conventional Jar-test apparatus equipped with six backers using $FeCl_3$ and PAC. For the jar-test experiment, leachate samples were removed from the cold room and were conditioned for about 1 hour under ambient temperature. Sample container was thoroughly shaken for re-suspension of settled solids. 500 mL of raw leachate was transferred into the 1-litre beakers with the height of 15 cm and internal diameter of 10 cm. Leachate samples were subjected to jar-test after pH adjustment and the pre-selected chemical coagulant addition. The experimental procedure consisted of three subsequent stages; the initial rapid mixing stage of 3 min at 110 rpm, the following slow mixing stage of 25min at 40 rpm and the final settling step for 30 min.

The pH of leachate were adjusted to the desired levels by addition of the appropriate amounts of 6N sodium hydroxide or 6N sulfuric acid. After the settling period, the supernatant was withdrawn from the beaker and was checked by chemical analysis. The withdrawal of sample was accomplished from a point about 2cm below the liquid level in the beaker by a pipette. A study established by Ebeling et al. (2003) stated that the combined of coagulation-flocculation studies are carried out in usual jar test equipment. For more than 50 years, the jar test has been the typical technique used in waste and drinking water industry to improve the addition of coagulant and flocculants (Galvez et al., 2005). The adding of coagulant in leachate, adjusted of pH to the desired levels, mixing speed for Jar-test apparatus in the laboratory were trailed based on the procedure followed by Hamzeh et al. (2009) and Ebeling et al. (2003).

Table 1 Characteristics of raw leachate of MSW in sanitary landfill lysimeter

Characteristics	Range	Mean*
Inorganic and organic compound		
pH	6.14-8.0	7.31
Turbidity	55-201	106
Leachate colour	1450-2340	2190
Chloride	1150-3580	2395
Total suspended solid (TSS)	18-1390	750
Hardness	1000-10000	2372
Alkalinity	980-10000	2930
Chemical oxygen demand (COD)	160-800	510
Conductivity	3.10-24.70	9.38
Metal concentrations		
Calcium (Ca)	153-256	214
Potassium (K)	856-1764	1444
Sodium (Na)	1012-2212	1489
Magnesium (Mg)	127-540	227
Iron (Fe)	1.80-82.6	25.0
Heavy metal concentrations		
Cadmium (Cd)	0.04-0.105	0.043
Nickel (Ni)	0.045-0.09	0.058
Zinc (Zn)	0.10-1.27	0.595
Copper (Cu)	0.04-0.97	0.213
Chromium (Cr)	0.01-0.08	0.06
Manganese (Mn)	0.80-8.50	4.40
Lead (Pb)	0.12-0.476	0.235

All values in mg/L, except pH, turbidity (NTU), leachate colour (Pt-Co) and Conductivity (mS/cm)
** Average of 4 samples was taken between 1st July and 30th August 2010.*

3. RESULTS AND DISCUSSIONS

3.1 Chemical Coagulant at Varying Concentrations

Experiments were conducted using $FeCl_3$ and PAC at varying OpCD for the removal of inorganic compounds in terms of turbidity, colour, COD, TSS as well as metal concentrations of Ca, K, Na and Mg and heavy metal concentrations of Cu, Zn, Cd, Ni and Pb in leachate and also illustrates in Figures 1-6. However, the effect of various dosages of using chemical coagulants and pH was also investigated and hence discussed in followings.

Table 2 Results of concentration dosage for treating leachate using $FeCl_3$

Parameters	Untreated leachate	Concentration (dosage) of chemical (500mL) (treated leachate)						
		0.5	1	1.5	2	3	4	4.5
Turbidity	106	52.3	7.58	5.29	6.83	7.82	20.6	21.5
Colour	2190	520.0	456.0	297.0	355.0	420.0	401.0	520.0
COD	510	345.0	272.0	255.0	268.0	282.0	310.0	380.0
TSS	750	190.0	170.0	140.0	310.0	410.0	420.0	520.0
Ca	214	103.0	12.0	0.0	10.0	14.0	28.0	35.0
K	1444	469.0	367.0	0.0	256.0	356.0	567.0	798.0
Na	1489	986.0	567.0	0.0	235.0	543.0	976.0	987.0
Mg	226.58	165.0	89.0	45.0	56.0	76.0	98.0	156.0
Cu	0.213	0.128	0.09	0.0	0.07	0.092	0.112	0.097
Zn	0.595	0.265	0.30	0.04	0.20	0.267	0.223	0.437
Cd	0.043	0.023	0.0013	0.0	0.012	0.027	0.029	0.034
Ni	0.058	0.026	0.03	0.0	0.004	0.027	0.022	0.044
Pb	0.235	0.126	0.11	0.0	0.12	0.127	0.122	0.147

COD=chemical oxygen demand, TSS=total suspended solid, Ca=calcium, K=potassium, Na=sodium, Mg=magnesium, Cu=copper, Zn=zinc, Cd=cadmium, Ni=nickel and Pb=lead. All values in mg/L, except turbidity [NTU] and colour [Pt-Co].

3.1.1Treatment with Fecl₃ at Different Concentrations

After treating of leachate with $FeCl_3$ at varying dosages, the concentration of turbidity, leachate colour, COD, TSS, Ca, K, Na, Cu, Zn, Cd, Ni and Pb in leachate was studied and hence presented in Table 2. However, percentage reduction of pollutant parameters in leachate in relation to the increasing of $FeCl_3$ dosages was studied and presented in Figure 1-3. Based on Figure 1-3, it can be noted that the increasing trend of percentage reduction in relation to the increasing of $FeCl_3$ dosages continued approached the OpCD after which the reduction rate started to reduce gradually. The similar trend was also observed by Hamidi et al. (2007). When $FeCl_3$ was added to leachate and stirred for a few minutes, ferric hydroxide micro floc agglomerated into longer, easily settleable flocs (Tatsi et al., 2003).

Based on Figure 1, it can be depicted that the reduction of TSS was 81% using $FeCl_3$ at OpCD of 1.5g/500mL. Similar research was performed by Tatsi et al. (2003) who reported that the optimal removal of TSS was 80% when $FeCl_3$ at OpCD of 1.5g/500mL was added to stabilize leachate. However, a study conducted by Jayabala (2005) found that $FeCl_3$ was able to achieve 97% removal of TSS in wastewater with 300mg/L. In this study, the reduction of turbidity was 95%, while for colour and COD was found 86 and 50%, respectively. The minimum removal of turbidity was 51% at a dosage of 0.50g/mL while the minimum reduction of colour was 76% at a dosage of 0.50g/mL and 4.50g/mL. Hamidi et al. (2007) found that the removal of colour and turbidity (at 200 mg/L of $FeCl_3$) was 66% and 57%, respectively.

Moreover, the maximum reduction of Ca, K and Na using $FeCl_3$ at OpCD of 1.5g/500mL was 100%, while Mg was reduced by 80% as shown in Figure 2. The reduction of heavy metal of Cu, Zn, Cd, Ni and Pb was also observed when the leachate sample was treated with $FeCl_3$ at OpCD of 1.5g/500mL as shown in Figure 3. Figure 3 reveals that the removal enhanced with increase in coagulant dosage approached the optimum point after which the reduction rate started to reduce gradually. Two mechanisms by which high dosage can improve the coagulation rate are: by increasing the concentration of metal hydroxide precipitates and thus increasing the aggregation rate and by catching particulates into even larger aggregates by sweep foc coagulation (Duan and Gregor, 2003). Results showed that the maximum reduction of Cu, Cd, Ni and Pb using $FeCl_3$ at OpCD of 1.5g/500mL was 100%, while Zn was reduced by 93% (Figure 3). Similar research was performed by Tatsi et al. (2003). He found that the maximum reduction of Cd, Pd and Cu using $FeCl_3$ was 100% and while Zn was reduced by 91%.

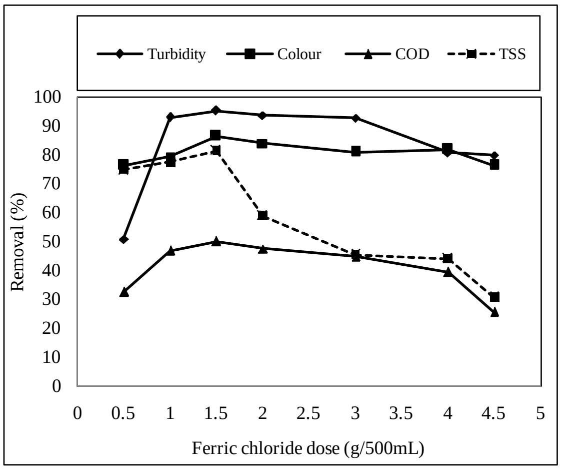

Figure 1 Percentage reduction of turbidity, colour, COD and TSS using $FeCl_3$ dose.

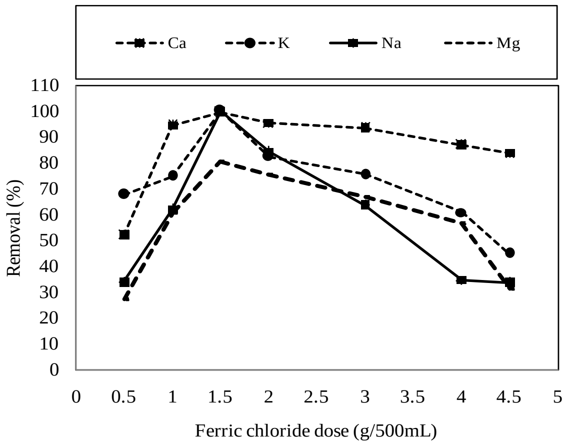

Figure 2 Percentage reduction of Ca, K, Na and Mg in leachate using $FeCl_3$ dose.

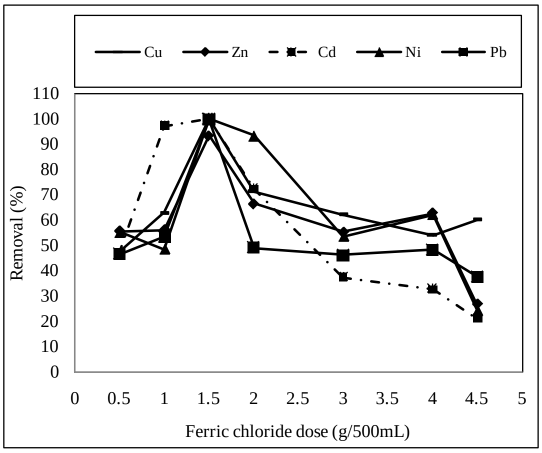

Figure 3 Percentage reduction of Cu, Zn, Cd, Ni and Pb using $FeCl_3$ dose.

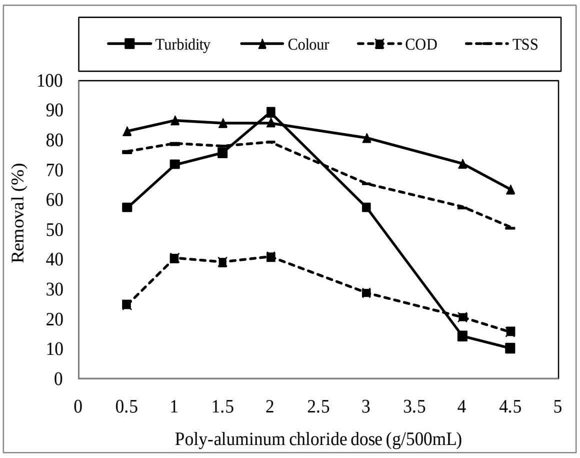

Figure 4 Percentage reduction of turbidity, colour, COD and TSS using PAC dose.

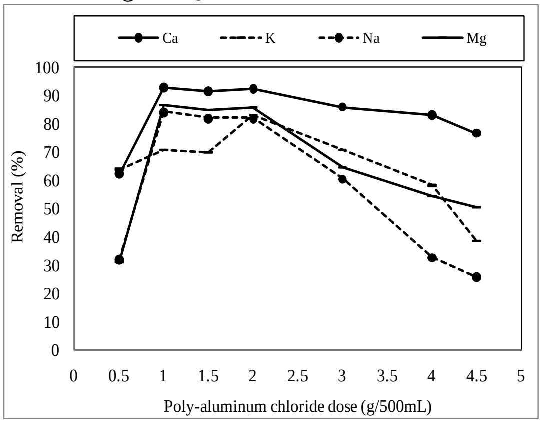

Figure 5 Percentage reduction of Ca, K, Na and Mg in leachate using PAC dose.

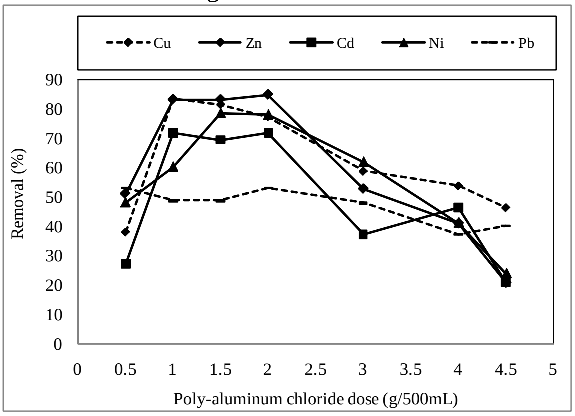

Figure 6 Percentage reduction of Cu, Zn, Cd, Ni and Pb in leachate using PAC dose.

3.1.2 Treatment with PAC at Different Concentrations

Figures 4, 5 and 6 show the result with the efficiency in removing of inorganic compounds of turbidity, leachate colour, COD, TSS as well as metal concentrations of Ca, K, Na and Mg and heavy metals of Cu, Zn, Cd, Ni and Pb in leachate, respectively, treated with increased addition of PAC. The experimental result reveals that the reduction of turbidity with increased addition of PAC at OpCD of 2.0g/500mL was 89%, while, for leachate colour, COD and TSS it was found 86, 41 and 79%, respectively as shown in Figure 4. The experimental result also showed that the maximum reduction of Ca using PAC was found 93%, while, for K, Na, and Mg it was 83, 82 and 86%, respectively (Figure 5). On the other hand, in case of heavy metals, the efficiency in removing Zn using PAC was 85%, while for Cu, Cd, Ni and Pb it was found 77, 72, 78 and 53%, respectively (Figure 6).

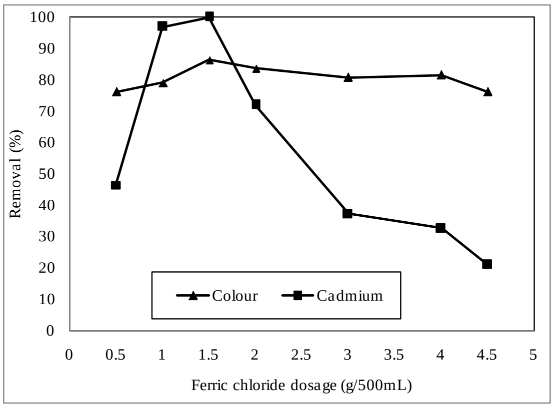

Figure 7 Effect of concentration of $FeCl_3$ for removal of leachate colour and Cd in leachate.

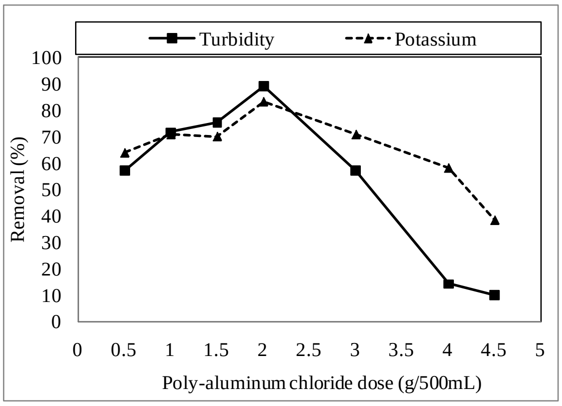

Figure 8 Effect of concentration of PAC for removal of turbidity and K in leachate.

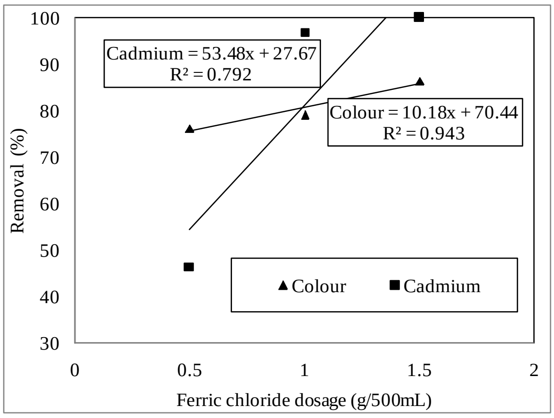

Figure 7a Regression analyses for removing leachate colour and Cd using $FeCl_3$ approached the OpCD.

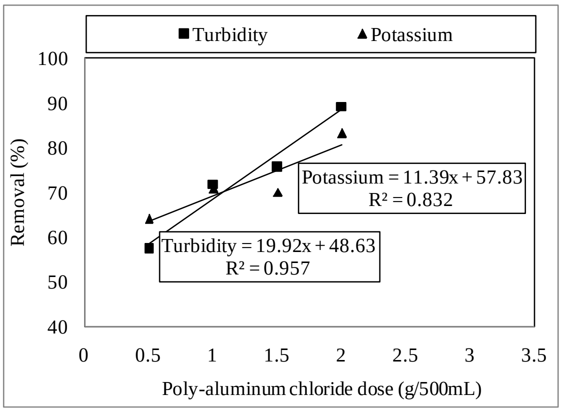

Figure 8a Regression analyses for removing turbidity and K using PAC approached the OpCD.

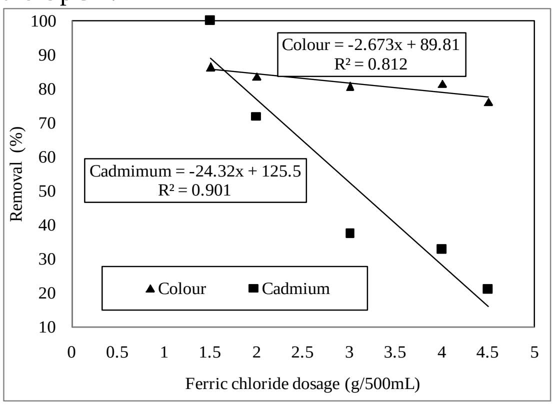

Figure 7b Regression analyses for colour and Cd using $FeCl_3$ after the OpCD.

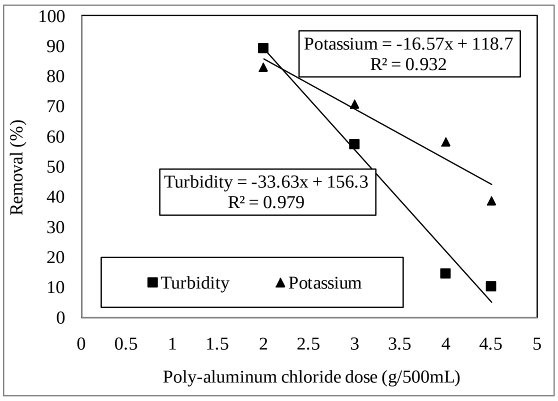

Figure 8b Regression analyses for turbidity and K using PAC after the OpCD.

3.2 Regression Coefficient for Optimum Coagulant Dosage (OpCD)

A researcher Tatsi et al. (2003) stated that to depict the accuracy of mixing amount of coagulation dosages and the performance of using chemical coagulants at varying concentrations for removing of pollutant in leachate may be represented by the regression coefficient (R^2). In the present study, the regression coefficients were analyzed to investigate the accuracy and efficiency of removing pollutant and hence evident in Figures 7-8. Figure 7 shows the correlation between the effects of different concentrations of $FeCl_3$ on the removal of colour and cadmium in leachate. The curve increased up to the OpCD of 1.5g/500mL and then it was decreased. Here, it is important to note that by separating the curve into two lines (Figures 7 a & b), regression analysis was performed to investigate the correlation of concentration of $FeCl_3$ on the removal of contaminants in leachate. It was showed that they are linearly significant and R^2 ranged between 0.943 to 0.812 and 0.792 to 0.901 for colour and Cd, respectively. For example the regression coefficient for removing colour was R^2 = 0.943

approached the OpCD (Figure 7a), and $R^2 = 0.812$ after the OpCD (Figure 7b). However, the curve (Figure 7) was then split into two parts for the analysis of Cd as also shown in Figures 7a & b. Similar trends were also observed when regression analysis was performed to examine the relationship of $FeCl_3$ and removal of metal concentration. The regression coefficient for removing of Cd was 0.792 approached the OpCD (Figure 7a), and after the OpCD R^2=0.901 (Figure 7b). However, Jayabala (2005) established that there was significant correlation in as removal of inorganic and metals concentrations treatment with $FeCl_3$, and the findings in this study were agreed well with the postulation given by Jayabala (2005). Figure 8 shows the correlation at varying concentrations of PAC and the removal of turbidity and K. In case of PAC, the curve increased to the OpCD of 2.0g/500mL and then it was decreased. It showed that they are linearly significant and R^2 ranged 0.957 to 0.979 and 0.832 to 0.932, for turbidity and K, respectively. For example the regression coefficient for removing turbidity and K was R^2=0.957 and 0.832 approached the OpCD (Figure 8a), and R^2 = 0.979 and 0.932 after the OpCD (Figure 8b), respectively. Moreover, the regression coefficient for removing colour and Zn was R^2=0.843 and 0.982, approached the OpCD and R^2=0.953 and 0.938 after the OpCD, respectively.

3.3 Effect of pH on the OpCD of Coagulation

Different pH was tested while the OpCD of different coagulants was applied to perform the coagulation-flocculation tests. The range of pH was examined from 4 to 9 of using $FeCl_3$ and PAC and as shown in Figures 9-12, while the original pH value of raw leachate was 7.31. From Figures 11-16, efficiency of coagulation-flocculation was obtained in the pH of 6.5 to 7.5 for all the coagulant (approximately near the original pH) and the optimum pH was 7.0. However, in order to evaluate the effect of pH on the reduction of inorganic compounds in terms of turbidity, leachate colour, COD, TSS as well as metal concentrations of Ca, K, Na and Mg and heavy metal of Cu, Zn, Cd, Ni and Pb in leachate, experiments were conducted by different chemical coagulants at varying pH ranging from 4 to 9 and hence discussed in followings.

Table 3 Results of leachate treatment with various pH at optimum concentration using $FeCl_3$

Parameters	Untreated leachate	Treated of leachate with pH at optimum concentration of $FeCl_3$					
		pH 4	pH 5	pH 6	pH 7	pH 8	pH 9
Turbidity	106	28.72	21.54	11.42	6.36	8.48	13.6
Colour	2190	540.0	358.40	306.60	262.80	306.0	350.40
COD	510	301.0	285.0	264.0	255.0	268.0	295.0
TSS	750	200.0	180.0	165.0	157.50	172.50	187.50
Ca	214	34.0	28.0	12.0	9.0	12.0	14.0
K	1444	364.0	256.0	245.0	201.0	236.0	268.0
Na	1489	317.0	259.0	227.0	201.0	256.0	236.0
Mg	227	83.0	48.0	44.0	32.0	90.0	96.0
Cu	0.213	0.40	0.40	0.40	0.30	0.50	0.50
Zn	0.595	0.160	0.115	0.105	0.100	0.120	0.127
Cd	0.040	0.021	0.012	0.011	0.0	0.002	0.015
Ni	0.060	0.031	0.022	0.011	0.0	0.012	0.015
Pb	0.24	0.120	0.100	0.021	0.0	0.022	0.017

COD=chemical oxygen demand, TSS=total suspended solid, Ca=calcium, K=potassium, Na=sodium, Mg= magnesium, Cu=copper, Zn=zinc, Cd=cadmium, Ni=nickel and Pb=lead. All values in mg/L, except turbidity [NTU] and colour [Pt-Co].

3.3.1 Effect of pH on Coagulation with $FeCl_3$

The effect of pH on the reduction turbidity, colour, COD, TSS, Ca, K, Na, Mg, Cu, Zn, Cd, Ni and Pb while maintaining the concentration of $FeCl_3$ at the OpCD of 1.5g/mL as presented in Table 3 and in also Figures 9 and 10. The Figure 9 reveals that the maximum removal of pollutant using $FeCl_3$ at OpCD of 1.5g/500mL at pH 7, while, the minimum removal was found for most of the pollutants at pH 4. At pH 7, $FeCl_3$ was effective in removing turbidity, colour, COD and TSS by 94, 88, 50 and 79%, respectively. However, the minimum reduction of pollutant occurred at pH 4 using $FeCl_3$ and it was 73% for turbidity, 75% for colour, 41% for COD and 73% for TSS. These results are the same as these obtained by Nor Asikir and Agamuthu (2007), who reported that the optimum pH in reducing pollutants by $FeCl_3$ was pH 7. Moreover, the reduction of Ca, K, Na and Mg and Cu, Zn, Cd, Ni and Pb was also observed when leachate was treated with $FeCl_3$ at different pH and the highest reduction was at pH 7 as shown in Figure 10. Based on Figure 10, it can be seen that the reduction for Ca, K, Na and Mg was reduced by 96, 86, 86 and 86%. The reduction of Cd, Ni and Pb with $FeCl_3$ at pH 7 was 100% except for Cu and Zn, it was 81 and 83%, respectively. The minimum reduction of heavy metals occurred at pH 4 using $FeCl_3$ and it was 81% for Cu, 73% for Zn, 51% for Cd, 47% for Ni and 49% for Pb. According to Jayabala (2005) the treatment of leachate to remove As and Cd with $FeCl_3$ at optimum concentration (40mg/L) achieved 99.8% reduction of As and 94% of Cd at pH 6. However, Tatsi et al. (2003) found that color removal was 100% with the addition of $FeCl_3$ to stabilized leachate, especially when the pH was adjusted at 10. Slightly lower efficiencies (up to 93%) were measured for fresh leachate, while Letterman et al. (1999) stated that the optimal pH depends on the leachate characteristics. So, it can be concluded that the findings in this study are in agreement well with the results of the other researchers.

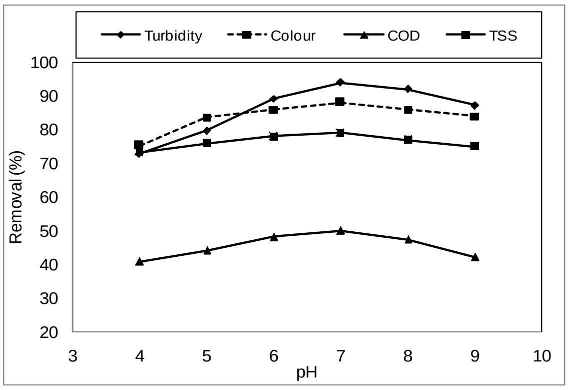

Figure 9 Percentage reduction of turbidity, colour, COD and TSS with varying pH using $FeCl_3$ at OpCD.

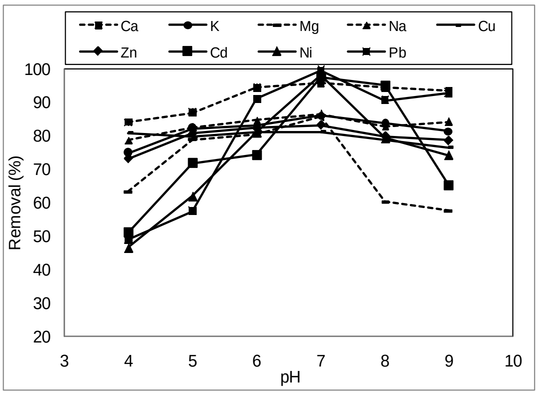

Figure 10 Percentage reduction of Ca, K, Na, Mg, Cu, Zn, Cd, Ni and Pb with varying pH using $FeCl_3$ at OpCD.

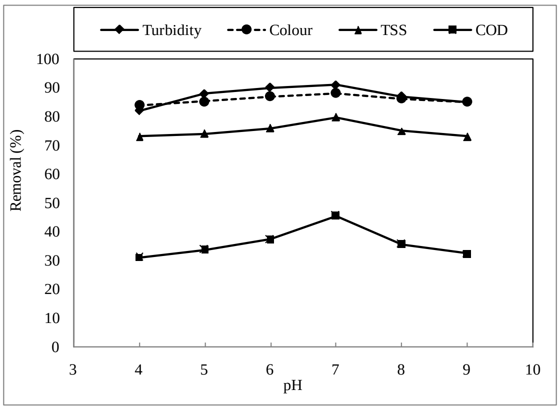

Figure 11 Percentage reduction of turbidity, colour, COD and TSS with varying pH using PAC at OpCD.

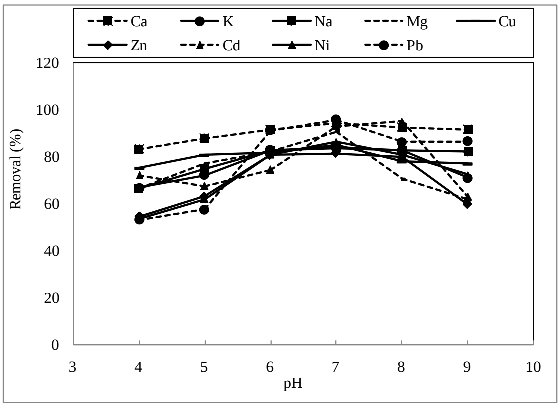

Figure 12 Percentage reduction of Ca, K, Na, Mg, Cu, Zn, Cd, Ni and Pb with varying pH using PAC at OpCD.

3.3.2 Effect of pH on Coagulation with PAC

The reduction of inorganic compounds of turbidity, leachate colour, COD, TSS as well as metal concentrations of Ca, K, Na and Mg and heavy metal of Cu, Zn, Cd, Ni and Pb in leachate using PAC at OpCD of 2.0/500mL as shown in Figures 11 and 12, respectively. Here, it is interesting to note that the removal efficiency was increased in relation to the increasing of pH vales up to the optimum values of pH 7, and then it was decreased. The similar trend was observed by Tatsi et al. (2003) using $FeCl_3$ and PAC at pH 7. Based on Figure 11, it can be concluded that the maximum removal efficiency was found for PAC at pH 7 of OpCD of 2.0g/500mL of turbidity with of 91%, while, the minimum was found for COD with 45%. Based on Figure 12, it can be seen that the maximum reduction for Ca of 94%, while minimum was Na of 84%. Moreover, the reduction for PAC at pH 7 for Cu, Zn, Cd, Ni and Pb was 85, 82, 93, 86 and 96%, respectively. The minimum reduction of heavy metals occurred at pH 4 using PAC and it was 75% for Cu, 55% for Zn, 72% for Cd, 53% for Ni and 53% for Pb.

3.4 Regression Coefficient for Optimum pH

Figures 9 and 10 shows the effect of pH ranging from 4-9 on the percentage reduction of pollutant in leachate using $FeCl_3$ at OpCD of 1.5g/500 mL. However, the good conformity correlations of inorganic compound and metals concentration in leachate against of various pH were investigated by evaluating the regression coefficient (R^2). Here, it was noticed that the regression coefficient (R^2) at varying pH was strongly linear and it was varied for approached and after optimum dosage of pH. The correlations of Cd and Pb were split into two parts is evident in Figure (13a) and Figure (13b). Based on the experimental results, it can be concluded that the regression coefficient for removing of Cd and Pb approached the optimum pH was 0.929 and 0.931 (Figure 13a), whereas, after optimum it was 0.808 and 0.528, respectively (Figure 13b). On the constraints, for the removal of pollutant in leachate PAC at OpCD of 2.0g/500mL, at varying pH of 4-9, was also performed. The experimental result reveals that the regression coefficient for removing of Cu and Ni approached the optimum pH was 0.923 and 0.954 (Figure 14a), whereas, after optimum was 0.836 and 0.979, respectively, (Figure 14b).

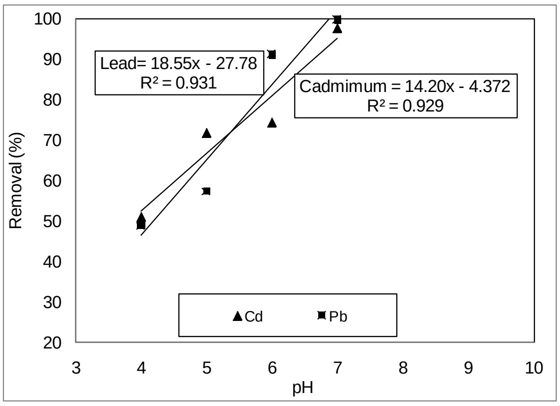

Figure 13a Regression coefficients for pH range 4-7 for removing Cd and Pb using $FeCl_3$ approached the OpCD.

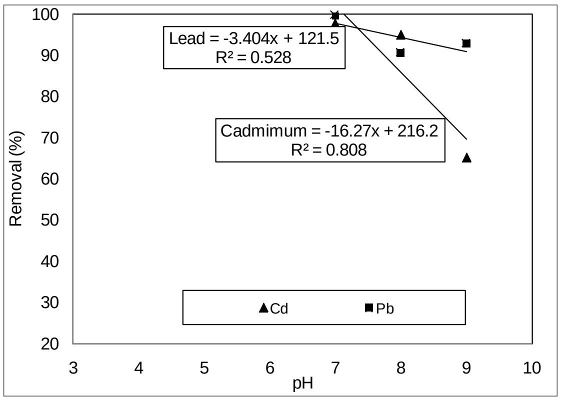

Figure 13b Regression coefficients for pH range 7-9 for removing Cd and Pb using $FeCl_3$ after the OpCD.

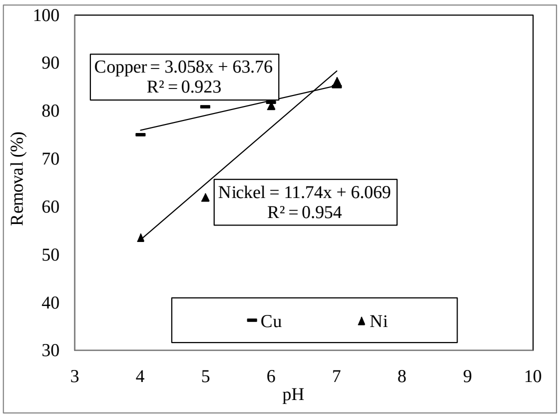

Figure 14a Regression coefficients for pH range 4-7 for removing Cd and Pb using PAC approached the OpCD.

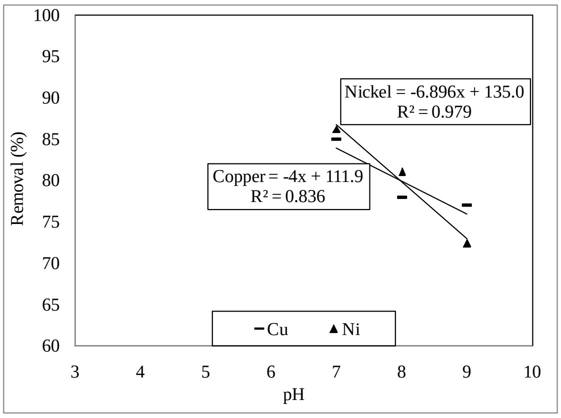

Figure 14b Regression coefficients for pH range 7-9 for removing Cd and Pb using PAC after the OpCD.

4. OPTIMUM RESULTS OF LEACHATE TREATMENT

The optimum results were achieved using $FeCl_3$ and PAC at OpCD of 1.5g/500mL, and 2.0g/500mL, respectively, at optimum pH 7 and mixing speed 110rpm for the removal of pollutants are evident in Table 4 and also in Figure 15. $FeCl_3$ was able to achieve complete removal of Ca, K, Na, Cu, Cd, Ni and Pb at OpCD, while at optimum pH it was able to achieve complete removal of Cd, Ni and Pb. Here, it was noticed that the optimum results shown the addition of $FeCl_3$ and PAC at OpCD to leachate resulted in 95 and 89%, respectively, the reduction of turbidity, and also 93 and 85% for the reduction of Zn, respectively. Based on the observed results it can be concluded that the removal efficiency for most of the pollutants, $FeCl_3$ was more effective than the other coagulants at optimum pH 7 and mixing speed 110rpm.

Table 4 Comparative reduction (%) of pollutant using chemical coagulant at optimum conditions

Coagulant	Optimum Condition	Removal efficiency of leachate pollutant (%)												
		Turbidity	Color	COD	TSS	Ca	K	Na	Mg	Cu	Zn	Cd	Ni	Pb
$FeCl_3$	1.5g/500mL	95	86	50	81	100	100	100	80	100	93	100	100	100
	pH 7	94	88	50	79	96	86	86	86	81	83	100	100	100
PAC	2.0g/500mL	89	86	41	79	93	83	82	86	77	85	72	78	53
	pH 7	91	88	45	80	94	84	84	91	85	82	93	86	96

COD=chemical oxygen demand, TSS=total suspended solid, Ca=calcium, K=potassium, Na=sodium, Mg=magnesium, Cu=copper, Zn=zinc, Cd=cadmium, Ni=nickel and Pb=lead.

Results indicate that the leachate treatment with $FeCl_3$ at OpCD of 1.5g/500ml at pH 7 with mixing speed 110 rpm was able to reduce the Cd, Ni and Pb by 100%, to 0 mg/L which is below the Environmental Quality requirements (Sewage and Industrial Effluent) as well as the maximum discharging standard for landfill leachate from selected countries (USA, Germany, Hong Kong and Malaysia) (Figure 16a). Moreover, treatment with $FeCl_3$ at OpCD of 1.5g/500ml at pH 7 with mixing speed 110 rpm was only able to reduce COD by 50%, to 255mg/L, which is still above the Environmental Quality requirements (Sewage and Industrial Effluent), Bangladesh standards for discharging treated leachate into Inland surface water as well as the maximum discharging standard for landfill leachate from selected countries (USA, Germany, France, Hong Kong, South Korea and Malaysia) (Figure 16b).

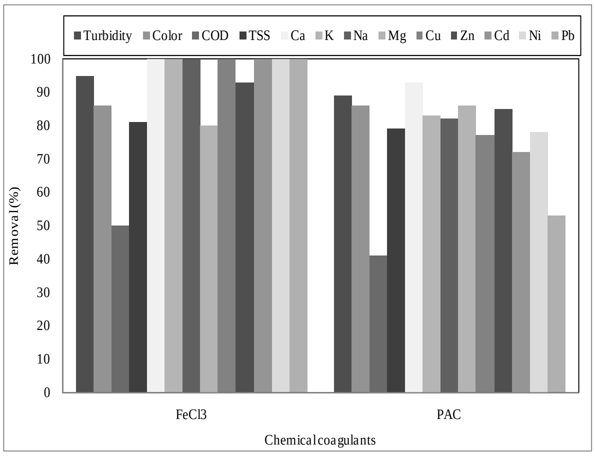

Figure 15 Optimum results of leachate treatment using coagulants.

On the other hand, PAC at 2.0g/500ml, pH7 and mixing speed 110 rpm has an ability to remove the Zn (Figure 17a) by 82%, to 0.11 mg/L, Ni (Figure 17b) by 86, to 0.008 mg/L which is below the Environmental Quality requirements (Sewage and Industrial Effluent) as well as the maximum discharging standard for landfill leachate from selected countries (USA, Germany, Hong Kong and Malaysia). The finding of this study using $FeCl_3$ and PAC for the removal of heavy metals is quite similar to the finding observed by Jayabala (2005) who established in his study that the $FeCl_3$ at 40 mg/L and pH 6 was able to reduce As and Cd to 0.0015 mg/L which is below the EQA requirements. However, in the same study using PAC at 4 mg/L was able to reduce As to 0.409 mg/L at pH 8, which was still above the EQA requirements.

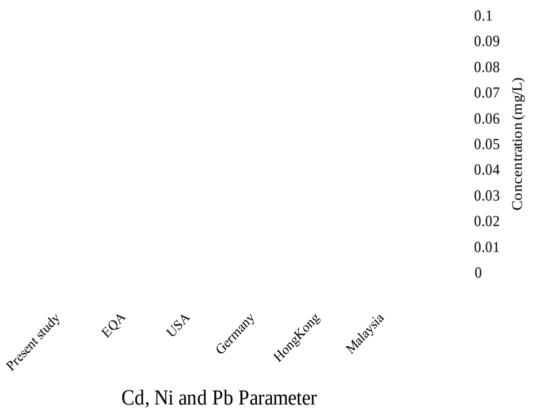

Figure 16a Effect of $FeCl_3$ on the removal of Cd, Ni and Pb at OpCD, pH and mixing speed compared to the standard tools.

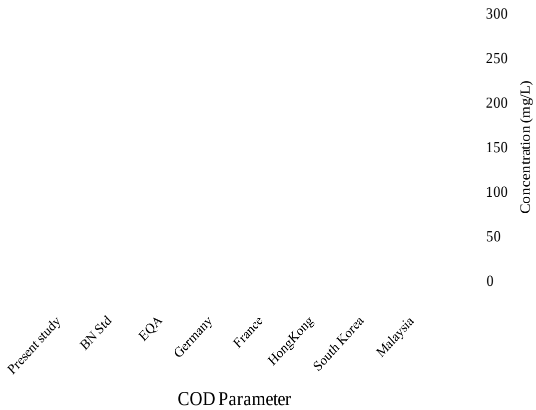

Figure 16b Effect of $FeCl_3$ on the removal of COD at OpCD, pH and mixing speed compared to the standard tools.

Figure 17a Effect of PAC on the removal of Zn in leachate at OpCD, pH and mixing speed compared to the standard tools.

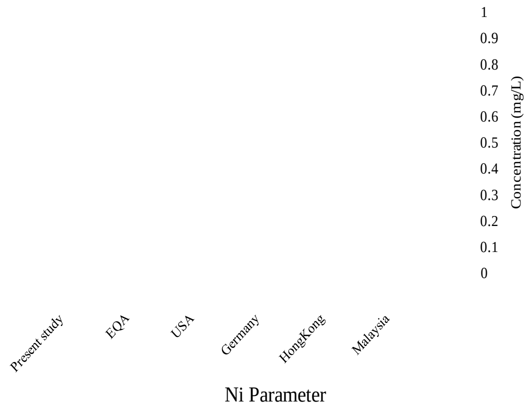

Figure 17b Effect of PAC on the removal of Ni in leachate at OpCD, pH and mixing speed compared to the standard tools.

5. CONCLUSIONS

The application of coagulation-flocculation as a single leachate treatment method for landfill lysimeter leachate was examined in this study. Results showed that the maximum reduction using $FeCl_3$ at OpCD of 1.5g/500mL of Ca, K, Na, Cu, Cd, Ni and Pb was 100%, while for Mg and Zn was 80 and 93%, correspondingly. In contrary, the reduction using PAC at OpCD of 2.0g/500mL for Ca, K, Na, Mg, Zn, Cu, Cd, Ni and Pb was 93, 83, 82, 86, 85, 77, 72, 78 and 53%, respectively. Finally, it can be concluded that $FeCl_3$ was found to be effective as coagulant in leachate treatment, prior to biological treatment because it was able to give more reduction than PAC.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF HEAVY METAL CONTAMINATION & SEDIMENT QUALITY IN THE TURAG RIVER, BANGLADESH

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ABSTRACT

The aim of the study is to investigate the extent of pollution of sediments of the river. We have analyzed the portion of sediment passing through the#200 sieve to estimate the levels of selected heavy metals (Pb, Cd, Cr, Cu and Zn) &spatial distribution. The metal contents were ranging over following intervals: Cd: 0-0.8 mg/kg dry weight; Cr: 32-75.5 mg/kg dry weight; Cu: 46.3-60 mg/kg dry weight; Zn: 94.6-119.6 mg/kg dry weight; Pb: 28.3-36.4 mg/kg dry weight. To assess metal contamination in sediment, US environmental Protection Agency's (USEPA) Guidelines were applied. The study showed that Cr, Cu, and Zn are the major pollutants falling in the criteria of moderately to highly polluted range, while Cd & Pb are the minor pollutants. The heavy metals were analyzed statistically by using principal component analysis (PCA) and hierarchical cluster analysis (HCA) methods. Strong positive correlation's between the contaminants of Cr and Zn(r=0.701), Cu and Pb(r=0.559), Cu and Zn(r=0.325); Zn and Pb(r=0.365), indicating the existence of common origin of these metals in sediment. Toxicity characteristics leaching procedure (TCLP) test (US EPA 1311) for sediment samples have been performed for heavy metals (Pb, Cd, Cr, Cu and Zn) to determine the readily toxicity level of heavy metals. The metal concentrations are well below the regulated level as per US EPA. The Metal contamination in the sediments were also evaluated by applying Index of geo-accumulation (Igeo), contamination factor (C_f).The sites were unpolluted to moderately polluted. As some of the heavy metals' concentrations are higher than the recommended value, which suggest that the Turag is to a certain extent a heavy metal polluted river and the water, sediment and fish are not completely safe for health.

Keywords: *Heavy metal, Principal component analysis, Cluster analysis, Geo-accumulation index, Contamination factor.*

1. INTRODUCTION

The Turag river has become one of the most polluted river in Bangladesh due to rapid urbanization and industrial development during last decade.152 industries wreaking havoc in Turag River, Savar, Dhaka, Bangladesh. Villagers along one polluted Turag moving to safer places. Indiscriminate discharge of liquid waste by the industries in and around Konabari industrial zone has ruined a large part of the Turag River and Baimailjheel, causing immense suffering to residents living on the banks. Industrial liquid waste and black smog created by brick kilns have doubled the suffering and compelled many to move their houses. Locals say brick kilns have left their fruit trees unproductive, while industrial waste exterminated fish in the river and nearby water bodies, increased mosquitoes, and made croplands infertile(The Daily Star, 12th September, 2011). Heavy metals contamination in aquatic environment is of critical concern, due to toxicity of metals and their accumulation in aquatic habitats. Trace metals in contrast to most pollutants, not biodegradable, and they undergo a global ecological cycle in which natural water are the main pathways. Of the chemical pollutants heavy metal being non-biodegradable, they can be concentrated along food chain, producing their toxic effect at points after far removed from the source of pollution (Tilzer and Khondoker, 1993). Exposure to heavy metals has linked to several human diseases such as development retardation and malformation, kidney damage, cancer, abortion, effect on intelligence and behavior, and even death in some cases of exposure to very high concentrations. The aim of this study is to assess the level of heavy metal concentrations and sediment quality in surface sediments of the Turag River and to explore the natural and anthropogenic input of heavy metals and to assess the pollution status on the area and to highlight relationships among metals pollution.

2. METHODS AND MATERIALS:

2.1 Study Area:

The study area (Turag River) is located beside Dhaka city, capital of Bangladesh. It is the upper tributary of the Buriganga, a major river in Bangladesh. The Turag originates from the Bangshithe Buriganga at Mirpur in Dhaka District (Wikipedia). The river is increasingly being polluted of the city's industrial units and sewerage lines dumping huge volumes of toxic wastes which contain lots of heavy metal into it day and night.

2.2 Field Sampling:

In order to achieve he research objective, samples were collected from five different locations of the Turag River. (Latitude and Longitude for each site are illustrated in Table 1). Criterias for selection of sampling station were based on the locations of industrial units and land use pattern to quantify heavy metal concentration. The Samples were taken from the Turag River during the dredging of Turag bed sludge using washed plastic container to avoid unpredictable changes in characteristic of samples.

Table 1: Location of sample collection

Designation	Location	Latitude	Longitude
T-1	Tongi Bridge	23^0 52' 54.58"N	90^0 24' 03.20"E
T-2	Ijtema Field	23^0 53' 15.54"N	90^0 23' 32.80"E
T-3	Kamarpara Bridge	23^0 53' 29"N	90^0 23' 24.30"E
T-4	Taltola Bridge	23^0 52' 45" N	90^0 20' 45.00" E
T-5	Beribadh	23^0 53' 45.77"N	90^0 22' 16.16"E

2.3 Laboratory Works:

After collection, sediment samples were dried in a vacuum oven at 105^0C until constant weight, lightly ground in an agate mortar for homogenization and prepared 500 ml solution. Finally, five heavy metals (Pb, Cd, Cr, Cu and Zn) concentration were determined in the environmental engineering laboratory, BUET by using atomic absorption spectrophotometer.

3. RESULTS AND DISCUSSIONS

3.1 Heavy Metal Concentrations

The metal concentrations for sediment passing through the #200 sieve are shown inTable2. Metal contents were ranging over following intervals: Pb: 28.3-36.4 mg/kg; Cd: 0-0.8 mg/kg; Cr: 32-75.5 mg/kg; Cu: 46.3-60 mg/kg; Zn: 94.6-119.6 mg/kg dry weights. Mean concentration of the metals were:Pb:32.4 mg/kg;Cd:0.2 mg/kg;Cr:44.3 mg/kg;Cu:50.7 mg/kg; Zn: 144.5 mg/kg, allowing to allow the metals from higher to lower mean content in this area as: Zn>Cu>Cr>Pb>Cd.

Table2: The test result (mg/kg dry weight) for the sediment sample (passing #200 sieve) of the Turag River:

Location	**Lead (Pb)**	**Cadmium (Cd)**	**Chromium (Cr)**	**Copper (Cu)**	**Zinc (Zn)**
T-1	36.40	0.10	36.00	60.00	179.30
T-2	34.40	0.10	33.50	46.30	113.80
T-3	30.40	0.00	75.50	46.40	190.10
T-4	28.30	0.40	32.00	50.00	94.60
T-5	34.40	0.80	38.10	49.30	119.60
Mean	32.78	0.28	43.02	50.40	139.48
Max	36.40	0.80	75.50	60.00	190.10
Min	28.30	0.00	32.00	46.30	94.60
SD	3.32	0.33	18.31	5.62	42.48

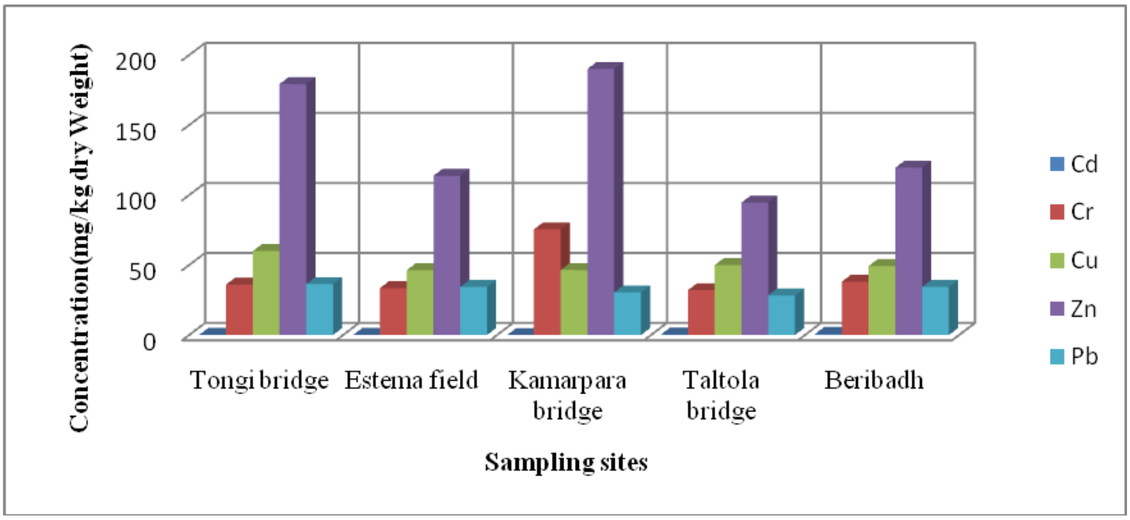

Fig1: Spatial variation in the concentrations of Pb, Cu, Cd, Cr, Zn for the sediment sample(passing #200 sieve) of the Turag River:

Pearson's correlation coefficient matrix among the selected heavy metals is presented in Table.3Significant correlations between the contaminants of Cr and Zn(r= 0.701), Cu and Pb(r=0.559), Pb and Zn(r=0.365) could indicate the same or similar source input.

Table3: Pearson's Correlation matrix between heavy metals in sediment samples (passing #200 sieve) from the Turag River:

	Cd	**Cr**	**Cu**	**Zn**	**Pb**
Cd	1				
Cr	-0.626	1			
Cu	0.067	-0.384	1		
Zn	-0.794	**0.701**	0.325	1	
Pb	-0.535	-0.289	**0.559**	0.365	1

Further confirmation of this hypothesis was secured through multivariate method so f statistical analysis (Hair et al. 1988). To this effect to multivariate techniques were applied: Principal Component Analysis (PCA) and Cluster analysis (CA).

PCA using Varimax normalized rotation was conducted for common source identification. The variables are correlated with two principle components in which 78.73% of the total variance in the data is found. The rotated Principal Component Loadings are given Table 3.

The first component with 35.75% of variance comprises Zn and Cr (bold figures). This association strongly suggests that these variables have a similar source. The second component (PC2) contributes Pb and Cu at 42.98% variance with high loadings.

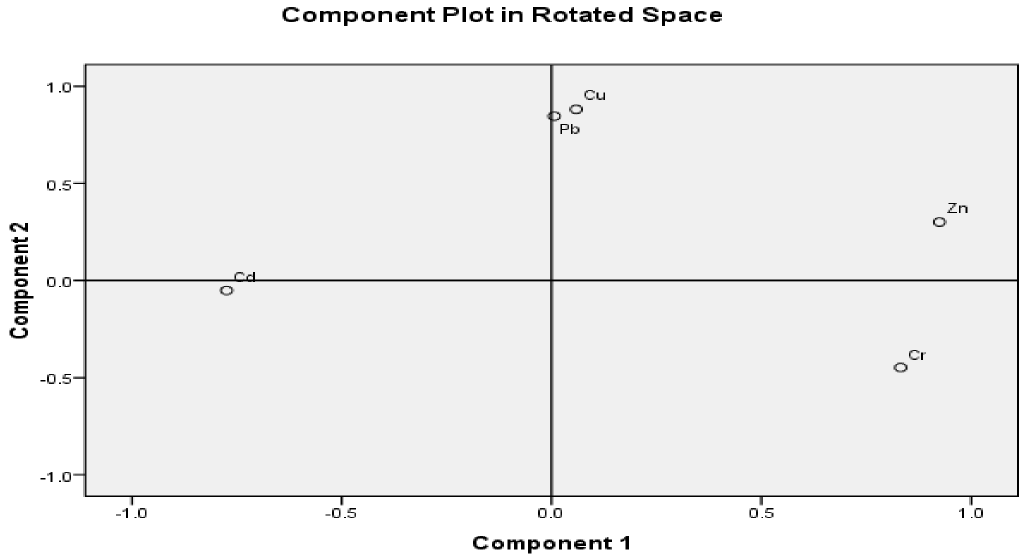

Figure 2: Principle Component plot in a rotated space

* * * * * * * * H I E R A R C H I C A L C L U S T E R A N A L Y S I S * * * * * * *

Dendrogram using Average Linkage (Between Groups)

Rescaled Distance Cluster Combine

```
  C A S E    0       5      10      15      20      25
 Label    Num  +---------+---------+---------+---------+---------+

 Cu        3
 Pb        5
 Cr        2
 Cd        1

      Zn        4
```

Figure 3: Dendogram of Cluster Analysis

The corresponding cluster analysis Dendogram (shown in Fig 3) showed good agreement with PC2 (relation between Cu-Pb) but not showed good agreement between Cr-Zn.

3.2 Physical Characteristics:

Sieve analysis result of sediment sample is presented in the table 4 .Average in all sites, 1.75% materials retain on #4 and above sieve and 25.34 % materials passing through the #200 sieve whereas 72.90% materials retain on #8 to #200 sieves that indicate the sediment size of the Turag River ranges over medium to fine size and it contains a significant amount of fine particle.

Table4. Sieve Analysis Result of The Sediment Sample of the Turag River

Sieve Analysis result of Sediment Sample					
Sieve Size(ASTM)	**Station 1**	**Station 2**	**Station 3**	**Station 4**	**Station 5**
	% Materials retains (gms)	**% Materials retains (gms)**	**% Materials retains (gms)**	**% Materials retains (gms)**	**% Materials retains (gms)**
1/4”	0.75	1.06	1.17	0.47	0
No 4	1.38	1.38	1.44	0.76	0.37
No 8	9.07	6.96	7.02	9.23	4.96
No 16	15.14	10.26	11.59	16.89	11.36
No 30	10.35	6.82	8.45	14.03	10.6

No 40	4.06	4.12	3.77	7.18	4.51
No 50	4.95	5.25	5.41	7.03	5.61
No 100	17.5	22.59	17.56	13.64	17.52
No 200	13.95	17.84	15.14	9.82	14.33
PAN	22.86	23.71	28.44	20.95	30.74
TOTAL	100	100	100	100	100
% Coarse	2.13	2.44	2.61	1.23	0.37
% Medium	75.02	73.84	68.94	77082	68.89
% Fine	22.86	23.71	28.44	20.95	30.74

4. Assessment of Heavy Metal Contamination

4.1 Assessment According to United States Environmental Protection Agency (USEPA)

The chemical contaminations in the sediments were evaluated by comparison with the sediment quality guideline proposed by USEPA. These criteria are shown in Table 5. Present study shows that all the sites are not polluted for Pb and Cd while for Cr, sites 1, 2 ,4and 5 are moderately polluted except the site 3 is heavily polluted. For Cu site 1 is heavily polluted while others are moderately polluted. For Zn, all the sites are in moderately polluted condition.

Table5. EPA Guidelines for Sediments (mg/kg Dry Weights)

Metal	Not polluted	Moderately polluted	Heavily polluted	Present study
Pb	<40	40-60	>60	28.30-36.40
Cd	---	---	>6	0-0.80
Cr	<25	25-75	>75	32-75.50
Cu	<25	25-50	>50	46.30-60
Zn	<90	90-200	>200	94.60-190.10

4.2 Assessment according to geo-accumulation Index (I_{geo})

A common criterion to evaluate the heavy metal pollution in sediments is the geo-accumulation index (Igeo), which was originally defined by Muller (1979) to determine metals contamination in sediments, by comparing current
concentrations with pre-industrial levels and can be calculated by the following equation (Muller 1979)

$$I_{geo}=log_2\,[C_n\,/1.5\,B_n] \qquad (1)$$

Where, C_n is the concentration of element 'n' and Bn is the geochemical background value [In this study, consider
Bn=world surface rock average given by Martin and Meybeck (1979)]. The factor 1.5 is incorporated in the relationship to account for possible variation in background data due to lithogenic effect. The geo-accumulation index (Igeo) scale consists of seven grades (0-6) ranging from unpolluted to highly pollute (shown in Table 6). According to the Muller scale, the calculated results of *I*geo values (Shown in Table 7) indicate, for Pb sediment quality be considered as unpolluted (*I*geo≤0) for station No 4 while other stations considered from unpolluted to moderately polluted. For Cd sediment quality was found to be unpolluted for station No.1, 2, 3 while station no 4 and 5 were considered from unpolluted to moderately polluted and moderately polluted. For Cr ,Cu and Zn,all the sites had unpolluted sediment quality(except station No 1,4 and5 for Cu I_{geo} values indicate from unpolluted to moderately polluted sediment quality).Higher Igeo values are showed for Station No 1 for Pb and Cu while for Cd Station No 5 has the highest value. On the basis of the mean values of *I*geo, sediments are enriched for metals in the following order: Cd >Cu > Pb > Zn > Cr.

Table6. Muller's Classification for the Geo-accumulation Index

Igeo Value	Class	Sediment Quality
≤0	0	Unpolluted
0-1	1	From unpolluted to moderately polluted
1-2	2	Moderately polluted
2-3	3	Moderately polluted
3-4	4	Strongly polluted
4-5	5	From strongly to extremely polluted
>6	6	Extremely polluted

Table7: Geo-accumulation index (I_{geo}) at different sampling station in Turag River by Muller (1979)

Location	Pb	Cd	Cr	Cu	Zn
T-1	0.28	-1.58	-2.01	0.32	-0.11
T-2	0.20	-1.58	-2.12	-0.05	-0.77
T-3	0.02	-	-0.95	-0.05	-0.03
T-4	-0.08	0.42	-2.18	0.06	-1.03
T-5	0.20	1.42	-1.93	0.04	-0.69
Mean	0.124	-0.26	-1.838	0.064	-0.526

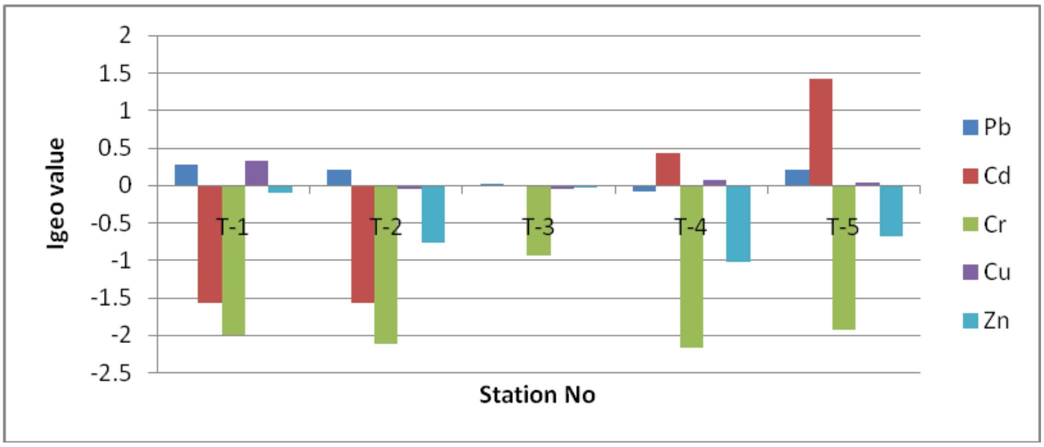

Fig 4: Variation of I_{geo} value at different sampling station of the Turag River for different Heavy metals

4.3 Assessment according to Contamination Factor

The contamination factor (C_f) or enrichment ratio(ER) and the degree of contamination (Cd) are used to determine the contamination status of sediment in the present study. C_f values for describing the contaminations level are shown in table 8 .The contamination factor are calculated according to the eq.2 and the degree of contamination (Cd) was defined as the sum of all contamination factors. Calculated contamination factor (Cf) and degree of contamination (Cd) for this study is shown in the table 9.

$$C_f = \text{Measured concentration/ Background concentration} \quad (2)$$

Where, Background value of the metal= world surface rock average given by Martin and Meybeck (1979).
In the present study, maximum contamination factor 4 was found indicating Considerable contamination in the station No. 5 where the degree of contamination is 8.58.Contamination factor, $1 \leq C_f < 3$(indicate moderate contamination) are found in station No 4 for Cd, in all station for Pb and Cu and in station No1 and 3 for Zn. Rest of the contamination factors were (Cf) <1 for all tested heavy metals. The mean value of the Cf is found:

Pb: 1.64 (moderate contamination); Cd: 1.4(moderate contamination); Cr: 0.44 (low contamination); Cu: 1.58 (moderate contamination); Zn: 1.08 (moderate contamination) .On the basis of the mean values of C_f, sediments are enriched for metals in the following order: Pb> Cu > Cd >Zn > Cr.

Table8. Contamination Factor and Level of Contamination (HAKANSON 1980)

Contamination Factor (Cf)	**Level of Contamination**
Cf < 1	Low contamination
1≤ Cf < 3	Moderate contamination
3≤ Cf < 6	Considerable contamination
Cf > 6	Very high contamination

Table9. Contamination Factor Values for the Sediment Sample of the Turag River

Sample Location	**Contamination factor (C_f) or Enrichment factor(ER)**					**Degree of contamination**
	Pb	Cd	Cr	Cu	Zn	
T-1	1.82	0.50	0.37	1.88	1.39	5.96
T-2	1.72	0.50	0.35	1.45	0.88	4.89
T-3	1.52	0.00	0.78	1.45	1.47	5.22
T-4	1.42	2.00	0.33	1.56	0.73	6.04
T-5	1.72	4.00	0.39	1.54	0.93	8.58
Mean	1.64	1.40	0.44	1.58	1.08	6.14

4.4 Assessment According to Toxicity Characteristics Leaching Procedure (TCLP)

Toxicity Characteristics Leaching Procedure (TCLP) is a very important tool for assessing readily contaminated heavy metal for sediment samples. In this study, heavy metal concentrations from leachate of Turag River Sediments were tested in the laboratory and pollution levels of leachate were assessed with comparison of EPA standard. Results of the TCLP test are presented in the Table 10. For all the sites, concentrations of heavy metal in leachate are very low than the permissible EPA standard. That indicate regarding the readily toxicity pollution by heavy metal, Turag River Sediment condition is not the severe state.

Table 10: TCLP test result (mg/l) for the sediment sample of the Turag River

Station No.	**Pb**	**Cd**	**Cr**	**Cu**	**Zn**
1	0.000	0.016	0.000	0.120	0.164
2	0.017	0.009	0.017	0.184	2.327
3	0.000	0.003	0.000	0.178	3.733
4	0.000	0.000	0.000	0.132	0.608
5	0.009	0.001	0.009	0.121	1.949
Mean	0.0034	0.0058	0.0018	0.147	1.756
Max	0.017	0.016	0.017	0.184	3.733
Min	0.000	0.000	0.000	0.120	0.608
SD	0.0076	0.0067	0.004	0.0315	1.425
EPA Standard	5	1	5		

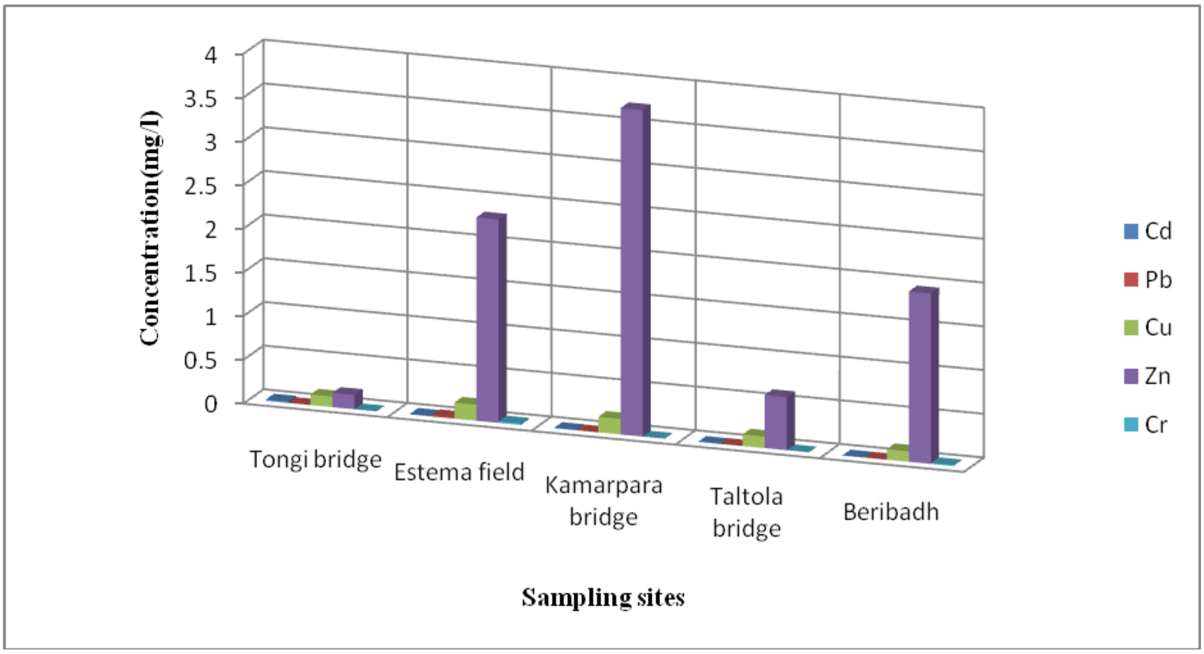

Fig 5: Spatial variation in the concentrations of Pb, Cu, Cd, Cr, Zn for the sediment sample(TCLP) of the Turag River:

5. CONCLUSIONS:

Identification and quantification of heavy metal sources, as well as fate of those heavy metals, are important environmental scientific issues. The present study presents valuable information about heavy metal content and physical characteristics of sediment from different sampling stations of Turag River.

Multivariate analyses, including the correlation matrix analysis and factor analysis used in this study provide an important tool for better understanding the complex dynamics of pollutants. The correlation analysis of concentrations data shows weak positive and negative correlations among Cd-Cu(r=0.067), Cr-Pb(r= -0.289) indicating that this metals have complicating geochemical behaviors. Strong positive linear correlations were found between Cr, Zn, Cu and Pb, indicating the existence of common origin of this metal in sediment. PCA summarizes (reduces) the data set into two major components representing the different sources of the elements. According to geo-accumulation index (I_{geo}) and contamination factor (C_f), the sediment samples are moderate to low polluted condition. TCLP test showed that there is no considerable risk of contaminations from leachate of Turag River Sediments.

Considering all assessment criteria Cu, Cr and Zn are responsible for considerable or significant amount of heavy metal contamination while contamination of Pb and Cd are in a lower level.

ACKNOWLEDGEMENT:

The authors realized to thanks to the BUET Authority and the staff of the BUET Environmental Engineering Laboratory, for providing all kinds of assistance to conduct this research work.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EXPERIMENTATION OF SELF TURNING REACTOR (STR) COMPOSTING SYSTEM IN THAILAND

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ABSTRACT

The This paper presents the effort of the authors to develop pilot-scale static reactor that mimics the central core of aerated static composting system, which combines the knowledge of gravity mixing technology and vertical reactor composting concept together with turning unit. The performance of this novel composting system was assessed by necessary pilot-scale experimentation of decomposition of substrates considering different state of variables of air flow and substrates. Comprehensive temperatures monitoring were done which identify the mechanisms of decomposition in presence of air. It is apparent from the results of experimentation that the central core has minimal radial spatial variation in the state variables. And finally, the quality of compost was addressed by evaluating maturity index and the test results of pathogens those may present in compost, which revealed that the final compost is safe to apply in agricultural use. Again, experimentation of recycling of compost might contribute in decision making of waste management system to adopt sustainable composting technology, especially in large scale operation.

Keywords: *Self turning reactor, pilot-scale, temperature, aeration, maturity index, recycle compost*

1. INTRODUCTION

There has been an increased interest in using biological processes to address environmental concerns such as solid waste disposal, hazardous waste remediation and treatment of contaminated water. One biological process which has received considerable attention for the remediation of organic solid waste is controlled aerobic decomposition (Kaneko et al., 2001), more commonly called composting. The commercial success of composting is dependent on successful odour control and stability and quality of the final product (Al-Muyeed, 2007). The compost product can be beneficial, supplying nutrients for plant growth, organic matter for soil improvement and agents for plant disease suppression. The aim of the stabilization is to produce a material that does not putrefy, self-heat, deplete oxygen, produce odors or attract vermin. These goals are met by managing oxygen, moisture and energy transport within the process (Rahman and Al-Muyeed, 2005). Poor oxygen transfer inhibits the growth of aerobic microorganisms and enhances the growth of anaerobes which have been reported to increase odour production low moisture content and high temperature inhibit microbial activity, which may result in an unstable product and reduce product quality) Addressing these issues involves developing a better understanding of the interactions between the process degradation kinetics and the mechanisms of heat and mass transport (Maekawa et al., 2003).

A general approach used to gain a better understanding of these interactions is to perform degradation studies on pilot-scale levels which can be implemented in large scale after confirming sustainability by trials operations of degradation. Large scale composting takes place through one of two basic principles - open air composting or closed or reactor composting (VanderGheynst et al., 1997b). With reactor composting the process can be controlled so the breaking down goes quickly and is more complete and the emissions are possible to measure and clean. The reactor based composting controls mass and energy exchanges between the organic matrix and the gas stream often are revealed through gradients in temperature, oxygen and water content. Therefore, to understand the coupling between degradation kinetics and heat and mass transport within an aerated static bed process, reactors must be designed to allow analysis of gradients across the reactor bed. Again, a program of quality control can guarantee the process and the high quality of the compost (Al-Muyeed, 2005). The finished compost can be returned to producing soil in a controlled way. Hence, this paper aims to describe a reactor composting technology comprising novel engineering approach of mixing and turning technology. This innovative concept of composting unit is termed as Self-Turning Reactor (STR) that consists of a reactor component for composting and the turning component consuming gravity energy only as a combined unit (Bongochgetsakul, 2005). It has high expandability in order to support the large-scale composting as demanded from place to place while maintaining enhanced efficiency and utilizing less space (Bongochgetsakul et al., 2007). Moreover, this report presents the effort of the authors to develop pilot-scale static reactors that mimic the central core of aerated static composting system. It is apparent that the central core has minimal radial spatial variation in the state variables; therefore, all spatial variation in the state variables considers mostly one dimensional variation along vertical axis of the reactor. In addition, general process performance results, such as maximum reactor temperatures and moisture variations, are reported for two different substrates for high and low air flow for unique moisture content. An assessment of the temporal and spatial variations in temperature, moisture content is also presented in this paper.

2. METHODOLOGY

Design concepts of STR composting system

The design concept of STR composting system allows material to flow vertically downward by gravity force. The first glance of the developed concept is shown in Fig. 1. (left). Mixture of organic waste and compost material enter the system from the top. The mixture is then subjected to a number of composting-turning cycles, which results a compost product from the bottom of the stack. Each STR unit offers two primary functions which are composting the input material in batch manner, and aerating, restructuring, releasing excess water and inhibitor from the composted mass. These two functions are performed in the reactor and turning unit respectively (Bongochgetsakul et al., 2007). Compost mixture is elevated to charge its potential energy, and is then put into the topmost reactor of the STR unit. The material will be retained in the reactor component for some days for composting and then be released to the self-turning unit below for turning, within a few second, before sending to the next STR unit for the next cycle. This cycle will be repeated until well-degraded organic biomass as a compost product is obtained, which is presented in the Fig. 1(lower middle). This STR unit can be extended vertically to increase number of composting- turning cycles, and can be expanded horizontally to increase overall treatment capacity in parallel, Fig. 1 (right). This design concept ensures the extendibility of the system scale to be as large as demanded. The STR units are connected vertically, which effectively utilizes land space especially where the location has limitation on land acquisition or budget. The Self-Turning unit utilizes gravitational energy in mixing and turning materials, which has major advantages over mechanical turning processes in terms of energy and time consumption. Again, this turning unit does not occupy any additional land space which other mechanical processes usually need in turning process.

Fig. 1. STR technology concept: Schematic overall concept (left), Composting and turning cycles scheme (lower middle), Expandability of the units when connected vertically and/or horizontally (right)

Turning of compost mass during decomposition process has many important functions, e.g. supplying oxygen, redistributing the waste products generated by degradation reaction (Ahn et al., 2008), redistributing some anaerobic zone, releasing excess water content, etc. It enhances and reactivates degradation processes by recharging microbes to continue degradation in post turning period. Turning and mixing are very akin in processes; both are related to agitating. However, mixing process needs thoroughly agitating in order to obtain higher degree of uniformity of the output material. The turning method that is presented in this paper is also able to mix the raw materials and wastes together or to adjust water content of the materials. (Matabee et al., 1998) has proposed a high performance continuous concrete mixing technology that utilizes gravitational driving force in mixing of the mortar and aggregates, so called MY-BOX. It was designed and developed based on kneading and lapping processes as can be seen in making of Japanese wheat pasta (Udon). MY-BOX is a box with two openings on the opposite side so that material can pass through it. The idea of the MY-BOX mixing technology was incorporated to develop both mixing and turning units of the STR composting system. As mentioned previously, the mixing method by utilizing MY-BOX requires no powered mechanical agitation but only gravity force (Bongochgetsakul, 2005). This mixing method requires very short time, few seconds, to accomplish the mixing process. A number of modifications were made on the original MY-BOX design to apply for the compost material while retaining the main concept of the mixing mechanism. Shape, size, inclined angle of the unit has been altered to be suitable for compost mixing and turning. It was found from trial experiments (Bongochgetsakul et al., 2007) that the inclined angle (of the slope paths) of 60 degree is the most suitable for compost material under normal range of water contents (20%-65%). The new mixing/turning unit is named as BioMY-BOX; its design and mixing mechanism are shown in Fig. 2. A unit of BioMY-BOX has two inlets and two outlets for entering of two different types of material. A material enters the box, impacts with inclined slope and then separate into two streams by a directional fin; this process is called kneading. Two kneaded different materials flowing into different inlets are forced to change its flow direction clockwise or counter clockwise while continue to impact with the inner wall, this process is called lapping. The layered material flows from the upper mixing unit to the consequential inlet of the lower one in the manner which the layered material is halved and equally enters the lower inlet. This processes repeats until the materials passed through all mixing units. BioMY-BOX contains two types, clockwise and counterclockwise, according to flowing direction. Each mixing unit has to be connected in alternated series of clockwise (CW) following of counter-clockwise (CCW) and then clockwise (CW) to ensure continuous series of kneading and lapping processes. Material passing through a mixing unit resulted in one kneading followed by lapping step. A series with n mixing units resulted in 2^{n-1} layers of mixed material, as shown in Fig. 2 (left). As a number of mixing unit n increased, degree of homogeneity of the output mixture increased. However, total height of mixing units can be consequently increased. Again, for the turning unit, the BioMY-BOX unit can be extended horizontally to fit beneath the reactor to ensure the fall of mass during turning (Bongochgetsakul and Ishida, 2008); Fig. 2 (right). The width of each unit of BioMY-BOX can also be flexibly chosen to exactly fit beneath the reactor itself.

Fig. 2. Mixing unit (left) and Turning unit (right) of BioMY-BOX

STR composting system design and construction

The pilot plant of the STR unit as well as its system components was constructed in Thammasat University, Thailand. This prototype system is designed with one and a-half (1.5) STR units. System design and operation flowcharts of the prototype technology are shown in Fig. 3. The STR unit consists of two reactors and one BioMY-BOX self-turning unit (9×2 array) connected vertically. The reactor has inner dimension of $1.3m \times 1.3m \times 1m$(made by 10mm smooth-lined, corrugated, high-density polyethylene panel). It is found that this type of high-density polyethylene panel is efficient in minimizing radial temperature gradient (VanderGheynst et al., 1997a). Five strips doors that can rotate simultaneously in the same direction were installed at the bottom of each reactor to allow the composted mass to pass next to the lower unit. The most important key on using the rotating type of doors is that the speed of material flow passing through the self-turning unit can be controlled; it prevents sudden clogging of the material passage. Compost mixture is transported to the upper reactor and BioMY-BOX mixer hopper by electrical winch. Air is supplied to each reactor through vertical air dispensers, which it has ability to supply compressed air to the reactor thoroughly to minimize the anaerobic zone (Alkoaik and Ghaly, 2006; Bari and Koenig, 2001; Ekinci et al., 2006; Laguerre et al., 2008). The dispensers supply compressed air from 1.5mm holes in interval of 250mm (helix) along length of 15mm diameter acrylic pipe, which minimize clogging of the air nozzles, which it can reduce time and cost of system maintenance. Lower end of the dispenser is capped, and the upper end is attached with one-touch connector for 10mm nylon pipe for supplying compressed air to the dispenser. Nine air dispensers are vertically attached with 390mm interval inside the reactor. Pressurized air from air compressor was regulated by a pressure regulator. Odorous gas generated during composting was collected by plastic hose and then was sucked by a blower to deodorizer (Biofilter with compost material), (Somda et al., 2002). Odor leakage from the reactors into the plant was also treated by the deodorizer, which is drawn through a wired aluminum hose connecting to the main odor stream at hose junction. Composted mass from the first reactor is released to the self-turning units below. Immediately after passing through the self-turning unit, the compost mass flows into the second reactor by gravity. After composting in the second reactor, the composted mass is then released to the collector below for determination whether it is necessary to repeat composting cycle or not. Odor from blower, which is collected from the reactors and inside the plant, is humidified by the air humidifier before sending to the main deodorizer (Fig. 4). The deodorizing bed is made up of the mixture of cow manure, rice hull and saw dust with the same proportion by weight. Air containing odorous substances will be biologically converted to non-odorous substances (Chen et al., 2005; Haubrichs and Widmann, 2006; Kim and Deshusses, 2008; Nikiema et al., 2005; Zilli et al., 2005) and then be released to atmosphere. Water and compressed air are supplied to the nozzle head in the air humidifier by small water pump and air compressor, respectively.

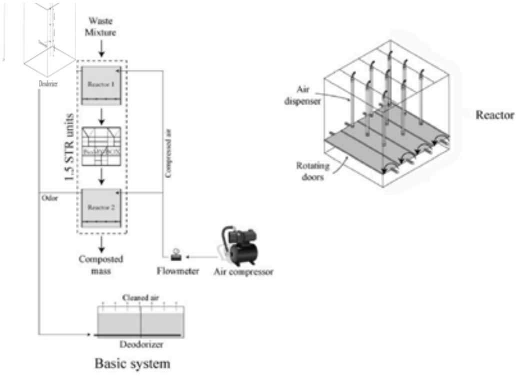

Fig. 3. Schematic diagram of STR composting technology

Fig. 4. Biofilter system

Experimentation of composting in STR unit

Materials preparation

Two substrates were used in the pilot-scale studies. One was synthetic food waste (SFW) which consisted of dry dog food (Pro puppy food, Pro-Pet Inc., Thailand) and the other was residual food scrap (FS) collected from the Thammasat University, Rangsit campus. Dog food (DF) was chosen because it has a fibre, fat and protein content similar to food waste or garbage and because it's physical composition is fairly uniform from batch to batch (VanderGheynst et al., 1997b). Hard wood chips mixed with available garden waste were added to both substrates as a bulking agent and as a carbon source. Prior to experimentation, the chips were size-reduced through a 12.7 mm screener manually (VanderGheynst et al., 1997b). In addition, cow manure is added as seeding material of degradation process (Farfel et al., 2005; Hadas et al., 2004). In order to control 80% porosity of the amendment mixture, saw dust was additionally added in the mix. Physical properties for the DF, FS and amendment are given in Table 1. The carbon to nitrogen dry mass ratios (C:N) and moisture contents of the experimental SFW and FS mixtures are given in Table 2. The amendment to substrate weight ratio (dry) was kept 3.0 for both types of substrates. The final moisture content of the mixture of compost mass was controlled to keep as 60% by weight.

Air flow regulation

Compressed air was used to aerate the reactors as described by (VanderGheynst et al., 1997b). The initial line pressure of the compressed air, 0.6-0.8MPa, was reduced using a pressure regulator (BN-3RT5-8, Nihon. Seiki, Japan). The flow rate of the pressure- reduced air was controlled using precision valve-flow meters (Kofloc, Japan).Then the air was supplied to the pilot-scale reactors via perforated air dispensers.

Table 1. Physical properties of raw materials

Raw materials	Carbon (% of dry solids)	Nitrogen (% of dry solids)	Moisture (% of wet solids)	Volatile solids (% of dry solids)
Dog food	64.6	4.3	9.0	90.5
Food scrap	36.3	3.1	80.0-87.5	55.2
Garden waste	30.2	0.8	25.5	95.5
Saw dust	25.5	0.5	9.5	90.0
Cow manure	55.5	8.5	15.5	90.0

Table 2. Mixing ratio of amendment to substrate and initial moisture content

Mixture	Mixing ratio (dry weight basis)	Initial moisture content (%wet basis)
Cow manure, garden waste and saw dust	1:1:1	40
DF and Amendment	3:1	60
FS and Amendment	3:1	60

Data measurement

Temperature was measured using copper, constantan thermocouple wire T-type (PP-24, Omega Engineering), which has resolution of 0.1 °C. The tip of each thermocouple was spot welded subsequently threaded to ensure proper measurement of data(Lau et al., 1992). The location of thermocouple is placed in the middle of the height of the reactor and at 25 cm from the top of reactor at centre, edge and corner of the reactor (same elevation), which ensures continuous measurement of temperature (Fig. 3: right).

Moisture content of the compost sample was measured using moisture balance (MOC-120H, Shimadzu, Japan). 5 gm of three samples of compost was taken from the reactor's different location at the center periodically and made the average sample by mixing all the three samples. A core sampler 2.5 cm in diameter and 76 cm in length was used to remove samples from the reactor (VanderGheynst et al., 1997b). Once samples were removed, a narrow bar was used to push material into the void created by the core sampler. Then the average sample was analyzed by moisture balance.

Data acquisition system

Temperature was recorded continuously for 20 days using the data logger (DIK-9421, Daiki, Japan) from where the data acquisition program was written to store the data in the personal computer. In case of moisture measurement, the measured data by the moisture balance was recorded manually and analyzed for further assessment.

Pilot-scale experiments

Pilot-scale experiments were performed using the dog food and food scrap mixtures at both 20 liter per minute (high flow) and 5 liter per minute (low flow) aeration rate and initial moisture content of 60% (weight basis). Table 3 provides an overview of the various operating conditions implemented in the decomposition experiments. Each of the experiments using the DF and FS were performed in duplicate to assess our ability to replicate process behaviour for fix initial moisture content but different aeration rate. The combination of moisture content, aeration rate and the two substrates were chosen to mimic process conditions typically used in industry and to capture the effect of each of these process variables on process performance (Bach et al., 1987; Cayuela et al., 2008; Nakano, 2002). All experiments were run for 20 days or until the temperature reached near to ambient temperature.

Results and Discussion

Reproducibility of reactor performance

An assessment of the reproducibility of the pilot-scale system can be attained from an analysis of temperature from two experiments with identical initial conditions and inlet air conditions. Fig. 5 shows profiles of temperature of DF composting experiments. Both experiments were similar, having an initial moisture content of 60% and an aeration rate of 20 liters/min. Trial 2 was performed 1 month after trial 1. These particular trials were selected because they represented the largest gradients observed in temperature among all the experiments. The temperature profiles shown in Fig. 5 correspond to those along the centre axis of the reactor at 25 and 50 cm bottom of the top of the reactor. At 50 cm both trials showed an initial 60-63°C temperature peak between second and third day and a stabilizing temperature profile after that period until about eighth day. Between third and eighth day the average temperature difference between the trials' 50 cm temperature profiles was less than 2°C, while between 0 and twentieth day the difference was less than 2-5°C. These differences were not much greater than the 2°C accuracy of the data acquisition system. At 25 cm (from the top of the reactor) both trials showed an initial 55-61°C temperature peak at second day. Although the profiles at 25 cm never reached a stable temperature probably because of drying as well as the loss of heat energy to the environment from the top uncover surface of reactor , the average difference between replicate profiles for the duration of the experiment was still less than 3°C. As the compost begins to cool after reaching peak temperature, turning the pile was operated at about twelfth day resulting a new peak of temperature because of the replenished oxygen supply and the release of inhibitor (probably generated NH_3 gas)and the exposure of organic matter not yet thoroughly decomposed. In both the trials' 50 cm drop of temperature due to loss of heat energy during turning was from 51°C to about 33°C and quickly recovered to a new temperature of 60°C within fourteenth day. After that the quick fall of temperature revealed that end of thermophilic phase (40°C-60°C) and the decomposition was taken over by mesophilic microbes (Andrews and Kambhu, 1973; Chanter and Spencer, 1974; Hasegawa et al., 2005; Kimura and Nakasaki, 1999; Saludes et al., 2007) through a long process of "curing" or maturation after end of degradation.

Table 3. Experiment conditions of different types of waste

Operating conditions		
Name of substrate	Dog food	Food scrap
Wt. of substrate, [kg-dry]	83	83
Wt. of Amendment, [kg-dry]	250	250
Added water [kg]	212.8	212.8
$\frac{Amendment}{Substrate},[\frac{kg}{kg}dry]$	3.0	3.0
Final water content (wt/wt) [%]	60.0	60.0
Aeration rate, liter per min	20.0	20.0

Verification of minimal radial gradients

Another requirement of the pilot-scale systems was minimal heat loss from the reactor walls. One method to determine if the reactor meets this requirement is an analysis of radial temperature gradients within the bed (VanderGheynst et al., 1997b). Fig. 6 shows radial temperature profiles at the mid-bed of reactor and an ambient temperature profile for an SFW (dog food) experiment (20 litres/min) with an initial moisture content of 60%. The magnitudes of the radial gradients observed in Fig. 6 are typical of the gradients observed in most of the pilot-scale experiments. At 50 cm the difference in temperature between the wall at both edge and corner and the centre of the reactor varied between 0 and 5°C, with an average difference less than 2°C. Maximum temperature differences (gradients >5.2°C) were observed at the final day of the experiment, which also corresponded to the onset of drying in the higher portion of the bed as shown in Fig. 7. If heat loss from the reactor walls were the cause of radial temperature differences, we would have expected to see the maximum heat loss and the maximum radial temperature difference when the difference between the wall temperature and ambient temperature was maximal (Mason and Milke, 2005). For this experiment the maximum difference in wall temperature and ambient temperature was approximately 37°C and the resulting radial gradient was less than 3°C. For both of the positions the magnitude of the radial temperature differences did not appear to be related to the ambient temperature, but did appear to be linked to drying and mechanisms of degradation by microbes (Nakasaki et al., 1987). A further case for the link to drying can be obtained from an analysis of the radial gradients observed in the high-aeration (20LPM) experiment which had relatively higher drying (Fig. 7).

Influence of aeration rate on decomposition

Another requirement of the pilot-scale system was to provide optimum air flow in order to ensure favourable environment of microorganisms to decompose organic substances. Fig. 8 depicts the variation of temperature at both high air flow (20 liter per minute) and low air flow (5 liter per minute) at the location of centre of mid-bed of the reactor of dog food composting experiment. In case of low aeration supply, the temperature initially followed the same path as the temperature of high air flow until half of a day but reached at the peak about 55°C and continued the steady state of this temperature until the turning at about twelfth day. It seemed that the initial oxygen consumption by the thermophilic microbes (Mari et al., 2003) was same in both high and low aeration rate but within a day, the rate of reaction ceased in case of low aeration rate and showed the steady state which revealed that inhibition effect is critical in case of low aeration supply than the same of high aeration supply. At the turning period, the temperature dropped from 54°C to about 46°C and recharged to about 57 °C and remained almost steady after reaching the peak. It suggests that the low aeration rate (5 LPM) is insufficient to decompose the organic substances or it is necessary to consider frequent turning operation so that oxygen can recharge the microbes and able to accelerate rate of degradation.

Degradation of different substrates

It is also critical to identify the mechanism of degradation for different types of substrates in pilot-scale experiment. The experiments were performed for unique 20 liter per minute of air flow and 60% of moisture content by weight. The average carbon to nitrogen ratio of dog food and food scrap is 15 and 12 respectively which shows higher amount of available nitrogen in food scrap than the dog food. Fig. 9 depicts the variation of temperature in different types of substrates (dog food and food scrap), where dog food exhibited higher temperature than the food scrap until turning operation was applied. The maximum difference of temperature between two types of substrates was about 6°C and average difference was less than 3°C until twentieth day. As dog food contained higher amount of protein and carbohydrate than the same of food scrap (Jakobsen, 1994), affected the development of biomass by microbes resulting higher consumption of substrates in case of dog food than the same of food scrap. Consequently, the temperature of dog food showed high value comparing to corresponding food scrap. Again, lower carbon to nitrogen ratio of food scrap possibly generated higher NH_3 during decomposition, which might inhibit (not extensively) between ninth to twelfth days as the temperature was showed almost stable; but after the turning operation the reactivation of temperature of food scrap was identified as same as the dog food which confirmed the release of possible inhibitor generated (Katayama et al., 1985) during decomposition of organic substrate of food scrap. After recharging by oxygen at turning operation, the both types of substrates degraded faster after reaching to peak temperature.

Effect of ambient temperature in airflow

Since the air compressor was used to supply constant flow of air inside the compost mass, it is important to relate the inflow temperature (T_{inflow}) of air with the ambient temperature (Masafumi et al., 2002). Fig. 10 shows the relationship between T_{inflow} and the ambient temperature, where it is evident that inflow temperature of air is strongly influenced by the ambient temperature.

Maturity of compost

The maturity of compost is important for application purposes; fresh and matured composts are distinguished prior to marketing to apply the compost as soil conditioner. There are some methodologies by which the maturity of compost can be identified, which include visual identification in field test to laboratory scale (Bernai et al., 1998; Canet and Pomares, 1995) by identifying indices of making the compost as mature. Usually, the degree of compost maturity is calculated by the maximum self-heating temperature (Körner et al., 2003) measured in different phase of composting. Fig. 11 depicts the maturity index of compost mass (DF case) after end of decomposition at twentieth days by self-heating approach from where, it is evident that the compost mass reached at the maturity phase IV from where few days of curing would be enough to ensure phase V level of maturation.

Hygienic implication of compost

Research has shown that compost achieving the “temperature/time” regime required for proper operation of large, permitted composting facilities is effective in pathogen destruction (although subsequent recontamination of the compost and re-growth of microorganisms can be a problem) (Hadas et al., 2004; Novinscak et al., 2007). Although it is commonly believed that reaching temperatures of 55°C for 3 days is sufficient to essentially eliminate bacterial pathogens (Yanko et al., 1995), recent work suggests that the control of bacterial pathogens in composting is more complex and not simply the result of thermal treatment (Droffner and Brinton, 1994). Salmonella, E. coli, and other bacteria survived high temperatures for a significant time (Hassen et al., 2001), but whether the high temperature resistant strains are pathogenic is unknown. Moisture level (Pereira-Neto et al., 1986), for example, is also important in the survival of E. coli through the composting process. It has been suggested that microbial competition is also important in the destruction of pathogenic organisms in compost. If so, if finished composts with low levels of competing microorganisms become inoculated with pathogens, there would be an increased potential for high pathogen levels due to re-growth in the absence of competition. However, microbiological tests were performed (Tests standard description in the Table 4) to ensure the safety of compost product after two weeks of curing of compost mass after finishing degradation. The tests results (total three samples) were described in the Table 5, from where it is evident that the compost is hygienically safe to use as soil conditioner.

Table 4. Microbial tests of pathogens

Name of the pathogen	Testing references
C. perfringens	Clostridium perfringens: Membrane Filter Method, ICR Microbial Laboratory Manual. USEPA Office of Research and Development, Washington DC. EPA/600/R-95/178 (1996)
E. coli	IDEXX Colilert System
Total Coliform	IDEXX Colilert System
Enterococci	IDEXX Enterolert System
Fecal Coliforms	Fecal Coliforms in Biosolids by Multiple-Tube Fermentations and Membrane Filtration Procedures: EPA Method 1680 (EPA-821-R-98-003)
Salmonella	Detection and Enumeration of Salmonella sp. (Kenner and Clark, 1974) as published by EPA (1992) Environmental Regulations and Technology. Control of Pathogens and Vector Attraction in Sewage Sludge. pp 107-115.

Table 5. Results of Microbial test of compost samples

Sample No.	Total bacterial count c.f.u/gm	Total coliform c.f.u/g	Salmonella spp. c.f.u/g	Fecal coliform c.f.u/g	E. Coli c.f.u/g
Sample -1	6.50E+09	1.1×1000	Nil	Nil	Nil
Sample-2	7.00E+09	4.5×1000	Nil	Nil	Nil
Sample-3	6.20E+09	5.1×1000	Nil	Nil	Nil

Recycling of compost

One of the key considerations of this pilot-scale experimentation is to assess the degradation of recycle compost. Using amendments for structural conditioning without product recycle can result in consumption of considerable quantities of amendment. This can be expensive, depending on local availability of suitable materials. Also, the quantity of final product is increased compared to systems that rely all or in part on compost recycle for conditioning. Whether this is an advantage or not depends on the expected market for the final product. A compromise between use of product recycle alone and use of amendment alone is possible. If recycle is used for part of the structural conditioning, smaller quantities of amendment can be used for the balance. However, recycling of compost products reduces the demand for raw materials, allowing waste materials to be used again and again and again. The Fig. 12 illustrates the recycling operation in composting schematically. Many researchers assessed the process of composting using recyle compost and (Haug, 1993), (Gajdo, 1998) reported that the rate of degradation would increase while the recycle compost would be used in degradation as cultures of microbes are already developed in the recycled compost which was responsible to foster rate of reaction. On the otherhand, excessive use of recycle compost may lead to reduce the porosity substantially and for which it is possible to cease the reaction of decomposition further. So, it is very critical to identify the optimum portion of recycle compost in this STR composting system. In order to identify the mechanism of degradation of organic substrates using recycle compost, two cases of experiments were performed. One wa half of the total amendment material was recycle compost (50% recycle) and the other one was total amendemnt was recycle compost (100% recycle). The amendment to substrates ratio as well as the moisture content were controlled to keep as 3.0 and 60% by weight respectively. The Fig. 13 shows the analogy of temperature profile of both first and recycle composting using dog food as substrate. From the Fig. 13, tt is evident that use of 50% recycle compost charged the microbes to foster the decomposition at the initial phase of composting for which the peak temperature for 50% recycle compost was higher (about 6°C) than the first composting (no recycling). Again, after the turning operation, the temperature of 50% recycle case depleted faster than the same of first composting revealed fastar degradation (rate of reaction) using recycle compost. On the contrary, use of 100% recycle compost as amendment material lead to cease of reaction than the same of first composting case reveealed less peak of temperature (about 6°C) than the same of first composting. The probable reason of ceasing of reaction might be reduction of free air space in the interstitial pore sturctures of compost mass for which oxygen uptake rate by microbes might decrease as well. Moreover, the generated gas might also be responsible to inhibit this rate of reaction for the case of 100% recycle compost case.

Summary and Conclusions

Pilot-scale system has been presented which mimic the central core of an aerated static reactor, so called STR composting system, which has considered an innovative and energy-efficient technology on compost material mixing and turning operation. An assessment of the ability of the system to duplicate reactor performance was done by analysis of temperature and moisture using two different substrates in the composting experiments with identical initial and inlet air conditions. The average difference observed between temperature profiles was less than 2.5°C and radial temperature gradients were measured to assess whether heat loss from the reactor walls was significant. On average, radial temperature differences were less than 2°C and their occurrence has been linked to drying rather than ambient temperature and heat loss from the walls. Based on these analyses it can be concluded that the experimental system replicates well and can be assumed to represent a process with spatial variation in the state variables.

The coincidence of axial temperature gradients with axial gradients in moisture has displayed the coupling of mass transport to energy transport. The observation of maximum temperatures at higher axial positions and of decreased maximum temperatures as aeration rate increased has shown the influence of aeration rate on microbial activity as well as its influence on cooling.

The degradation of different substrates has depicted the importance of carbon to nitrogen ratio of the substrates degradation. The increasing amount of protein and carbohydrate revealed energy liberation by microbes after forming biomass which might be an important consideration in assessment of decomposition of substrates of pilot-scale experiments. This has shown that the reaction may cease due to inhibition which should be considered carefully in handling different types of substrates. Indeed, the optimum carbon to nitrogen ratio of the substrate might eliminate this inhibition problem.

Again, the assessment of maturity index represents the end of degradation and it is found that after 20 days of degradation, the compost mass reached at phase IV level of mature compost and soon after two weeks of curing would ensure hygienically safe compost to apply in the agricultural purpose.

Again, experimentation of recycle compost in these pilot-scale experiments may lead to contribute a lot in operation and management of STR composting system in treatment of organic substances as it was found that use of recycle compost is justifiable in composting operation.

It is apparent from the results presented that within a static bed degradation process, there exists considerable temporal and spatial variation in state variables such as temperature and moisture content. Now that the pilot-scale reactors have been characterized and their limitations defined, the temporal and spatial gradients in process parameters can be used to gain insight into transport phenomena and bio-kinetic mechanisms within aerobic high-solids fermentation systems.

Acknowledgements

This research was supported by the Japanese government scholarship; e,g, Monbukagakusho as a doctoral research on Biotechnology in the University of Tokyo, Japan.

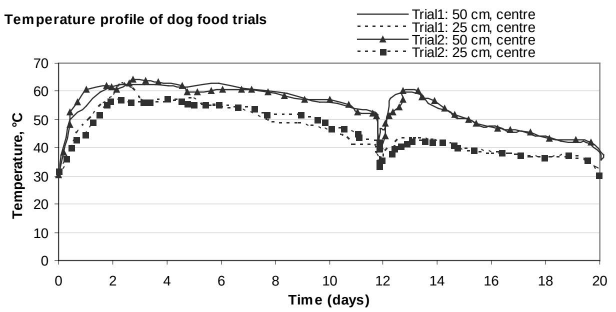

Fig. 5. Temperature profiles obtained at 50 cm and 25 cm from the top of the reactor

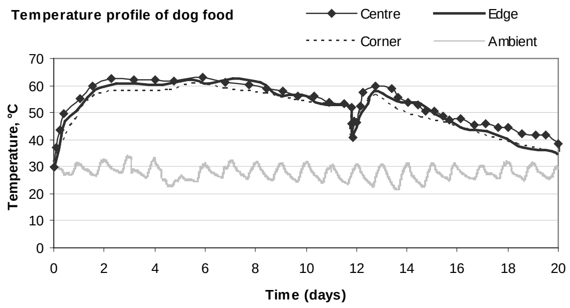

Fig. 6. Ambient and compost temperature profile at the middle of the reactor

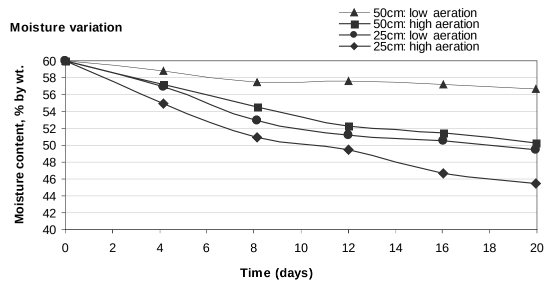

Fig. 7. Moisture content profiles of dog food obtained 50cm and 25cm from the top of the reactor for low aeration (5LPM) and high aeration (20 LPM) rate

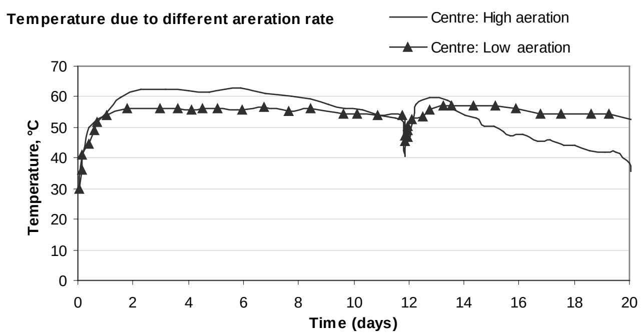

Fig. 8. Temperature variation of dog food at middle of centre of reactor due to different aeration rate

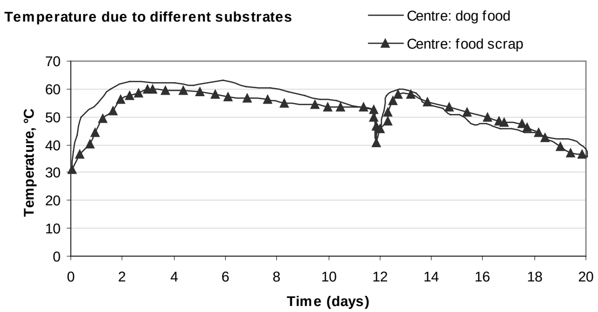

Fig. 9. Variation of temperature at centre of mid-bed of the reactor considering different types of substrates

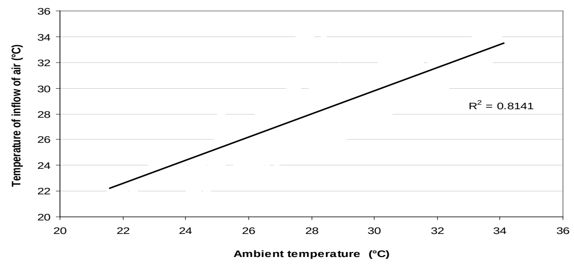

Fig. 10. Relationship between the ambient temperature and the temperature of inflow air

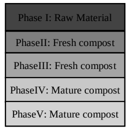

Fig. 11. Maturity index of compost using substrate as dog food

Fig. 12. Schematic representation of concept of recycling

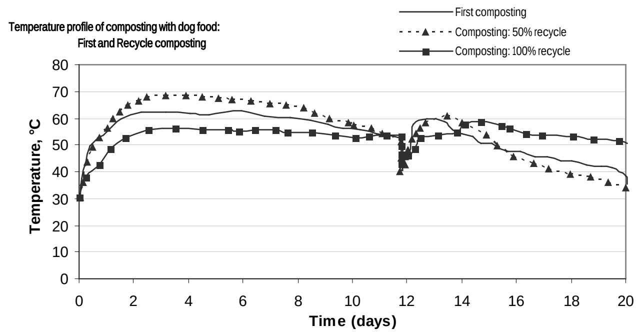

Fig. 13. Analogy of temperature profile of first and recycle composting

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Water Resources Engineering

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PHYSICAL MODEL STUDIES TO SUPPORT THE DESIGN OF MAIN SPILLWAY OF THE PROPOSED GANGES BARRAGE

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ABSTRACT

A barrage across the Ganges River at Pangsha has been proposed by feasibility consultants to store water and augment dry season flow into the distributaries of the Ganges. The waterway width (1728m) of the barrage consists of main spillway and undersluice bays, fish passes, navigation lock and hydro-power station. River Research Institute (RRI) has conducted detail model study for main spillway to support its design in terms of afflux, discharge co-efficient, water surface profile, scour profile downstream of the stilling basin with and without block protection under submerged unregulated flow conditions and nature of flow in the stilling basin under regulated flow conditions. The scale ratio of the model is set as 1:24. The model has been constructed in a straight flume. The barrage section reproduced in the model consists of one full bay (18m), two piers of 2.5m width and half of the portion of bays on the other side of each pier. The flume width is 1.71m. The stage-discharge curve established for the barrage site has been used to conduct model investigations. This paper presents the experimental results of design alternatives of main spillway and stilling basin and also discusses the results in terms of their reliability and effectiveness. It is revealed that the design of main spillway is appropriate and the low Froude number basin works well to stabilize the hydraulic jump within the stilling basin.

Keywords: *Barrage, Main Spillway, Afflux, Stilling Basin, Hydraulic Jump, Froude Number*

1. INTRODUCTION

The construction of the proposed Ganges barrage is of immense importance for effective utilization of the Bangladesh share of Ganges flow and for the benefit of the people living in the Ganges Dependent Area. The layout of barrage is shown in Figure 1. Physical model studies are being conducted in RRI to support the Feasibility Study and Detailed Engineering for Ganges Barrage Project. The maximum waterway width of 1728m is established by model studies. The total spillways and undersluices length has been taken as 1620 m, corresponding with 78 spillway bays and 18 undersluice bays with 18 m wide openings.

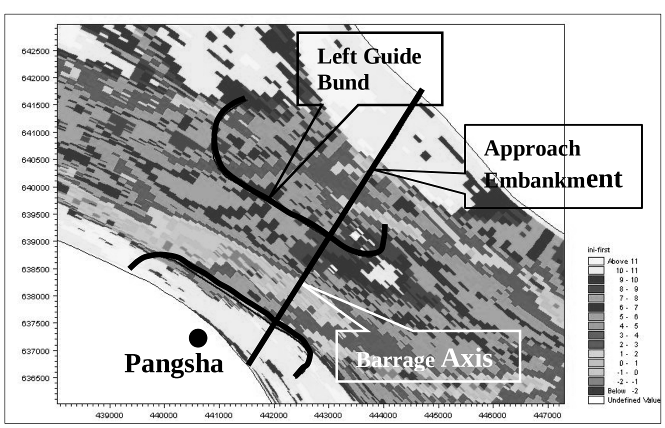

Figure 1: Layout of the proposed Ganges barrage at Pangsha

The main spillway is an important component of a barrage. Physical model studies play a vital role in planning and design of hydraulic structures like main spillway. Design of hydraulic structures is generally refined on the basis of detail physical model studies. The principal features and dimensions of the proposed Ganges barrage have been designed by the detailed engineering consultants on the basis of physical overall model and numerical model investigation results. The detail model for main spillway mainly aims to determine the afflux, surface water profile, discharge co-efficient etc. under different submerged unregulated flow conditions and to observe the hydraulic performance of stilling basin as designed and to suggest the modification of stilling basin if any for better performance in gated conditions. Appropriate scale between model and prototype structure plays imperative role in terms of rationality and reliability of the model results. This type of models gives best results if undistorted because of three-dimensional nature of flow pattern. Such undistorted models are also well fit for study of scour and stability of scour protection, which in many cases are an integral part of the local studies.

The main spillway is the main body of the proposed Ganges barrage, normal RCC slab that supports the steel gate. The section of the main spillway is shown in Figure 2.

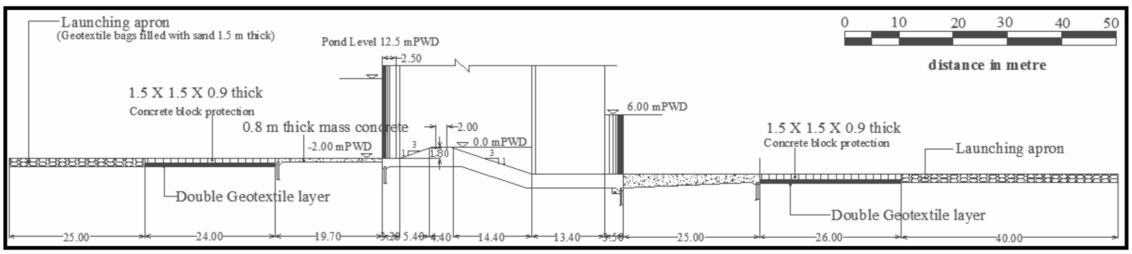

Figure 2: Section of main spillway

The section of the main spillway consists of upstream concrete floor at elevation -2 mPWD, crest at elevation 0 mPWD, upstream slope (3:1), downstream slope (3:1) and downstream concrete floor at elevation -5 mPWD. There are also upstream sheet piles, downstream sheet piles, intermediate sheet piles and concrete block protection and launching apron both upstream and downstream.

The scale ratio of the model is set as 1:24. The model has been constructed in a straight flume using indoor modelling facilities of RRI. The barrage section reproduced in the model consists of one full bay, two piers of 2.5m width and half of the portion of bays on the other side of each pier. The flume width is 1.71m for the selected scale ratio. The test section of the model is constructed in this flume one side of which is fitted with steel sheet and the other side with transparent plastic panels to facilitate visual observations inside the flume. The radial gate, cement concrete blocks and loose protection works have been fabricated in the model as per design.

Detail model investigations for main spillway have been conducted with some specific objectives in view. Five test series have been conducted for determination of the needed information in unregulated and gated conditions for different discharges. The information derived for unregulated submerged flow conditions are (i) water surface profiles (ii) discharge co-efficient (iii) reduction of discharge co-efficient with submergence with and without basin elements in place and (iv) water surface profiles and scour profiles downstream of the stilling basin with and without block protection and launching apron in place. The information derived for gated conditions are the nature of the hydraulic jump and the position and depth of maximum scour downstream for a critical downstream release condition with alternative designs of stilling basin in place. The other tests in gated conditions are yet to be conducted. Some findings of the model study have been presented in this paper.

2. METHODOLOGY

2.1 Model Design

The model for main spillway has been constructed using RRI indoor modelling facilities. The model has been designed to accommodate a short section of the main spillway. In the model the prototype situation has been replicated in a flume on a scale ratio of 1:24. The model is undistorted and the geometric scale is selected based on the available laboratory space, pumping capacity, measurements, dimension of the structure, governing processes to be simulated and scale conditions to be fulfilled. With this scale the model is of sufficient size that surface tension is minimized and that surface resistance is not greatly out of scale. In order to reproduce water flow and scour downstream of the structure following scale conditions are to be fulfilled.

i. Geometric Condition

The geometric condition is important to be fulfilled in order to diminish scale effects related to the three dimensional flow. The geometric condition is fulfilled when the length scale is equal to the depth scale i.e.

$L_r = h_r$, where L_r = length scale and h_r = depth scale

ii. Roughness Condition

In an undistorted model the roughness condition is fulfilled when the scale of the Chezy roughness is equal to 1 or when the water slope in the model is equal to the water surface slope in prototype *i.e.*

$C_r^2 = L_r / h_r = 1$, where C_r = roughness scale

iii. Froude Condition

The Froude condition is important to have a dynamic similarity for free surface flow with respect to the influence of gravity. The Froude condition is fulfilled when $V_r = L_r^{0.5} = h_r^{0.5}$ ($L_r = h_r$). The discharge scale for the undistorted model can be determined from $Q_r = L_r^{2.5} = h_r^{2.5}$

iv. Transport Intensity

One of the main objectives of the model study is to determine scour downstream of the structure under different discharges. The following scale condition has to be satisfied for reproduction of the transport intensity when most of the sediment in the prototype is transported as suspended load.

$V_r = C_r D_r^2 \Delta_r$, where,
V_r = velocity scale
C_r = roughness scale
D_r = diameter scale and
Δ_r = relative density scale

The above scale condition has led to the selection of appropriate d_{50} of the model bed sand so that the velocity scale needed for fair reproduction of the transport intensity in the model almost corresponds to the velocity scale according to Froudian law of similitude. The relationship of dimension and hydraulic quantities between model and prototype, which is based on Froudian law is given in Table 1.

Table 1: Relationship of dimensions and hydraulic quantities between model and prototype

Parameter	Unit	Scale
Length (L)	m	24
Depth (h)	m	24
Velocity (V)	m/s	4.9
Discharge (Q)	m^3/s	2822
Time (T)	s	4.9
Froude number (Fr)	-	1
Roughness co-efficient (n)	$m^{-1/3}s$	1.7

2.2 Model Setup

The model setup includes model scale, model discharge, water re-circulation system, gauging stations, tailgate and proposed structures. The layout of the model is shown in Figure 3. The model discharges are selected from the established stage-discharge curves of the river at the barrage site. The stage-discharge curves consist of upper, middle and lower curve. The middle curve has been used as tailwater rating curve during model investigations. The maximum discharge required by the model of about 0.75 m^3/s has been supplied by pumping. Two 1.8m rectangular weirs, located in an extension of the main flume have served as a means of measuring the inflow discharge. 4 (four) gauge wells, two in the upstream and two in the downstream have been installed with point gauge to read water surface elevation. Besides, there are two staff gauges upstream and downstream (near the toe of the glacis) of the weir for visual observation of water level. A hinged tailgate at the end of the flume has been installed for controlling downstream water level. The design of the structures reproduced in the model (Figure 4) has been done by the detailed design consultants. The radial gate, cement concrete blocks and loose protection works have been fabricated in the model as per design.

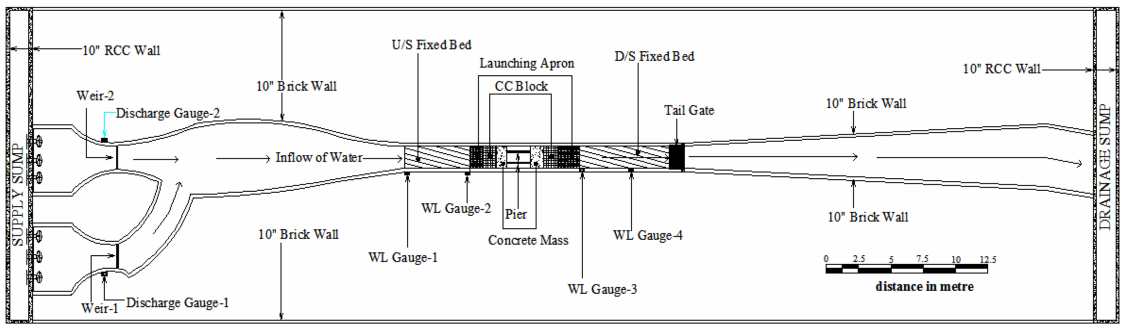

Figure 3: Layout of the detail model for main spillway

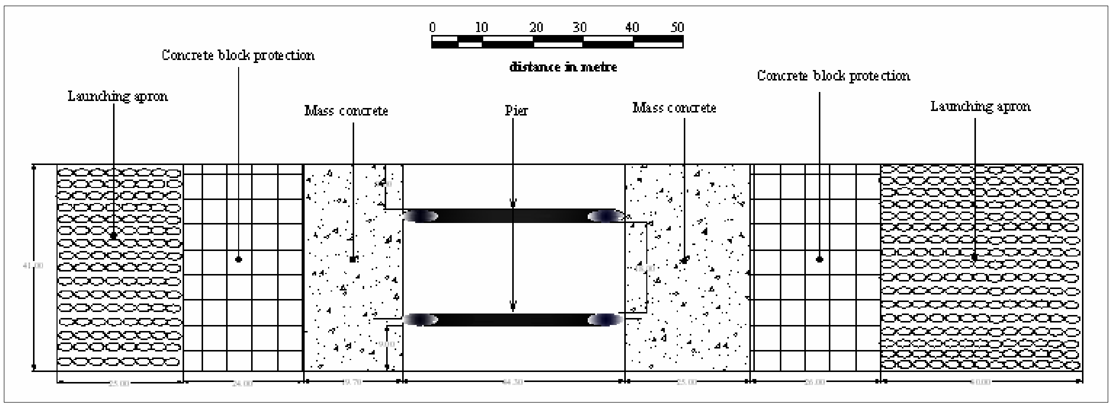

Figure 4: Plan of barrage portion for flume test

2.3 Test Procedures

The test conditions have been established in order to achieve the study objectives. Five test series have been conducted for determination of the needed information in unregulated and gated conditions for different discharges. The discharges and corresponding tail water levels have been selected from the stage-discharge curve. Model discharges corresponding to different prototype discharges have been determined for 2(two) bays. It is to be noted here that initially prototype discharge for 2 bays (36m opening) has been determined considering 90 bays (including undersluice bays) of the barrage and tests have been conducted accordingly. Afterwards some of the important tests have been repeated as because it is known that the Feasibility Consultants have decided to increase the total numbers of bays from 90 to 96. In case of submerged unregulated flow the model measurements include water level at pre-fixed locations including headwater level, water depth upstream and downstream of the weir, velocity of approach, downstream velocity and downstream scour level. On the other hand, a critical downstream release condition has been tested for regulated flow with alternative designs of stilling basin. The tests involve measurement of headwater level, water level at jump initiation point and after jump, velocity of approach, velocity after jump and distance of the jump initiation point from the middle of the crest for varying gate openings. The equipment that have been used for recoding different data include A-ott propeller type (vertical axis) current meter, levelling instrument, point gauge, stop watch, meter scale and video and still camera.

3. TEST RESULTS AND DISCUSSION

Necessary measurements have been taken during the tests and the measured data has been processed and analysed. The test results are discussed hereafter.

3.1 Head-discharge and Head-discharge co-efficient Curve

The tests have been conducted for a number of discharges ranging from 1000 m^3/s to 100000 m^3/s and both with and without basin blocks in place to gain understanding of the effects of basin blocks on discharge co-efficient and afflux. It is to be noted here that for each discharge tailwater level has been varied between a range of water levels higher and lower than the water level obtained from the intermediate tailwater water curve to see its effect on discharge co-efficient, submergence ratio and afflux.

During the tests for determination of co-efficient of discharge the velocity of approach has been measured by current meter and also computed on a discharge area basis. The total head-discharge curve has been plotted first. Since the co-efficient of discharge is very sensitive to slight errors, points of head and discharge have been read from the curve and substituted in the following expression for establishing the shape and position of the co-efficient curve.

$C_d = Q/LH^{3/2}$ where, Q = Measured discharge (m^3/s)
C_d = Co-efficient of discharge ($m^{1/2}/s$)
L = Length of crest exclusive of pier (m) and
H =Total head on the crest (m)

The above equation indicates that the co-efficient of discharge is not dimensionless. The prototype to model ratio of the discharge co-efficient when expressed dimensionally gives $C_{dr} = g_p/g_m$. Since the acceleration of gravity in model and prototype is same, C_{dr} =1. It means the co-efficient of discharge obtained from the model can be applied directly to the prototype. The head-discharge and head-discharge co-efficient curves have been plotted for the prototype. It is thus possible to determine the co-efficient of discharge for the design headwater from the head-co-efficient of discharge curve. The head-discharge curves for different conditions appear in Figure 5 to 7. The head-co-efficient of discharge curves for different conditions appear in Figure 8 to 10.

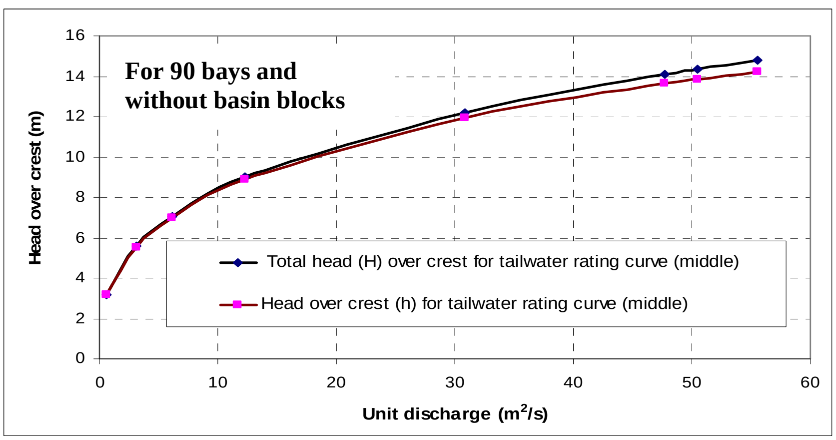

Figure 5: Head-discharge curve for 90 bays and without basin blocks

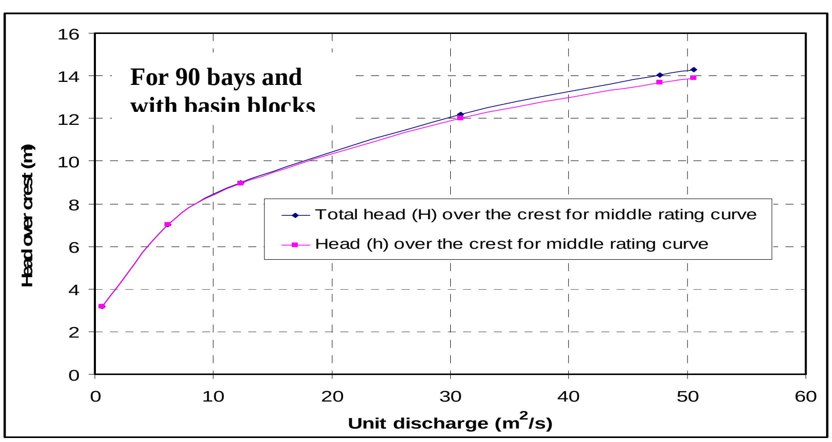

Figure 6: Head-discharge curve for 90 bays and with basin blocks in place

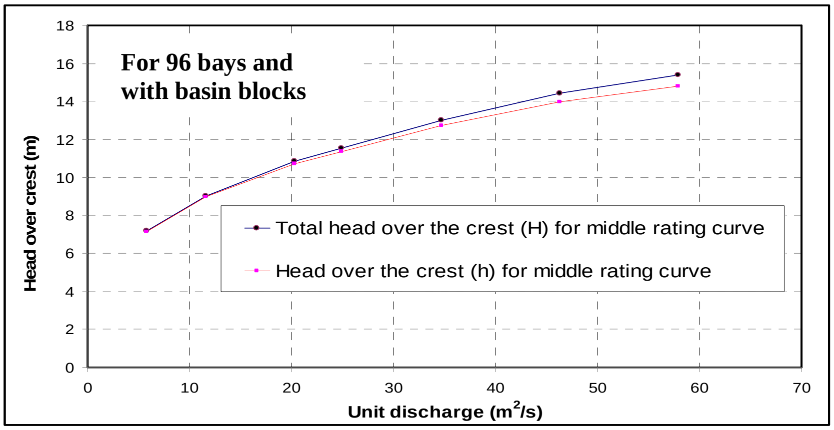

Figure 7: Head-discharge curve for 96 bays and with basin blocks in place

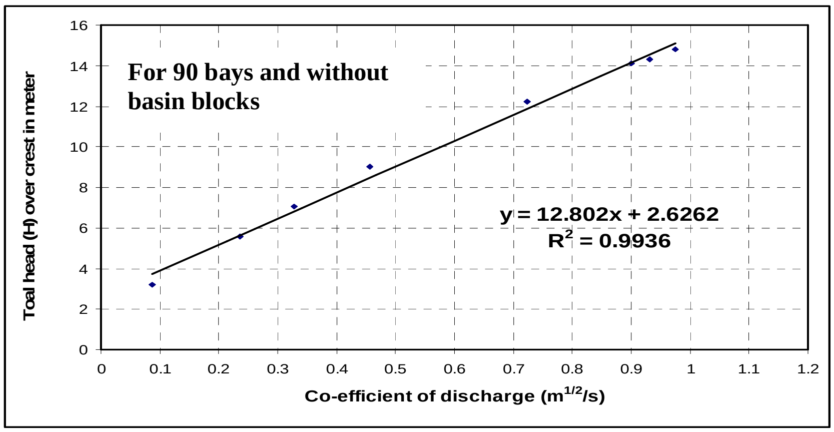

Figure 8: Head-co-efficient of discharge curve for 90 bays (without basin blocks)

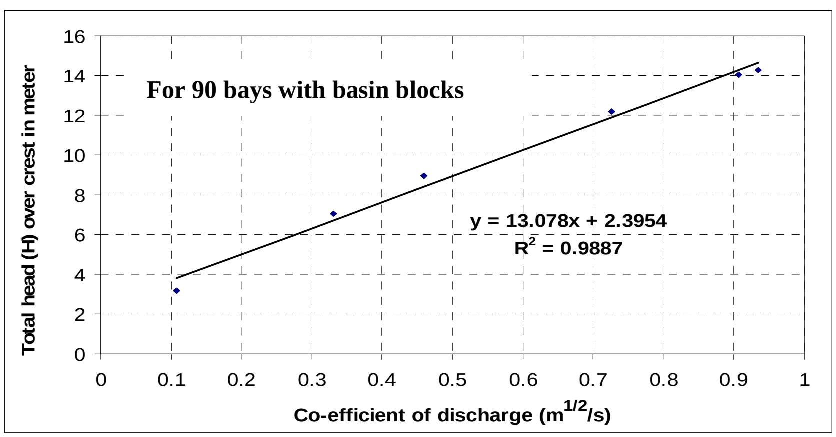

Figure 9: Head-co-efficient of discharge curve for 90 bays and with basin blocks

It is observed from Figure 5 to Figure 7 that the basin blocks do have small influence on head over the crest for all discharges. A slight decrease in the head is observed for initial design of basin blocks. On the other hand, for the same design of basin blocks an increase in the numbers of bays from 90 to 96 causes a reduction in the velocity head but a small increase in the total head for all discharges under submerged unregulated flow condition. For design discharge total head over the crest is found to be 14.2 mPWD and 14.4 mPWD for a total numbers of 90 bays and 96 bays respectively with basin blocks in place. As to co-efficient of discharge it can be seen from Figure 8 and Figure 9 that the initial design of basin blocks does not have any noticeable effect on the same for any discharge. However, co-efficient of discharge does decrease due to increase in the total number of bays as can be seen from Figure 9 & Figure 10. For the design headwater the co-efficient of discharge is found to be 0.93 $m^{1/2}/s$ and 0.85 $m^{1/2}/s$ for total numbers of 90 bays and 96 bays respectively with basin elements in place.

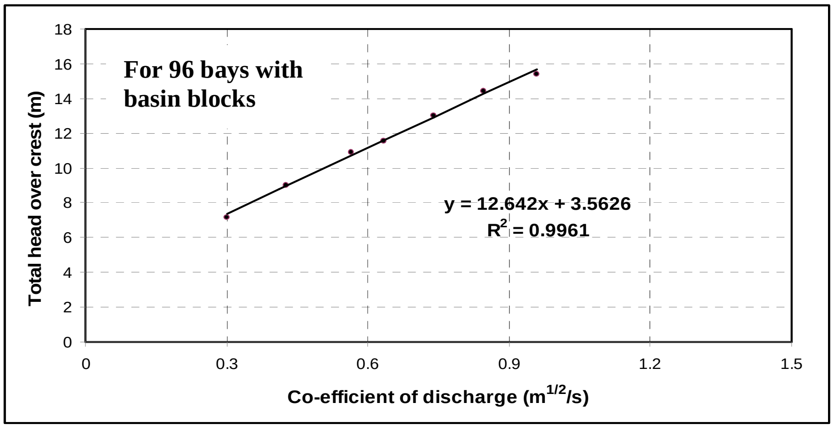

Figure 10: Head-co-efficient of discharge curve for 96 bays and with basin blocks

3.2 Effect of Submergence on Discharge Co-efficient

The reduction in the co-efficient of discharge with submergence is shown in Figure 11 and Figure 12 for without and with basin blocks in place respectively. C_S/C_F in the Figures is the ratio of co-efficient of submerged discharge to free discharge, corresponding to the same headwater level. On the other hand, h_1/H is the ratio of downstream head over the crest to upstream total head over the crest. During the tests the tailwater level obtained from the rating curve for each discharge has been varied within a range. In reality the variation of the tailwater level for a particular discharge due to likely future retrogression and accretion will be high for low discharges and low for high discharges. For a particular discharge afflux is found to have increased with a decrease in the tailwater level i.e. afflux varies with submergence. It is to be noted here that a decrease in the tailwater level for a particular discharge results in a corresponding decrease in the headwater level and an increase in the approach flow velocity. It can be seen from the Figures that for the same submergence discharge co-efficient is higher for basin blocks in place compared to without basin block situation. Based on the test results for submerged unregulated flow afflux has been determined. It can be stated from the test results that the afflux value will be very less due to high submergence and as tailwater rating curves show that there will be little variation in the tailwater level for a particular high discharge. However, for design discharge a likely drop in the tailwater level by about 1m may result in an afflux of about 0.5m.

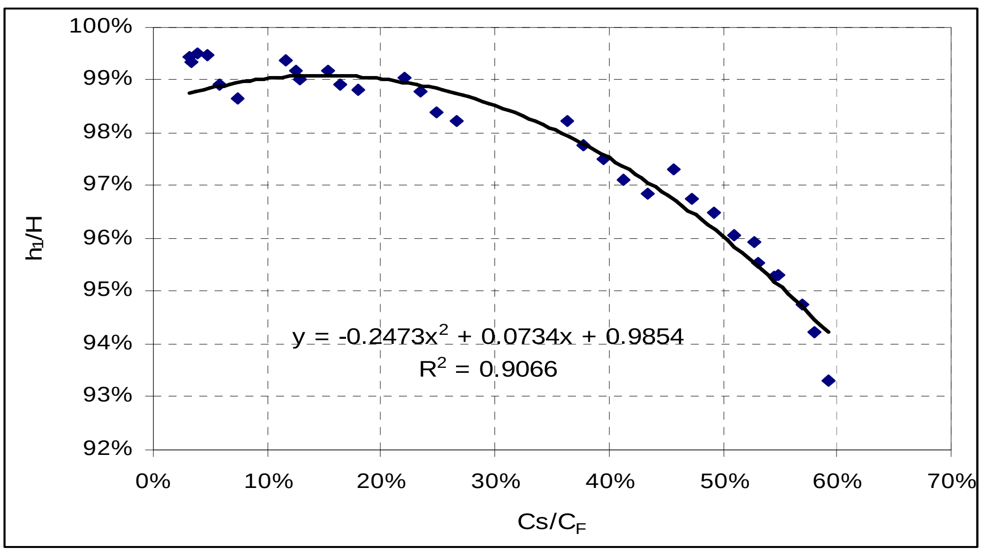

Figure 11: Reduction of discharge co-efficient with submergence (without basin blocks)

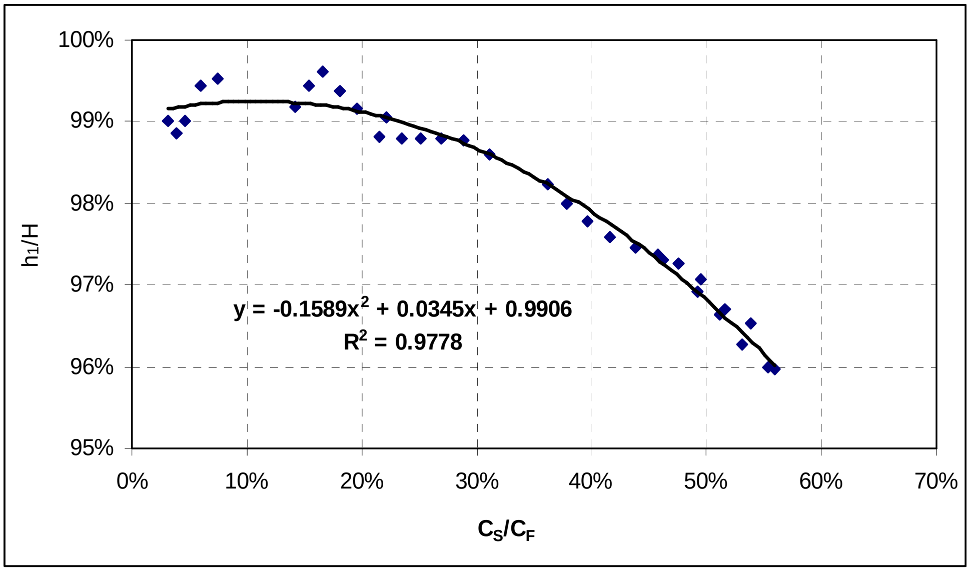

Figure 12: Reduction of discharge co-efficient with submergence (with basin blocks in place)

3.3 Water Surface Profiles

Water surface profiles for different discharges throughout the barrage section have been measured. For a particular discharge the profiles have been recorded for different levels of tailwater. The variation of water surface profile under varying tailwater levels for a discharge of 81820 m^3/s is shown in Figure 13.

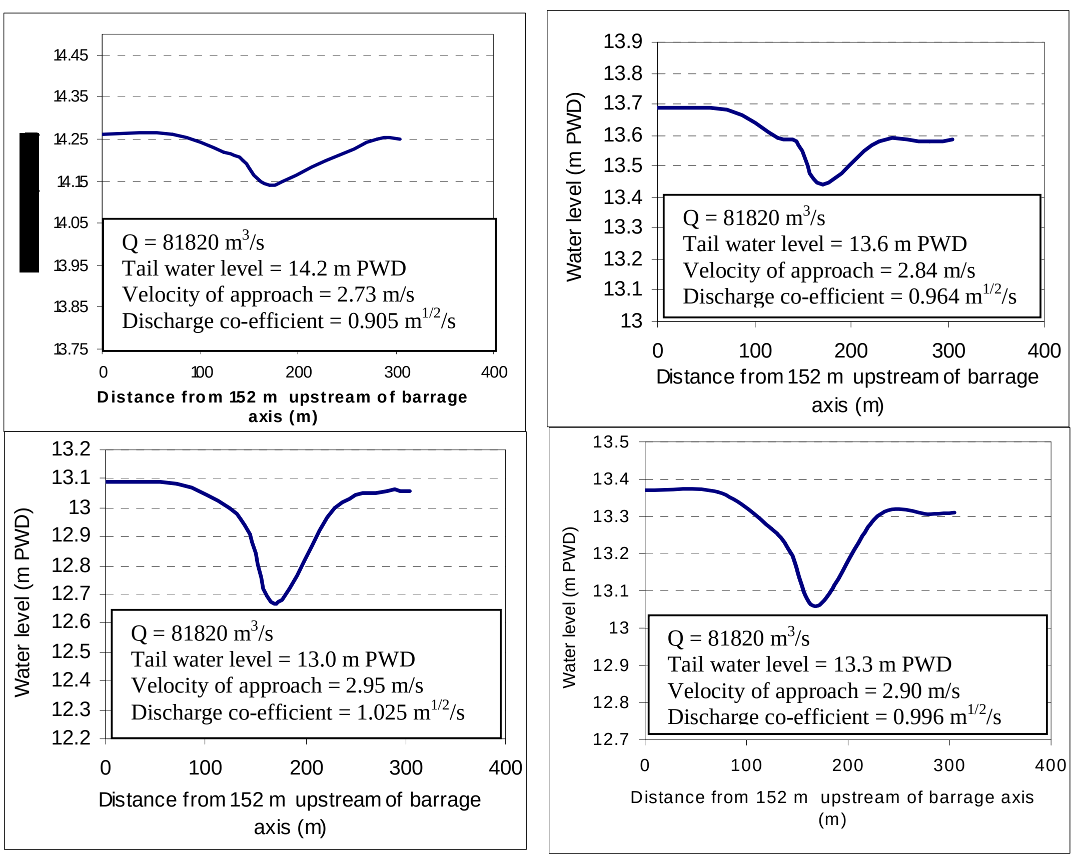

Figure 13: Variation of water surface profile under different tailwater levels for a discharge of 81820 m^3/s

It is to be noted here that 5 (five) well gauges have been installed in the model to record the water levels for determining water surface profiles. Among the five well gauges two lie in the upstream and three lie in the downstream of the barrage axis.

3.4 Scour and Water Surface Profiles

Tests have been conducted to determine the scour and water surface profiles downstream of the stilling basin with and without block protection and launching apron for different discharges in unregulated submerged flow conditions. Basin blocks have been reproduced according to the initial design (USBR Type II). During the test run for each discharge the development of scour with time has been recorded at a regular time interval and test run is continued until an equilibrium scour condition is reached. It is found that the scour holes for different discharges have reached an apparent state of stability within 2 (two) hours to 3 (three) hours. This will represent about 10 (ten) hours to 15 (fifteen) hours in the prototype. The test results without block protection and launching apron appear in Figure 14. The scour potential downstream of the stilling basin can be seen from Figure 14. For design discharge scour level downstream of the stilling basin could be as low as about -10.0 mPWD. Scour level can go down to -12.0 mPWD for a discharge of 100000 m^3/s. During these tests a few test runs have been conducted keeping the bed beyond the upstream floor exposed to erosion. Large bed scour potential is also observed upstream of the barrage. However, no scour measurement has been taken there. Based on this information next tests have been conducted with block protection and launching apron both upstream and downstream of the barrage.

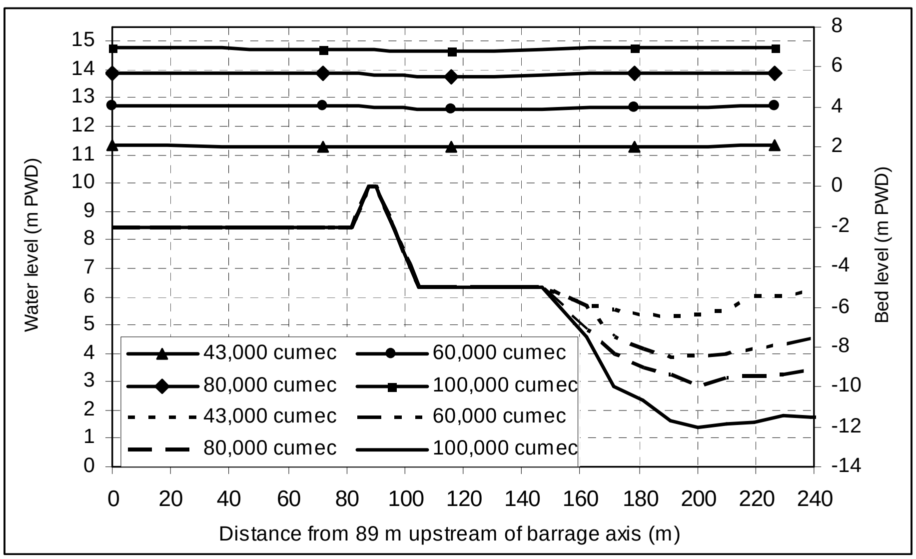

Figure 14: Water surface and scour profiles downstream of stilling basin for different discharges

The block protection and launching apron both upstream and downstream of the barrage have been fabricated as per design. The placement of block protection downstream of the stilling basin is shown in Figure 15. The model is run with different discharges until equilibrium scour condition is reached. The water surface and scour profiles downstream of the block protection and launching apron for different discharges have been shown in Figure 16. It can be seen from Figure 16 that the deepest scour level downstream of the launching apron (assorted cc blocks) is as low as -8 mPWD and -10 mPWD for a discharge of 80000 m^3/s and 100000 m^3/s respectively. It is observed during the tests that bock protection remains fully intact for all discharges both upstream and downstream of the barrage.

3.5 Investigation of Stilling Basin

For investigation of the hydraulic performance of the stilling basin as designed, tests have been conducted in gated conditions. A critical test condition has been considered for the investigation. The downstream release through one bay is 200 m^3/s with tailwater elevation at 2.0 mPWD. The pond level is 12.5 mPWD. It is obvious that under this condition the amount of gate opening will have influence both on the nature of hydraulic jump downstream of the barrage and headwater level upstream.

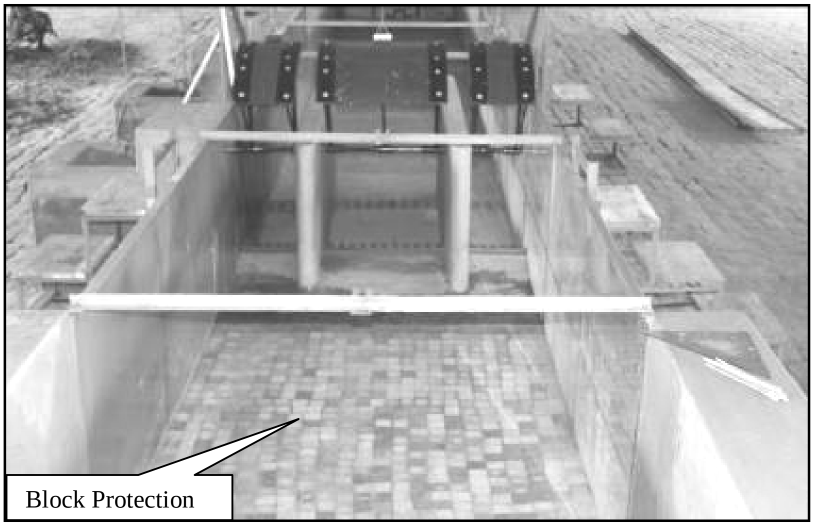

Figure 15: Placement of block protection downstream of the stilling basin

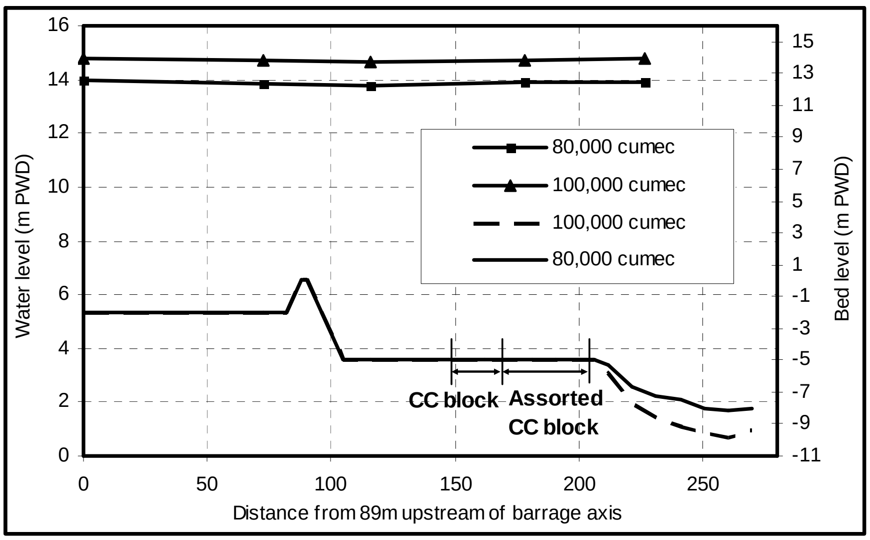

Figure 16: Water surface and scour profile downstream of the bed protection works for different discharges

It is observed that under this condition an oscillating form of jump occurs. The entering jet intermittently flows near the bottom and then along the surface of the downstream channel. This oscillating flow causes objectionable surface waves that carry far beyond the end of the basin. The hydraulic performance of two stilling devices has been tested. The stilling devices are (i) USBR Type II Basin and (ii) Low Froude Number Basin. The performance of the USBR Type II basin appears to be not up to the mark. The test results show that 0.96 m gate opening results in a headwater level of 11.8 mPWD. The hydraulic jump is found to have formed over the glacis at about 5.25 m downstream of the barrage axis. However, in the stilling basin waves have been observed that continue downstream even beyond the end sill with much wave activity. Substantial scour is observed at loose bed in the downstream of the stilling basin. High flow velocity is found just downstream of the end sill. It means Type II basin is less effective for dissipating the bulk of energy of flow. The alternative design of stilling basin (low Froude number basin) has chute blocks, baffle piers and a dentated end sill (Figure 17). It is observed that with this design of stilling basin the hydraulic jump gets stabilized within the stilling basin very well. Immediately downstream of the end sill velocity is found to have reduced substantially. And also scour downstream of the stilling basin is reduced. A relationship has been developed between gate opening and headwater level wherefrom it is seen that about 0.92m gate opening will be needed for maintaining pond water level (12.5 mPWD) under this condition. Pertinent information of the hydraulic jump is furnished in Table 2.

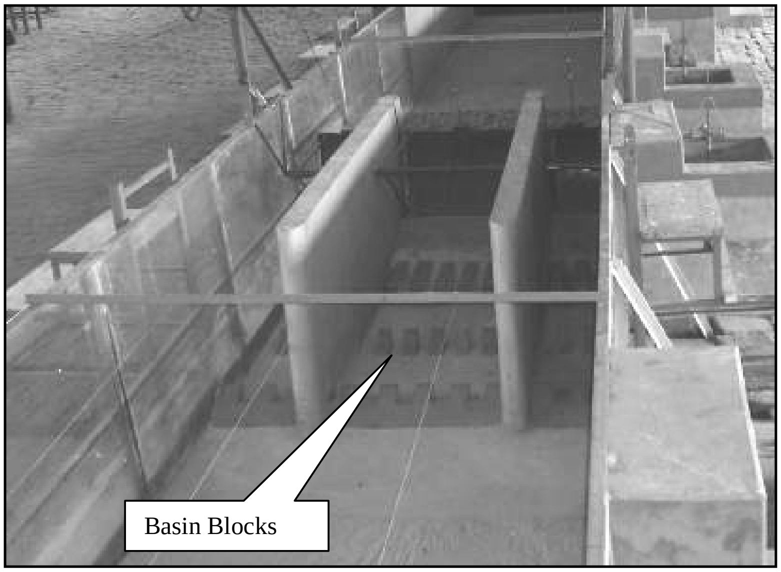

Figure 16: Arrangement of basin blocks in the model for low Froude number basin

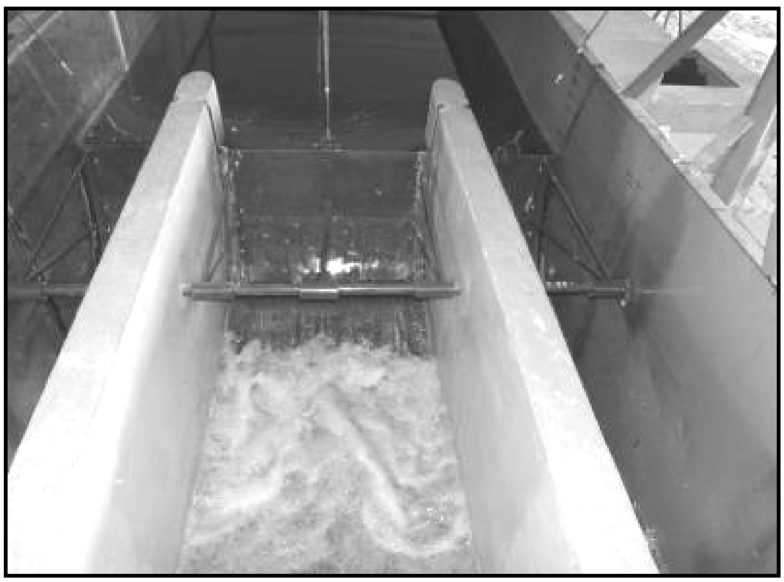

Figure 17: Nature of flow in the low Froude number stilling basin for the considered critical test condition

Table 2: Hydraulic parameters of flow in low Froude number stilling basin (for critical test condition)

Prototype unit discharge (m^2/s)	**Prototype Gate opening (m)**	**Headwater Level (mPWD)**	**WL at jump initiation point (mPWD)**	**Vel. at d/s of the end sill (m/s)**	**Dist. from jump initiation point to crest (m)**
11.11	0.84	13.36	-0.5000	1.47	5.20
11.11	1.01	11.56	-0.3600	1.55	4..90
11.11	1.18	9.85	-0.2200	1.65	4.56
11.11	1.34	7.52	-0.1000	1.87	4.39

4. CONCLUSIONS

The following conclusions have been drawn based on the study results:

1) The adopted sill level, crest width, glacis slope and upstream and downstream floor level of the main spillway appear to be appropriate in terms of anticipated afflux, discharge co-efficient and bed scour upstream and downstream of the barrage etc.

2) Total head over the crest and co-efficient of discharge for design discharge is 14.4 mPWD and 0.85 $m^{1/2}/s$ respectively with basin blocks in place (96 bays including undersluice bays)

3) The anticipated scour level downstream of the stilling basin of the main spillway is observed to be as low as -10.0 mPWD for design discharge.

4) Due to high submergence afflux will be very less for all discharges. However, a likely drop in the tailwater level by about 1m may result in an afflux of 0.5m for design discharge.

5) Low Froude number basin with baffle blocks in addition to chute blocks and end sills functions well to stabilize the hydraulic jump within the stilling basin and to keep the jump position within the glacis slope. However, the adopted dimensions of the basin blocks may be reviewed for optimization of design.

6) The block protection upstream and downstream of the main spillway as designed remains intact for very high (higher than design discharge) discharges.

7) For a discharge intensity of 11.11 m^2/s and tailwater level of +2.0 mPWD about 0.92 m gate opening will be needed for maintaining pond water level of 12.5 mPWD.

8) Under prevailing tailwater condition the sweep out of hydraulic jump is unlikely to occur

ACKNOWLEDGEMENT

The authors are grateful to the Bangladesh Water Development board (BWDB) for financial support and Development Design Consultants Ltd. and Associates for providing necessary data, design drawings and feedback to conduct the study.

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TEMPORAL AND SPATIAL DISTRIBUTION OF SUSPENDED SEDIMENT CONCENTRATION IN BHAIRAB RIVER

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ABSTRACT

In this study the water samples were collected from different depth and width wise locations in Shiromoni and Raligate Point of Bhairab River. Using a dataset up to twelve months long in the Bhairab River, the variability of suspended sediment concentration at different temporal scales (monthly) is analyzed. The water samples were collected in three depth wise and three width wise sections. So, each month nine water samples were collected in each point. After collecting the water samples, its P^H, salinity and suspended sediment concentration were determined. The depth wise variation of suspended sediment concentration was fitted with theoretical standard distributions and the unknown coefficients are determined and compared with that of Jamuna River in Bangladesh.

Keywords: *Suspended Sediment Concentration, Bhairab River, Salinity, High tide and Low tide period.*

1. INTRODUCTION

Bangladesh is a land of rivers. Country's most of the land is formed through silt brought by the many rivers that flow through it. Khulna is a port city of Bangladesh surrounded by several rivers. The Bhairab River on northern side, Rupsha River in the middle part and Pasur on the southern side flow along eastern margin of the city, and Mayur on the northern and Hatia River on the southern side flow along the western side of the city. The life in Khulna largely depends on waterways. Our study location is situated at Shiromoni and Raligate Point of Bhairab River in Khulna city. Bhairab River is one of the tidal rivers in Bangladesh, which is the branch of Gorai River that originated from Ganga River. In the region of Rupsha in Khulna city, its name is Rupsha River. The flow of Bhairab River depends on upstream flows comes from Gorai River which plays an important role on its suspended sediment concentration. The characteristic of a river depends on the sediment concentration. The flow path of the river changes due to excessive sediment, and also erosion of the river occurs due to excessive sediment transport. The transport of salinity and P^H are important elements to determine the water quality in a river. The objectives of the study were to determine the variation of suspended sediment concentration along depth and width wise directions and to determine the temporal variation of suspended sediment concentration. The temporal variation of salinity and p^H were also studied. The measured SSC were fitted with the available theoretical standard distributions and the coefficients in the equations are compared with that of Jamuna River.

2. EXPERIMENTAL TECHNIQUES

Water samples were collected by locally made sampler. After collecting sample water, suspended sediment concentration, P^H, and salinity were determined in the laboratory.

2.1 Water Samples Collection

Along the depth wise direction, the water samples were collected in three sections: from just below the surface, at 1/3rd of river depth and at 2/3rd of river depth below the surface. Along the width wise direction, samples were collected in three points in each horizontal section and they are at 1/4th of river width, at 1/2 of river width and at 3/ 4th of river width. Sample collecting points are shown in Figure 1. The horizontal sections are denoted as HI, H2, and H3 and vertical sections are denoted as V1, V2 and V3 respectively.

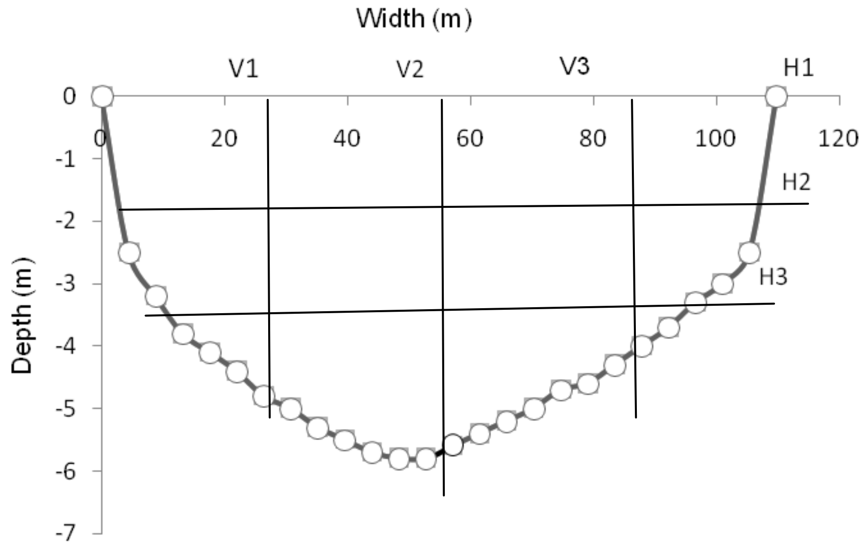

Figure 1: Measured cross-section of Bhairab River at Shiromoni point (low tide period, Dec. 14, 2010)

Table 1: Summary of water quality test of collected water samples at different months.

Month	Collection of water sample	SSC	Salinity	P^H	Remarks
June 2010	Yes	Yes	Yes	Yes	High tide
July 2010	Yes	Yes	Yes	Yes	High tide
August 2010	Yes	Yes	Yes	Yes	Low tide
September 2010	One point	Yes	No	No	High tide
October 2010	Yes	Yes	Yes	Yes	High tide
November 2010	Yes	Yes	Yes	Yes	High tide
December 2010	Yes	Yes	Yes	Yes	Low tide
January 2011	Yes	Yes	Yes	Yes	Low tide
February 2011	One point	Yes	Yes	Yes	High tide & Low Tide
March 2011	Yes	Yes	Yes	Yes	High Tide
April 2011	Yes	Yes	Yes	Yes	High Tide

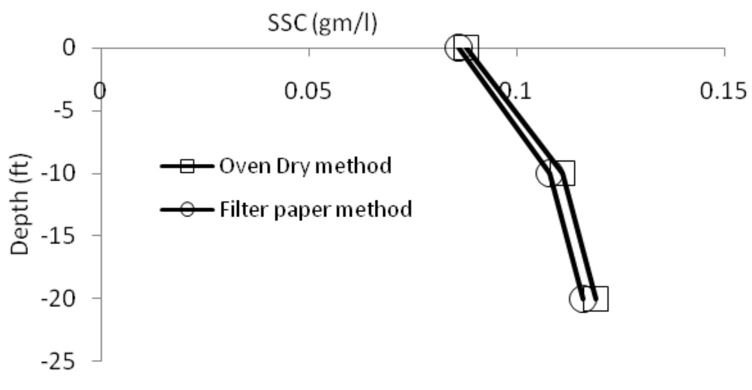

Figure 2: Comparison of SSC by oven dry and filter paper method

2.2 Laboratory Tests

After collecting the water its pH and salinity were determined. Then the suspended sediment concentrations of the samples were measured by filter paper method. At first blank filter paper was being weighed. Then water samples were filtered and then the filter paper were dried and weighed again. The difference of these two weights was the weight of suspended sediment concentration. In the oven dry method, the samples of water with suspended sediments were kept in an oven for 2 days at a temperature of 100-120ºC and the settled sediments were decanted and weighed using electronic balance. The details of water sample collection and the name of performed laboratory tests are presented in Table 1.

To assess the authenticity of results obtained by filter paper method, the SSC for three water samples in different depths were measured by both filter paper method and oven dry method. Figure 2 shows the comparison of measured SSC between oven dry method and filter paper method. The difference of the two results is negligible. Reasonably, the amount of SSC of oven dry method is slightly (about 1.9%) greater than that of filter paper method.

3. RESULTS AND DISCUSSION

3.1 Temporal Variation of Suspended Sediment Concentration

Figure 3 shows the temporal variation of suspended sediment concentration. It shows that the SSC is higher during June to September and low during October to March. Therefore, the suspended sediment concentration (SSC) of Bhairab River in rainy season is higher than dry season. In rainy season water comes in Bhairab River from upstream Gorai River which carries a lot of sediment particles. On the other hand rain water from catchment area also carries a lot of sediment. So in rainy season suspended sediment concentration is higher. But in dry season SSC is lower, because there is no upstream water flow and no rainfall in catchment area. In rainy season the SSC in Raligate and Shiromoni point are found as 0.945 gm/l and 0.982 gm/l respectively. In dry season the SSC in Raligate point and Shiromoni point are found as 0.534 gm/l and 0.571 gm/l respectively.

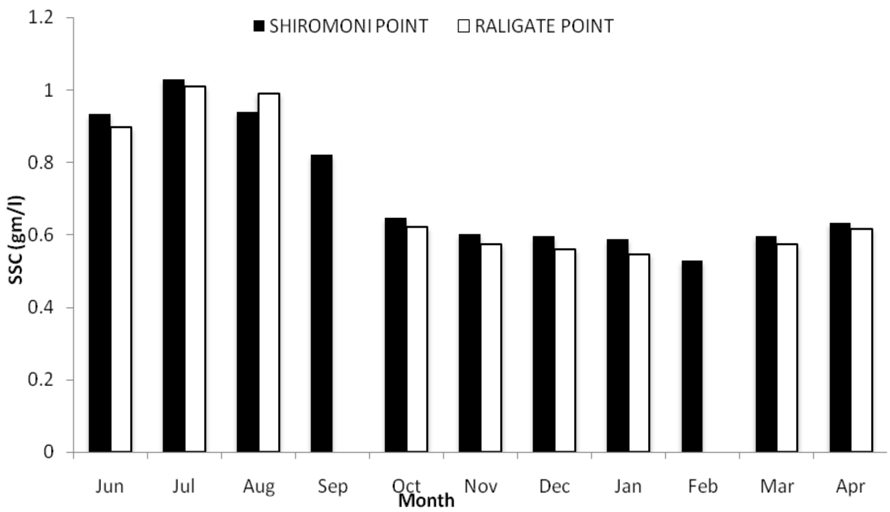

Figure 3: Temporal variation of suspended sediment concentration

3.2 Temporal Variation of Salinity

Figure 4 shows the temporal variation of salinity. It shows that the salinity is low in rainy season from June to September and high in dry season from October to April. In comparison to previous figure, it is observed that when the SSC is high the salinity is low and when SSC is low salinity is high. In other words, salinity is lower in rainy season and higher in dry season. Average SSC during rainy season at Shiromoni and Raligate point are found as 1.55 gm/l, and 1.525 gm/l respectively. During dry season average SSC at Shiromoni and Raligate point are found as 4.3 gm/l and 4.15 gm/l respectively.

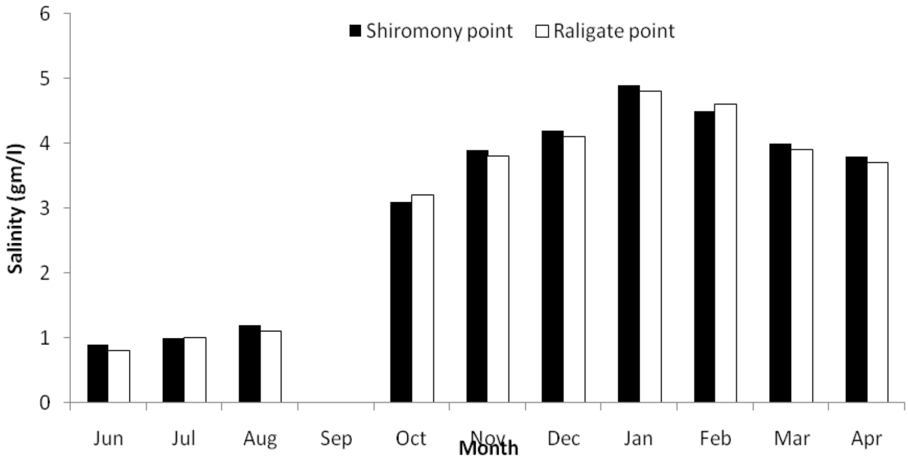

Figure 4: Temporal variation of salinity

3.3 Temporal Variation of P^H

Figure 5 shows the temporal variation of pH in the water of Bhairab river at Raligate and Shiromoni point. The pH distribution of the river water showed that in dry season the water is slightly alkaline. High pH values of 8.0-8.2 were recorded during the dry season while pH dropped to values of 7.3-7.7 during the rainy season. This drop in pH from 8.2 to 7.3 was probably due to stirring effect of the incoming flood and rain water from the rivers and streams.

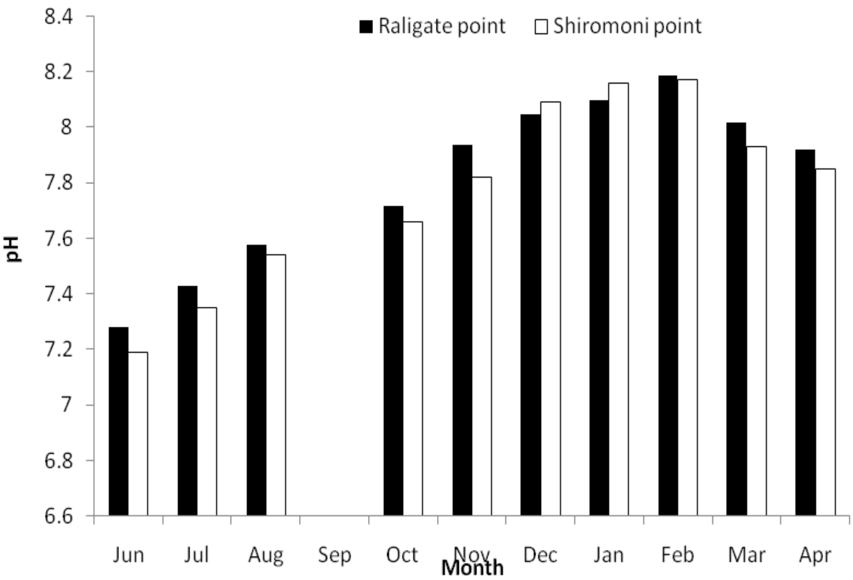

Figure 5: Temporal variation of pH in the water of Bhairab river at Raligate and Shiromoni point.

3.4 Spatial Distribution of SSC (Depth Wise Direction)

Figure 6 illustrates spatial distribution of the seasonally averaged SSC along depth wise direction in rainy season and dry season in the year of 2011 at Shiromoni point. The experimental data revealed that the variation of suspended sediment concentration increases with depth from surface. That means, if the vertical depth from water surface increases then the SSC also increases. Every month shows same pattern of variation. Although the SSC in dry season is lower than the rainy season, they showed similar variation with depth as two profiles are seen to be nearly parallel to each other.

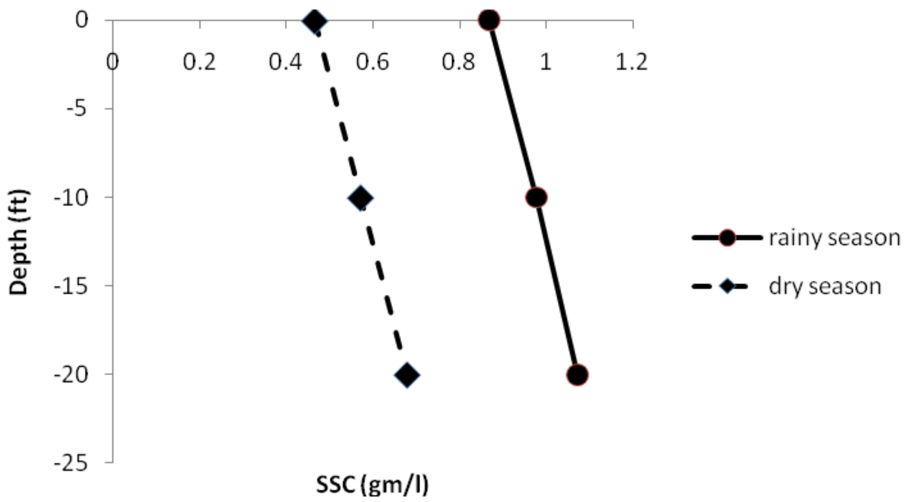

Figure 6: Depth-wise distribution of the seasonally averaged SSC for rainy and dry season in the year of 2011 at Shiromoni point of Bhairab river.

3.5 Spatial Distribution of SSC (width wise direction)

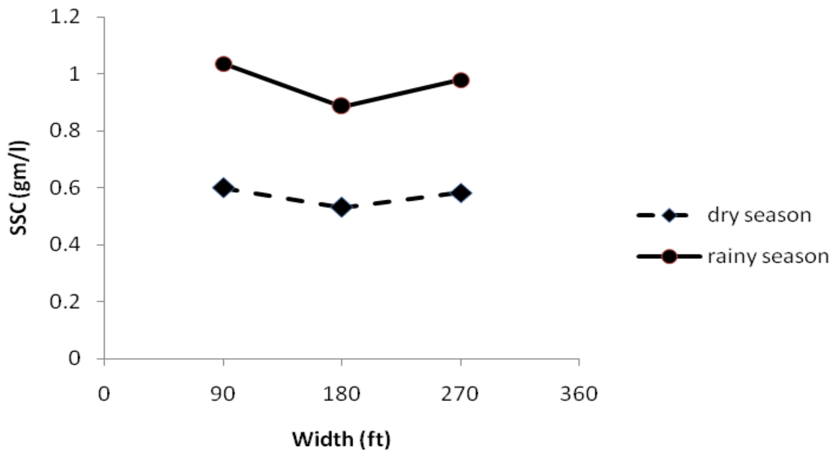

Figure 7: Width-wise distribution of the seasonally averaged SSC for rainy and dry season in the year of 2011 at Shiromoni point of Bhairab river

Figures 7 illustrates the average spatial distribution of SSC along width wise direction in rainy season and dry season in the year of 2011at Shiromoni point of Bhairab river. From observed data it is seen that the suspended sediment concentration at middle of the river is lower than two sides of the river for all the horizontal section. This type of variation is also observed in depth wise direction. The velocity of water in the middle of the river is higher than the bank of the river. On the other hand turbidity is higher near the bank of the river and relatively fresh water can be found in the middle of the river.

3.6 Variation of P^H and Salinity with Suspended Sediment Concentration

Figure 8 shows the variation of pH with Suspended Sediment Concentration. It is observed that when pH is higher, then SSC is low and when pH is low, then SSC is higher. So pH shows an inverse relationship with SSC.

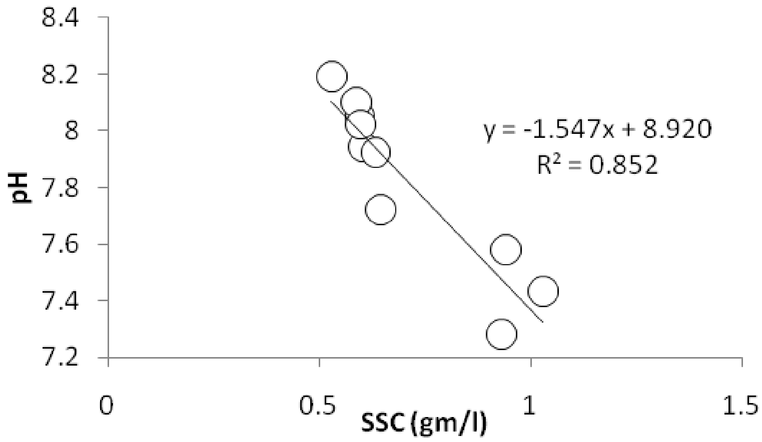

Figure 8: Variation of pH with Suspended Sediment Concentration

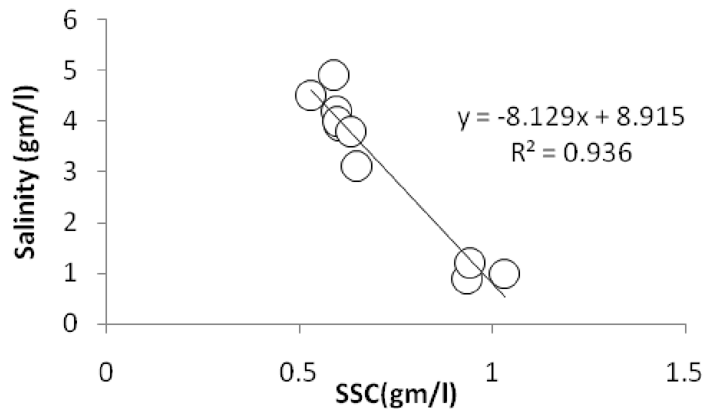

Figure 9: Variation of Salinity with Suspended Sediment Concentration

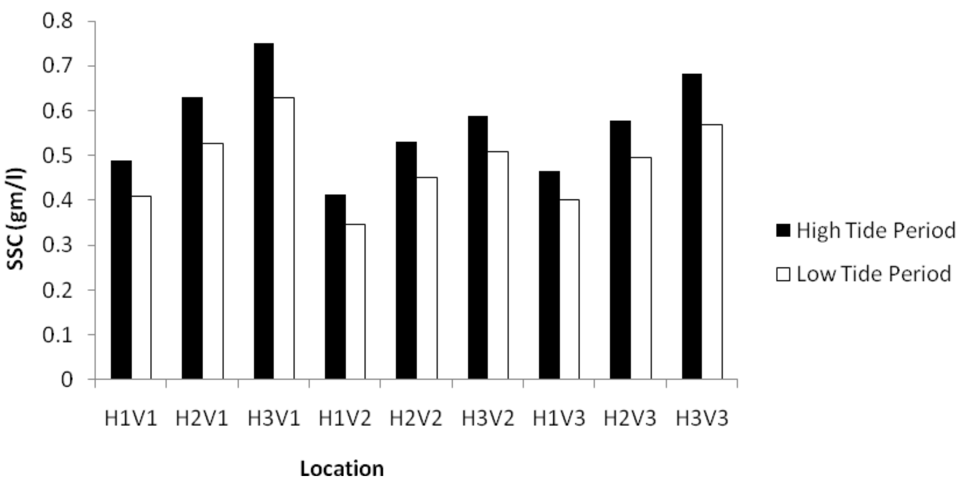

Figure 10: Variation of suspended sediment concentration in high tide and low tide period at Raligate point in February 23, 2011.

Figure 9 shows the variation of Salinity with Suspended Sediment Concentration. It is observed that when salinity is high, then SSC is low and when salinity is low, then SSC is high. So salinity has also an inverse relationship with suspended sediment concentration.

3.7 Comparison of SSC with High Tide & Low Tide

Figure 10 shows the comparison of SSC with High Tide and Low Tide period in February 2011 at Raligate point of Bhairab river. In comparison of SSC between high tide and low tide period it is observed that the average SSC in high tide period is 0.573gm/l and in low tide period is 0.518gm/l. It is found that the SSC in high tide period is 9.63% greater than that of low tide period.

3.8 Correlation of Experimental SSC with Theoretical Standard Distribution

Theoretical distribution of suspended sediment concentration in a river can be written in the form of equation (1) and equation (2) as shown below,

$$C = Ca \times e^{\left(-\left(\frac{w_0}{\varepsilon_s}\right)\times(y-a)\right)} \quad (1)$$

$$\frac{C}{C_a} = \left(\frac{a(d-a)}{y(d-a)}\right)^z \quad (2)$$

The parameters in the equations are defined in Figure 11. Here, C indicates suspended sediment concentration at any depth y from river bed (mg/l), ‘a’ is the depth from river bed where upward and downward sediment movement is in equilibrium (m), ‘Ca’ is sediment concentration at depth ‘a’ from river bed (mg/l), W_0 is Fall Velocity (m/s) and ε_s is Sediment diffusion Coefficient for the diffusion along any distances. In this study, the measured distribution of SSC is compared with equation (1) and (2), and the unknown coefficients a, Ca, w_0/ε_s and z are determined for Bhairab River.

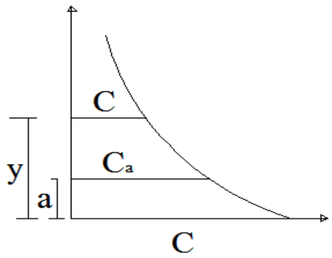

Figure 11: Sketch of typical SSC profile along vertical direction

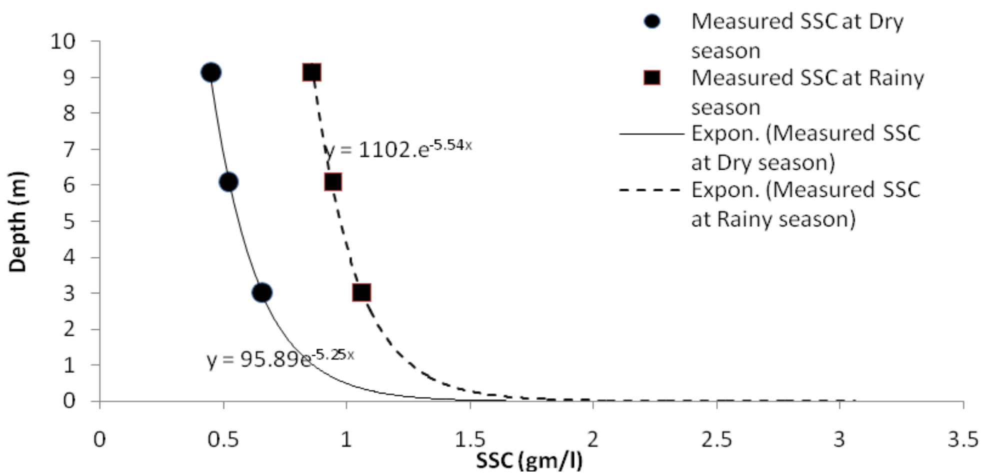

Figure 12: Vertical distribution of measured SSC for Bhairab River with exponential trend line

3.8.1 Calculation of C_a for Equation (1) & (2) from fitted experimental profile

Figure 12 shows the vertical distribution of measured suspended sediment concentration for dry season and rainy season. Here each profile is fitted with exponential trend line from which C_a is calculated. From the trend line for any value of y (=a), the value of SSC (= C_a) is calculated.

3.8.2 Comparison of depth wise distribution of SSC

Figure 13 shows the correlation of measured suspended sediment concentration at rainy season and dry season with theoretical standard distribution (equation 1). Comparing theoretical distribution with measured suspended sediment concentration, we calculated the value of coefficient w_0/ε_s as 0.035 and 0.089 for rainy season and dry season respectively. From this graph we can calculate the value of suspended sediment concentration for any depth of any point of Bhairab River without collecting water sample.

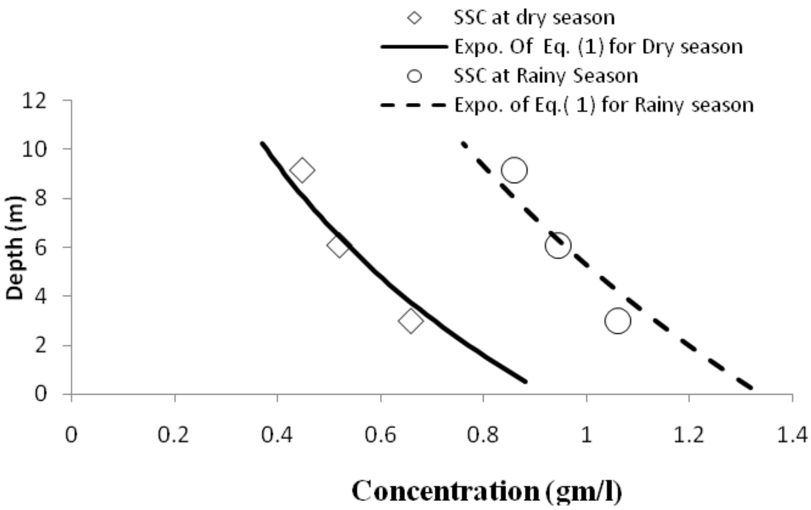

Figure 13: Correlation of measured SSC with theoretical standard distribution (Equation 1)

Table 2: Calculated coefficient of equation (1) and (2) for Bhairab River

Season of the year	**a(m)**	**Ca(gm/l)**	**w_0/ε_s**
Rainy Season	2	1.10	0.035
Dry Season	1.30	0.82	0.089

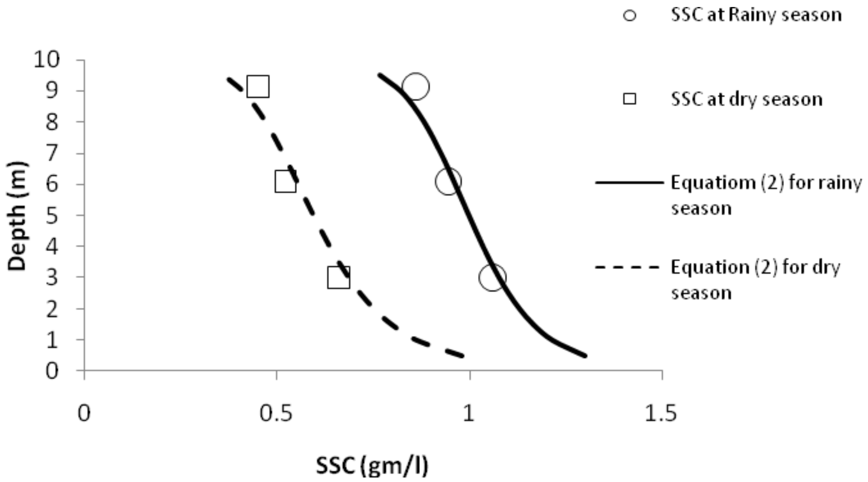

Figure 14: Correlation of experimental SSC with theoretical standard distribution (equation 2)

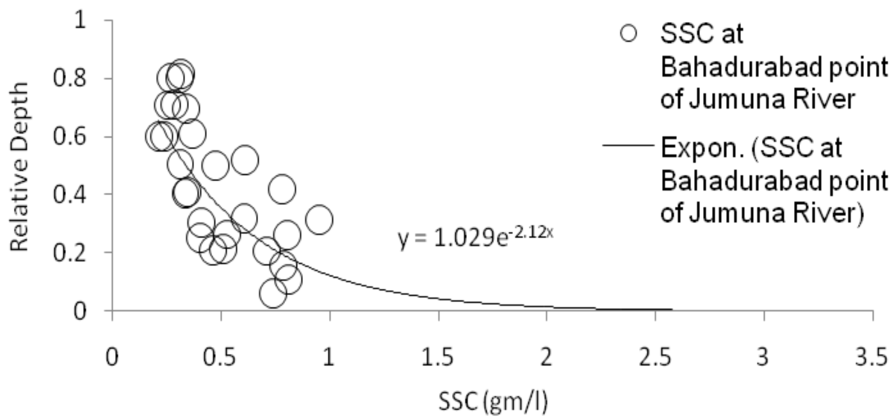

Figure 15: Vertical distribution of measured SSC for Jamuna River (Sarkar and Klaassen, 2011) with exponential trend line

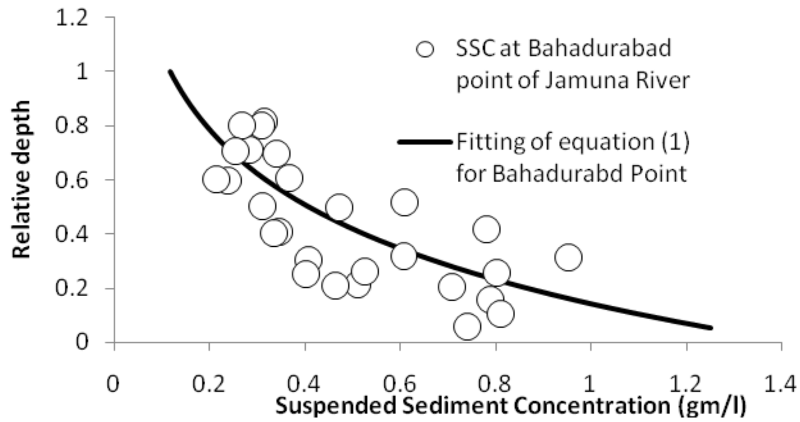

Figure 16: Correlation of suspended sediment of Jamuna River with theoretical standard distribution (Equation 1)

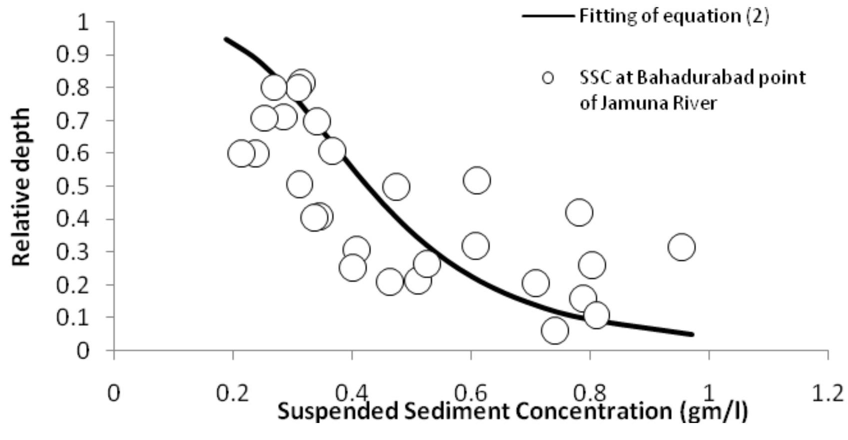

Figure 17: Correlation of SSC of Jamuna River with Theoretical Standard distribution (equation 2)

3.8.3 Calculation of exponent Z in Equation (2)

Figure 14 shows the Correlation of experimental SSC with theoretical standard distribution (equation 2). Using the same procedure as described before, the value of Z in (equation 2) is estimated both for dry and rainy season, and their values are found as 0.09 and 0.17 respectively.

3.9 Correlation of Measured SSC of Jamuna River with Theoretical Distributions

Figure 15 shows the distribution of measured SSC for Jamuna river (Sarker and Klassen, 2011) with exponential trend line and Figure 16 shows the correlation of suspended sediment concentration of Jamuna river at Bahadurabad point with theoretical standard distribution (Equation 1).

Using the same procedure as described before, the value of Z in equation (2) is estimated for Jamuna River and the value is estimated at 0.28 (Figure 17). Table 3 shows the comparison of coefficients of equation (1) and (2) for Bhairab River with that of Jamuna River. It is observed that the value of SSC in Bhairab River is comparable with that of Jamuna River.

Table 3: Comparison of coefficients of equation (1) and (2) for Bhairab River with that of Jamuna River

River	Season	SSC(gm/l)	a(m)	C_a(gm/l)	w/ε	Z
Bhairab River	Rainy Season	0.955	2	1.10	0.035	0.17
	Dry Season	0.543	1.3	0.82	0.089	0.09
Jamuna River	Dry Season	0.484	0.05	1.40	0.25	0.28

4. CONCLUSIONS

We have collected and tested the water samples from Bhairab River for various depths and width wise locations. From the experimental results, following conclusions can be made:

- The suspended sediment is varied temporally as well as spatially.
- The suspended sediment during rainy season is higher compared to dry season. In rainy season suspended sediment is higher due to upstream flow and runoff due to rain, which carry a lot of sediments.
- Seasonal variation of P^H in the river water is considerable; and the value of pH is found lower during rainy season and higher during dry season.
- In comparison of SSC between high tide and low tide period, it is observed that the SSC in high tide period is about 10% higher than that of in low tide period.
- The variation of suspended sediment concentration is found to be increased with depth and the suspended sediment concentration at middle of the river is found lower than two sides of the river.
- The experimental data are fitted with standard theoretical distribution of SSC. The coefficients in the equations of standard distribution are estimated for Bhairab River. The experimental results of SSC are found to be comparable with that of other rivers (such as Jamuna) in Bangladesh. Using the estimated values of coefficients, the value of suspended sediment concentration for any depth of Bhairab River can be calculated without performing the experiments.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECTS OF STREAM BARB ON STRAIGHT CHANNEL BED CONFIGURATION: NUMERICAL SIMULATION

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ABSTRACT

The focus of this study is to observe the effect of barbs on channel bed configuration by using iRIC-Nays2D, a multi-dimensional computational model for river flow and riverbed variation analysis solver package developed by Hokkaido University, Japan. The two-dimensional (2D) numerical simulation is represents the effects the pre-existing condition (i.e. before construction of barbs) and in the presence of the barbs installed on sidewall of the channel. The average flow depth, and discharge were used as 0.044 m and 0.028 m^3/s, respectively. The numerical simulation reproduced the features of barbs with time at upward side and creates scour holes at the leeward side of the structure. The simulation results confirmed that the vicinity of the barb head achieved a substantial reduction in scour, increase stream-wise velocity and sediment deposition at near bank. Bars were formed along the bank-line at the upstream portion of the structures. On the other hand, at the downstream side near the barb head scour occurs due to hydraulic jump and turbulence flow mixing.

Keywords: Stream barbs, bed configuration, numerical simulation, bars, turbulence flow.

1. INTRODUCTION

The changes of river are interrelated not only with bank erosion but also with bed deformation. This is because of the relationship between water flow and sediment transport. Beside of bank erosion it is needed to study about the bed deformation for maintaining river ecosystems and landscape in the field for environmental engineering purpose.

Stream barbs are relatively a new form of bank protection structure. It is defined as wide crested trapezoidal-shaped structures that project out from the stream bank into the main channel flow to modify flow patterns and bed topography. Typically, constructed of large angular rock (riprap); barbs protrude into the flow at an angle upstream to the channel sidewall (bank) for the purpose of deflecting current away from the bank and minimizing the erosion potential (Ghodsian and Tehrani, 2001; Breusers and Raudkivi, 1991). Barbs are quite similar to spur dikes, groynes (groins), and submerged vanes; they have some distinct features as shown in Figure 1. The most defining feature is the trapezoidal shape of the structure with inclined sides and a wide sloped crest, which allows the barb to behave as a partially submerged structure (weir) when flow is low and fully submerged when bankfull flow conditions are present. When pointed upstream the submerged weir section forces the water flowing over the structure into a hydraulic jump (Fox, 2002). The flow separation induced by the hydraulic jump promotes the formation of eddies and sediment deposition on the leeward side of the barb (Lloyd and Stansby, 1997). Generally, barbs are used to protect banks for gentle (wide radius) meanders, or relatively straight banks. The primary function of barbs is to deflect the strong main flow away from critical zones towards the channel center and therefore, prevent erosion of banks (Hansen and Winter 1996). It also used on long reaches of relatively straight or gently curving banks that need protection. The approaches to protect stream banks from erosion during periods of high flows including various bio-technical type channel revetments (Schiechtl and Stern, 1997) and groin like structure including bend-way weirs (Davinroy, 1990, 1994) and barbs (USDA, 2001). While bend-way weirs and barbs are design primarily to protect unstable outside bank zones of channel bend from the bank-undermining and local erosion process associated with bend secondary currents. As well as providing bank protection, these structures promote vegetated stream banks, create resting pools and scour holes for fish habitat, and increase bio-diversity for aquatic species.

Fig.1. Photographic view shows the series of stream barbs installed along the riverbank during flow season (left side) and low flow season (write side). The red circle indicates sand bars formed after installation of barbs.

Barbs have been used by the Natural Resources conservation Service (NRCS) of US department of agriculture, in Oregon for river and stream bank protection since the late 1980. The Washington State Department of Transportation (WSDOT) has employed barbs for bank protection along highways or river crossings and/or to improve aquatic conditions, especially in shallow gravel bed streams (Papanicolaou et al. 2004).

Study of channel bed configuration has become crucial for examination of the river environment in recent years, because the channel bed is the foundation of the river eco-system. Barbs often perform poorly in strongly aggrading or degrading channels. According to Chow (1959), barbs are a trapezoidal shaped rock structure, which extends into the main flow of the river. In view of the fact that barbs tend to redirect water to the center of the stream, they encourage deposition between the barbs along the bank. When barb designed effectively, as a result the structure induced a zone of subcritical flow at the upstream and along the channel bank. Water level increases in the zone of upstream of the barb through the backwater as described by Chow (1959). The upstream progression of subcritical reaches flowing slowly, the water loses some of its capacity to carry load and sand or silt deposits on the riverbed, building it up or aggrading it as well as sand bars are formed. In the near bank region its control erosion and ultimately leads to deposition of sediment along the protected bank line. Flow across the barb occurs cross-channel accelerations being forced around the emerging bars, however tend to decrease bottom shear stress higher up on the bars. As a result the bottom shear stresses decrease over the shoaling bar head and particles carried onto the bar head tend to be deposited (Lisle, 1991). Bars developed rapidly due to some causes such as initial bed condition, inlet flow and sand supply and propagate (increase in the bars wavelength with time) new bars to downstream one by one just like a chain-reaction as stated Fujita and Muramotu (1985). Barbs influence near bank velocity and shear stress distribution through disruption of helicoidal currents and partial interception of cross-stream flow. Flow across the barb occurs somewhat normal to the longitudinal axis of each structure and intersects the contraction-accelerated discharge at each barb end. The convergence of these flow components results in energy dissipation through turbulent flow mixing and forces the resultant vector flow direction away from the protected bank (NRCS, 2005).

The channel without any training structure, Mosselman (2009) studied by theoretical analysis and Crosato and Desta (2009) by their experimental observation, on current accepted view is that the bars formed as steady only if the width to depth ratio is at the value of resonance or if the bars are forced by steady local perturbation. The steady bars can also form by small-amplitude unsteadiness in the system. As the width to depth ratio increases, the strongly nonlinear competition between different modes not only affects significantly bars height and celerity but also enhance the selection of higher-mode configurations namely, central bar mode or multiple row bar mode (Colombini and Tubino, 1991).

There are various researches that has been done for flows around hydraulic structures like spur dikes, barbs, groins, submerged vanes etc., the majority of the investigations have treated flow around these structures and prevent erosion of the bank. Kothyari and Raju (2001) reported that, the primary focus of previous investigations, such as, Gill (1972), Garde et al. (1961) and Ahmad (1953), on the study of scour at abutments and spur dikes was to predict an equilibrium or maximum design scour depth. Barbs induced energy re-distribution away from the outer bank towards the center of the channel results in scour near the ends of the barbs and realignment of the thalweg. Field observation and laboratory result showed by Johnson, et al. (2001), Matsuura and Townsend (2004), Kuhnle et. al. (2002) that, the scour depth occurs at the barb end and immediately downstream of the structure. Barbs are currently undergoing limited field test on selected bend of several shallow wide streams in Illions, USA (Matsuura and Townsend, 2004). Moreover, investigation and estimation of the depth of local scour and deposition around a structure, such as a barb, remains a perplex problem for hydraulic and restoration engineers. Most investigations have just reported measurements of the maximum depth of scour without giving any information about the geometry (i.e. shape) of the scour hole and how this geometry changes with time (Kuhnle et al., 2002). Furthermore, it is need to investigate the effects of barbs on bed configuration considering uniform flow condition. This paper reports two-dimensional numerical simulation to observe the effects of barb on channel bed evolution from pre-exist straight channel and after install stream barb with erodible bed among a relatively high width-to-depth ratio.

2. METHODOLOGY

A 13.3 m long and 0.8 m wide grid channel generated by using iRIC-Nays2D, computation flow model, a multi-dimensional river flow and riverbed variation analysis solver package developed by Hokkaido University, Japan. Several numerical models have been developed to simulate the deformation of a relatively narrow, deep, straight channel with erodible, noncohesive banks in the laboratory (Shimizu 2002). The channel shape was considered as rectangular flat movable bed with simple cross section. The slope of the channel was used as 0.005, as the model is designed by subcritical flow. Grid cells were generated along the longitudinal and lateral direction of hundred and ten numbers, respectively. It was estimated that the change of width of the grid channel in the direction of flow is constant. To evaluate the barbs hydraulic performance in channel bed configuration, the model was run two settings: first, in the pre-existing condition (i.e. before constructing the barbs) second, in the presence of the barbs installed on sidewall of the channel. At first flow approached into the channel pre-existing condition and find the velocity vector, flow depth and the elevation of the bed topography, respectively. Before starting flow provided small perturbation value at 5 selected grid nodes that are persuade the bars formation. The elevation set as zero along the whole channel alternately around 2.5 m equal distance. The barb structure is installed as obstacle at one channel sidewall and it is 4.5 m apart from the upstream end of the channel as shown in Fig.2a and 2b. The length of the barb considered as 24 cm, one-third of the cross-section top width at bankfull stage and build as rectangular riprap shape. At the end of barb the nodes elevation also decreased for creates its sloping crest. According to NRCS (2005), each barb extends from the bank-line to the proposed thalweg location. Generally the length of the barb not exceed one-third of the cross-section top width at bankfull stage, as Matsuura and Townsend (2004) observed in their study that, the overall length should not exceed about one quarter the channel-forming flow width. By decreasing the nodes elevation point at the barb end, the structure set as a sloping ridge from the bank to the main channel flow.

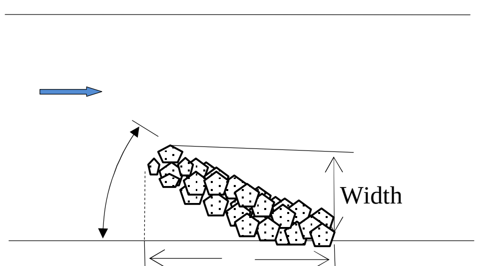

Fig. 2a. Sketch of the barb prototype

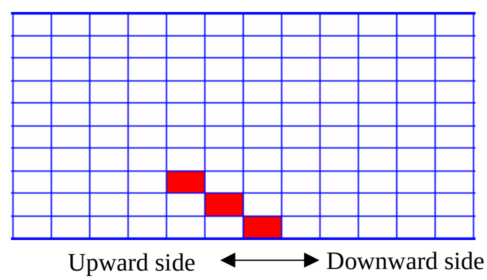

Fig.2b. Riprap barb structure setting on grid channel bank sidewall

The barb placed at an angle of 45^0 with the upstream bank of the channel as NRCS (2005) state the horizontal angle between the tangent line placed along the upstream bank and the centerline of the longitudinal axis of the barb have varied from 30^0 to 60^0.

The generated grid channel was used to set the calculation condition and simulation. The table 1. below summarize the flow calculation condition for simulation. The symbol L represents as the total length, B is the internal width and S is the slope of the channel, D is the normal flow depth, Q is the discharge, U is the velocity of the flow, B/D is the width to average flow depth ratio, C_f is the roughness coefficient correspond to d_{50}, where d_{50} is the mean sediment diameter and Froude number, $F_r = \frac{U}{\sqrt{gD}}$, and g is gravity acceleration. The friction coefficient C_f is found to be expressed as follow:

$$C_f = \left[\frac{1}{K} ln \frac{11D}{2.5 d_s}\right]^{-2} \qquad (1)$$

The bottom share stress from the Reynolds equation, $\tau = \rho g D S$ and from the Keulegan relation for turbulent flow, the bed shear stress, $\tau = \rho C_f U^2$. Where ρ is the density of water. Considering unit flow depth into the channel then, $U = Q/D$. Using from the relation $\tau = \rho g D S = \rho C_f U^2$ the discharge and velocity is calculated from the equation as follows:

$$\frac{Q}{D} = \sqrt{\frac{gDS}{C_f}} \qquad (2)$$

Table. 1. Details flow calculation condition

L	B	S	B/D	D	Q	U	C_f	Fr	d_{50}
m	m	m/m	-	m	m^3/s	m/s	-	-	mm
13.3	0.8	0.005	18	0.044	0.028	0.64	0.005423	0.977	0.77

In the calculation condition used the professional solver type with bed deformation. The finite differential methods of advection term was used as CIP method. For calculation of numerical simulation the cyclic boundary condition was used with initial water surface as uniform flow. The upstream velocity and water surface at downstream was also considered as uniform flow. Constant discharge time series were used from the beginning up to 5 hours (Table 1). Output time interval was select as 10 sec. The calculation time step was set according to Courant–Friedrichs–Lewy (CFL) condition as stated in equation (3). The CFL condition is necessary for convergence of flow while solving certain partial differential equations numerically by the method of finite differences (Courant et al. 1956).

$$\frac{\Delta t}{\Delta x/u} \leq C \qquad \Rightarrow \Delta t = C * \Delta x/u \qquad (3)$$

Where C is a dimensionless constant and it's vary from $0.1 \sim 0.3$. The maximum iteration times of water surface calculation was set as 20. The diameter of the bed material was used as 0.77 mm and bed roughness calculated from the bed materials using equation (1).

3. RESULTS AND DISCUSSIONS

The result of the flow fields and bed-topographies are presented in this section to understand the morphodynamics resulted from dealings of two settings: in before constructing the barbs and in the presence of the barbs. In order to compare these results of pre-existing condition and presence of barb there are three forms of result are considered in this numerical simulation. The difference between the results of two settings was considered to be the effects of barb on the channel bed configuration. The velocity profile was considered to recognize the flow pattern, bed elevation for identify the sediment deposition and flow depth to know the scour depth from both arrangements. Figures 3 to 9 comparatively illustrated the effects of barbs on velocity vector, bed elevation, water flow depth.

3.1 Velocity field

The velocity vectors at water surface were obtained from simulation for pre-existing condition (without barbs) and with single barbs, respectively. Fig. 3 (a) and (b) shows the simulation result of velocity vector without barb structure after 1 hour and 5 hours, respectively. When the flow approaches into the channel without barbs, the flow towards the channel and creates parabolic flow pattern that is influences to formation of double row bars. The flow pattern is not change into entire channel after long time simulation. Fig. 4 (a) and (b) shows the simulation effect of velocity vector at the water surface by installing single barb after 1 hour and 5 hours, respectively. In that case due to the flow separation at barb head return current developed at the upstream side induced zone of subcritical flow and along the stream bank. Barb influenced the flow at downstream side and creates a mixing zone just behind the structure along the near bank line. The flow across the structure occurs contraction-accelerated discharge at the barb end. The convergence of these flow components result turbulent mixing around the barb head and vector flow directed towards the outer bank near water surface.

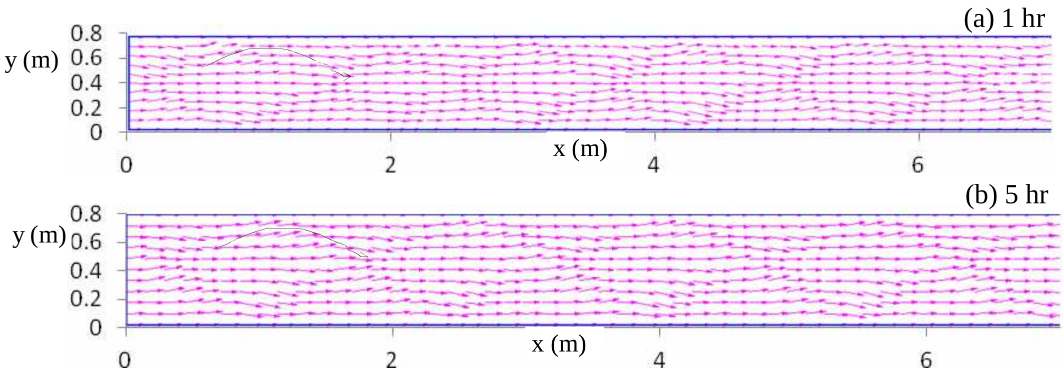

Fig.3. Velocity field simulation result at water surface into the channel without barb (a) after 1 hour (b) after 5 hour.

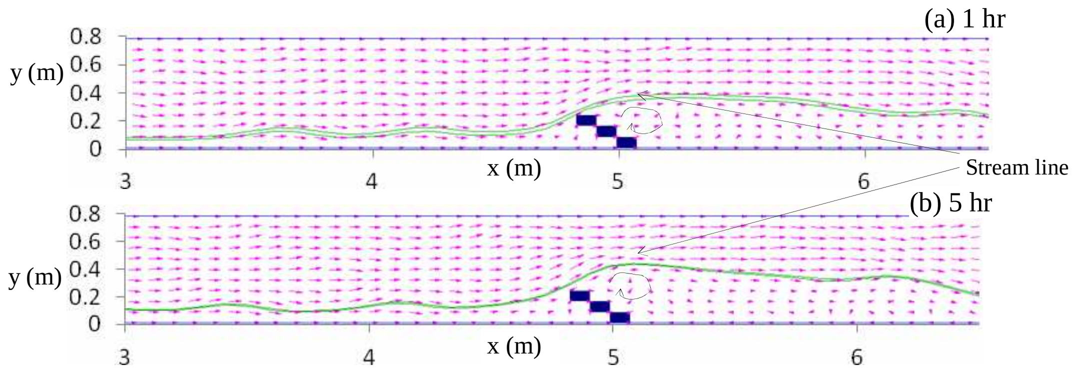

Fig.4. Velocity field with a barb installed at one sidewall of the channel (a) after 1 hour simulation (b) after 5 hour simulation.

3.2 Bed Elevation Around the Atructure

The flow patterns with sediment transport were persuaded to build the scour-hole and deposition. In general, the flow initiates and controls the scours and deposition. Fig. 5 shows the simulation result with the depositional contour lines of pre-existing channel after 1 hour and 5 hours, respectively. In pre-existing channel double row bars are formed due to flow and these formation of bars are becomes more perceptible after 5 hours simulation. Fig. 6 shows the simulation result with the depositional contour lines among single barb after 1 hour and 5 hours, respectively. Deposition occurred as sand bars along the bank-line at the upstream side of the structures. The formation of bars changing trend from double row to alternate due to long time simulation on upstream side of the barb field. Because of potential energy increased in the zone of upstream through backwaters effects. The upstream progression of subcritical reaches in the near bank region controls erosion and ultimately leads to deposition of sediments along the bank-line. In addition, reduction of near bank velocity gradient through the subcritical backwater effect promotes sediment deposition upstream of the barb structure. At the downstream side of barb especially near the barb, head scour occurs due to hydraulic jump and turbulence from flow mixing. The scour depth occurs at the barb end and immediately downstream of the structure because of energy re-distribution away from the outer bank towards the center of the channel results in scour near the ends of the barbs (Matsuura and Townsend (2004). This scour result is from contraction flow acceleration by local reduction of width to depth ratio. The remark was that sediment were scoured in the downward portion of the barb and deposited in the upward side of the barb. Fig. 7 represents the bed elevation of pre-exist channel and after barb installation at different times. This result shows clearly the applicability of barbs reduced the erosion around the structure and increases the pool habitat near the thalweg, which can be useful for natural rivers training work.

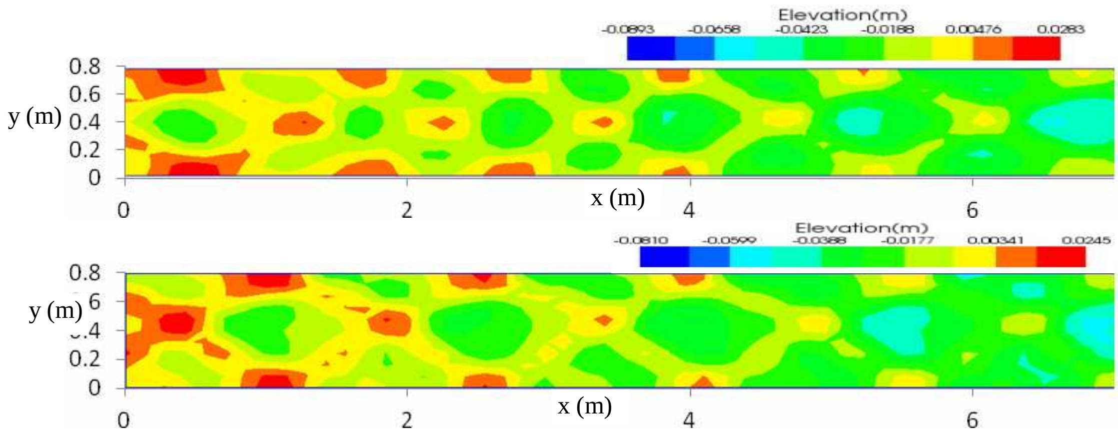

Fig.5. Bed level contour result of the channel without any structure (top: after 1 hour simulation, bottom: after 5 hour simulation).

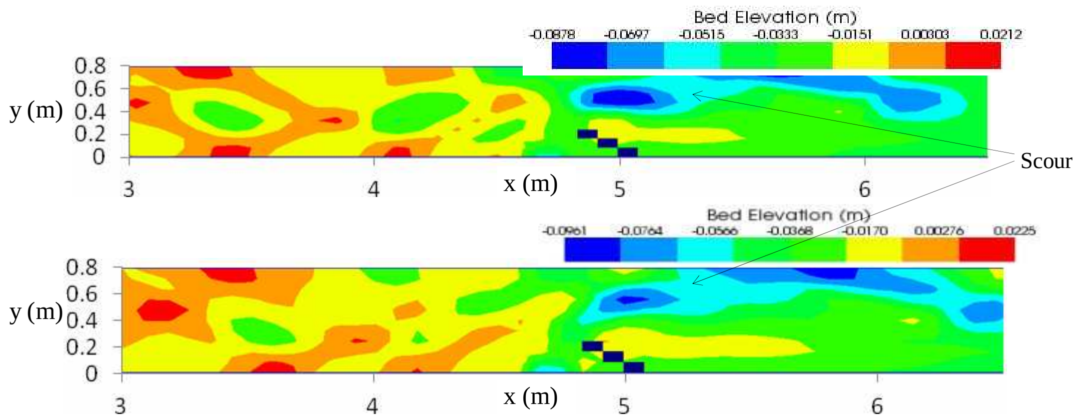

Fig.6. Bed level contour result of the channel with a barb installed at one sidewall of the channel (top: after 1 hour simulation, bottom: after 5 hour simulation).

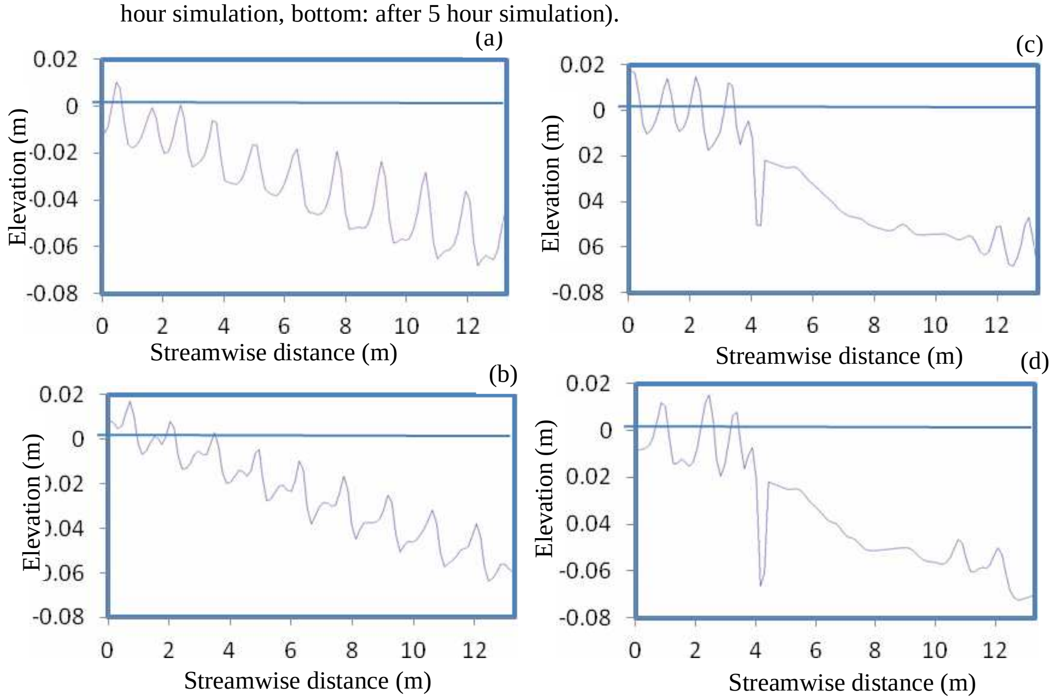

Fig.7. Evolution of bed profile & Comparison with pre-existing condition and after installed single barb at different times. (a) & (b) represents the bed elevation of pre-existing channel after 1 and 5 hours, respectively. (c) & (d) represents bed elevation with single barb after 1 and 5 hours, respectively.

3.3 Flow depth

The comparisons of flow depth between the pre-existing channel and with barb structure, the simulation contour results are shown in Fig. 8 and 9. According to experimental studies carried out by Gill (1972) it may be concluded that the maximum scour depth increased at a decreasing rate with the increasing in approaching flow depth. The scour depth increased proportionally with the flow depth for shallow flows. In pre-existing channel flow depth with scour hole varied from 1 cm to 9 cm and marked a little variation of depth in entire simulation time. It was also shown that the shape of the flow depth along the centerline for different time step roughly follows as the same trend. In case of barb structure, highest depth occurs near the thalweg of the channel due to turbulence. As expected flow, depth was present around the end of the barb and the water depth with scour whole varied between 2 cm to 12 cm (Figure 9).

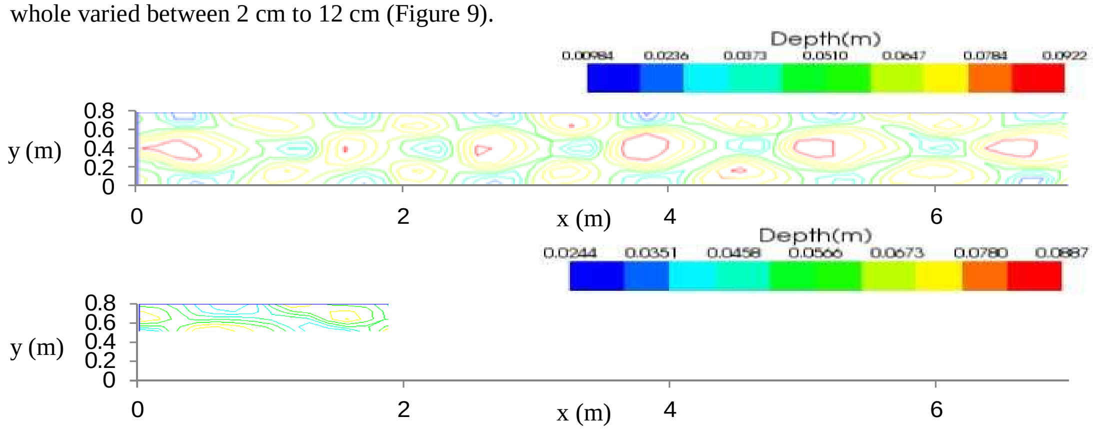

Fig. 8. Water depth contour result of the channel without barb structure (top: after 1 hour simulation, bottom: after 5 hour simulation).

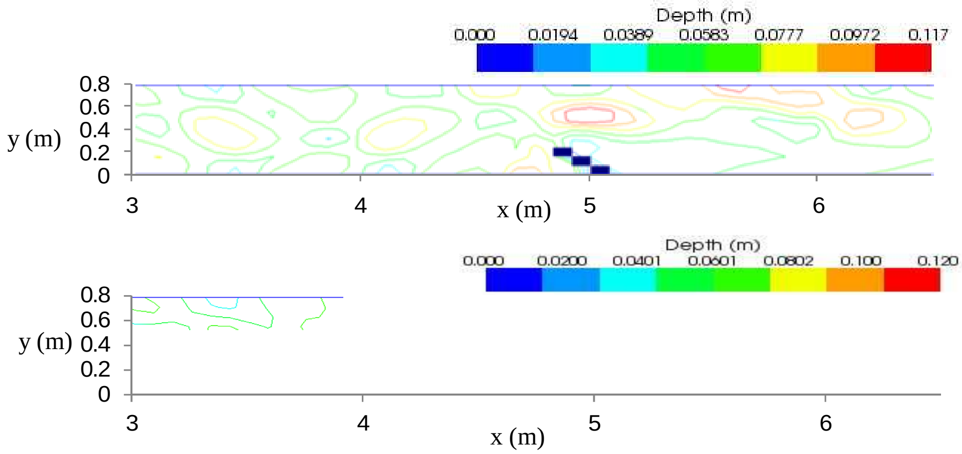

Fig.9. Water depth contour result of the channel with a barb installed at one sidewall of the channel (top: after 1 hour simulation, bottom: after 5 hour simulation).

4. CONCLUSIONS

The study was conducted by two-dimensional numerical simulation to observe the effects of barbs on bed configuration installed in channel sidewall. Informations were provided regarding flow patterns and bed configuration (scour and deposition). A comparisons was done between pre-exist straight channel and after installation of a stream barbs with erodible bed among a relatively high width-to-depth ratio. Using nearly uniform sand under controlled steady flow conditions the bars are formed at the upstream portion of the channel. The flow towards the pre-exist channel creates parabolic flow pattern that is influences to formation of double row bars. In case of after barb installation, due to the flow separation at barb head, return current developed at the upstream side induced zone of subcritical flow along the stream bank. It influenced the flow at downstream side creates a mixing zone just behind the structure and along the near bank line. Potential energy increased in the zone of upstream through backwaters effects. The upstream progression of subcritical reaches in the near bank region controls erosion and ultimately leads to deposition of sediments along the bank-line. At the downstream side of barb especially near the barb head scour occurs due to hydraulic jump and turbulence from flow mixing. In pre-existing channel flow depth with scour hole varied from 1 cm to 9 cm and marked a little variation of depth in entire simulation time. As expected flow depth was present around the end of the barb and the water depth including scour whole varied between 2 cm to 12 cm.

In conclusion, the simulation results showed that the installed barb successfully reduced the flow velocity along the bank and increased the turbulence velocity at the end of the barb structure, thereby successfully increasing the conveyance in the center of the channel. This means the barbs would be able to reduce the erosion by forming bars along the near bank of the channel. However, experimental study need to verify simulation result and required to test their general acceptability.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

A SCENARIO-BASED HOLISTIC APPROACH FOR MULTIPURPOSE RESERVOIR OPERATION CONSIDERING EFR, IRRIGATION AND HYDROPOWER DEMAND: A CASE STUDY

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ABSTRACT

This study presents the detailed outline of a holistic methodology for the operation of a multipurpose reservoir located in the Hari Rod River basin of Afghanistan by taking account of all the existing water demands (e.g. environmental flow requirement (EFR) by the riverine ecosystem, irrigation and hydropower generation) in the basin. At first, environmental flows required for healthy ecosystem in the Hari Rod River have been assessed for establishing suitable assessment techniques to be applicable for basin-wide management of the aquatic flora and fauna. Available historical data and information forms the basis of this study. Four well-established hydrologic-based approaches (e.g. Tessman method, 7Q10 method, Flow Duration Curve method, Indicators of Hydrologic Alteration method) and one hydraulic-based approach (e.g. Wetted Perimeter method) are applied to quantify the required environmental flows by the wholesome riverine environment. For proper allocation of environmental flows along with the water requirements from other existing demands (i.e. irrigation and hydropower generation) in the Hari Rod River basin, a reservoir simulation model for the planned Salma dam reservoir in the basin is developed within the modeling framework of HEC-ResSim software. In the developed reservoir model, water demand from irrigation sector and hydropower generation followed by the estimated additional EFR demand is incorporated in the priority rule of water allocation. Three different alternative scenarios are generated from the developed reservoir model to determine the potential impacts of allocating water for existing demands along with additional demand by EFR. At the same time, different mitigation options are suggested to alleviate those impacts completely. Using simulation technique, a new conservation zone rule curve was developed to decrease the water shortages for irrigation caused by the estimated additional demand due to water allocation for EFR in the Hari Rod River. However, irrigation water requirement is calculated by using the CropWat software. This study estimates that the water shortages can be considerably alleviated by improving the existing irrigation efficiency in the basin. This study finally provides another impact mitigation solution, by which water shortages can be reduced substantially by a little improvement of the existing irrigation efficiency in the basin.

Keywords: *Afghanistan, CropWat, EFR, Hari Rod River Basin, HEC-ResSim, Salma Dam Reservoir*

1. INTRODUCTION

Worldwide, increasing concerns over environmental sustainability and preserving ecosystems and their associated functions in rivers persuade the water managers to recognize the requirement of allocating specific amount of flow in the rivers with an acceptable level of quality which is often regarded as the environmental flow requirement (EFR) (Song et al., 2007; Tharme, 2003). EFR is the amount of water from the original river flow regime that needs to flow all the way to the outlet into the sea in order to maintain the specified valued features of the river ecosystem in a desirable condition (Pal et al., 2009; Song et al., 2007; Gordon et al., 2004; Smakhtin et al., 2004). Through mimicking the natural flow regime, EFR ensures the provisions of ecosystem goods and services that rivers provide on which humanity rely in numerous ways. Therefore, such flow is recommended in all the regulated rivers across the world to preserve the ecosystems integrity to a certain desired level (Mullick et al., 2010). Hari Rod River basin in Afghanistan has unstable environment where the ecological flora and fauna have been affected due to the years of civil war, severe drought, growing pressure on available water by irrigation sector within the catchments, and lack of attention to environmental impacts. The primary focus of water resources development projects in Afghanistan particularly in the Hari Rod River basin are irrigation and hydropower. Until now, less attention was paid on low flow and EFR downstream of these developments. However, with increasing consciousness and approbation for maintaining environmental sustainability, the apprehension to minimize adverse environmental impacts of such plans is getting much more attention nowadays. Legislation in Afghanistan emphasizes that its water resources management plan should reserve satisfactory amount of water to be required for environmental purposes in all its river basins. It is also highlighted in the water sector strategy of the country's national development plan. However, little is known about the existing environmental and natural circumstances of the river basins in Afghanistan except few reports identifying the possible management plans and development options (Qureshi, 2002). This has led to the recognition that environmental flow (EF) in each of the river basins in Afghanistan along with the potential impacts need to be quantified.

The available discharge in the Hari Rod River, which is mainly used for irrigation purpose within the basin, is deficit in the summer and excess in the spring. Accordingly, water user groups in the basin have adjusted their agricultural and irrigation water supplies to natural flow regime available in the river. However, there is a lack of distribution efficiency in the major sub-branches of the river caused by corrupted canal diversions, alignments and devastated bifurcations (Qureshi, 2002). Considering the present state of water management, establishment of environmental water requirement for the river is regarded as an essential part prior to any water resources development projects in the Hari Rod River basin of Afghanistan. The primary intention of EF management in the basin must be the anticipation of further degradation that may arise from flow regulation intensification, particularly loss of high flood flows, reduced base flows and further alteration in the seasonal inflows. Aiming to contribute to the research gap, this paper presents detailed outline of a holistic methodology for the operation of a multipurpose reservoir (proposed Salma dam) located in the Hari Rod River basin of Afghanistan, where natural flora and fauna have been affected due to the years of civil war, severe drought and growing pressure on the available water resources due to growing water demands by different competing sectors. Appropriate allocation and sustainable exploitation of the water resources while preserving riverine ecosystem integrity are essential in all the river basins of Afghanistan. Therefore, the present study attempts to estimate EFR required for the healthy ecosystem and quantify holistically the potential impacts of EFR allocation along with other water demands for irrigation and hydropower production on the proposed Salma dam reservoir operation and the irrigation water management policy in the Hari Rod River basin of Afghanistan.

2. MATERIALS AND METHODS

2.1 Description of the Hari Rod River Basin

Hari Rod River basin is located in the western part of Afghanistan (Figure 1), which is relatively more developed than other basins in that area of the country. Upper part of the basin is located in the Ghor province with an altitude of 4000 m above mean sea level (MSL), and lower part is in the Herat province having an altitude of 750 m above MSL. The basin area covers about 3.9 Mha with a population of 0.46 million. Hari Rod River has narrow valley with gravely bed in the upper reach, in the middle reach the river valley becomes wide, flat and meanders greatly below the village Obeh. In the lower reaches, the river forms part of the international boundary between Afghanistan and Iran, and finally flow into Turkmenistan where it disappears into the sand in downstream. The basin is characterized by distinguished climate with cold winters with snowfall and rainfall increasing with rising altitude. Usually, the rainfall occurs in spring and the mean annual precipitation is 236 mm with uneven spatial distribution. Runoff comes from snowmelt, which is the major source of surface water and groundwater of the basin spanning over only two months (February/March to April). High flood flows from March to June, and very low flows from August to February are observed in the Hari Rod River. Discharge from the river is mainly used for irrigation purpose in the basin and farmers have adapted their agricultural activities to this flow regime. In addition, groundwater is used as a supplement to satisfy the irrigation demand throughout the year. The proposed Salma dam reservoir is located near the Chisht-e-Sharif in Herat province, which is planned as a multi-purpose reservoir to support the irrigation activities and power generation. Now the project is under active construction that is to be utilized in the upcoming years. Significant lacking is observed in the consideration of environmental and ecological aspects as an important issue for this water resources development project (proposed Salma dam reservoir) in the basin. The reservoir should meet the current irrigation demand of 42,000 ha, and generate an installed capacity of 42 MW early after construction, and expected to meet the future irrigation demand for 74,859 ha after the development of full irrigation facilities in the basin. Considering the present status of demand and reservoir storage, no shortage is observed for irrigation when reservoir is operating only for hydropower generation. Even the EFR is satisfied in this base condition (Atef, 2009). Thus, the present study attempts to evaluate the future impacts, when the irrigation demand is expected to be very high, and the reservoir will be operated in the interest of both irrigation and hydropower demand along with the EFR demand.

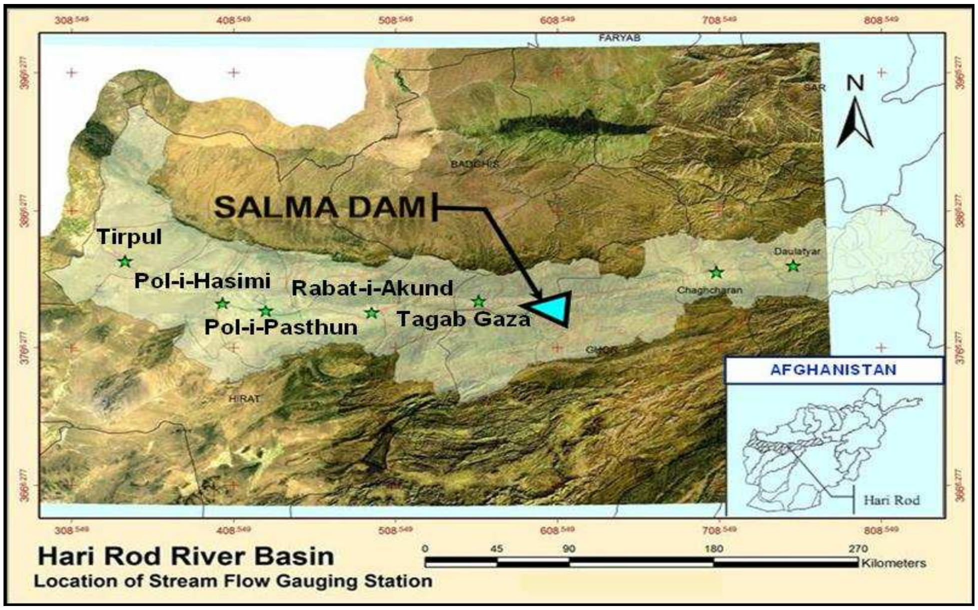

Figure 1: The Hari Rod River basin in Afghanistan

2.2 Estimation of EFR in the Hari Rod River

The framework of methodology (Figure 2) mainly consists of two subsets of activities: quantifying EFR and establishing reservoir operation policy by applying simulation technique to mitigate the impacts of incorporating EFR on other demands. The methodology followed in this study is largely based on observed data and information in the field. Recent (2001-2008) and past (1961-1980) available daily discharge and river cross-section data at Tagab Gaza monitoring station (located immediately downstream of the reservoir) in the Hari Rod river were collected. However, there has not been any observation made during 1981 to 2000 due to several years of conflict. The primary hydrological data (river cross-section) is collected from field survey at different points of the Hari Rod River and the secondary hydrological data is taken from irrigation department and FAO-EIRP office in Herat city of Afghanistan. Simple approaches have been adopted to analyze the available hydrological data. Well-known hydrological methods (e.g. Tessman or modified Tennant method, Flow duration curve (FDC) method, 7Q10 method, Indicators of hydrologic alteration (IHA) method considering Range of Variability Analysis (RVA) technique) and hydraulic method (e.g. Wetted perimeter method) were applied to quantify EFR. A detail review of all these approaches can be found in Smakhtin and Anputhas (2006).

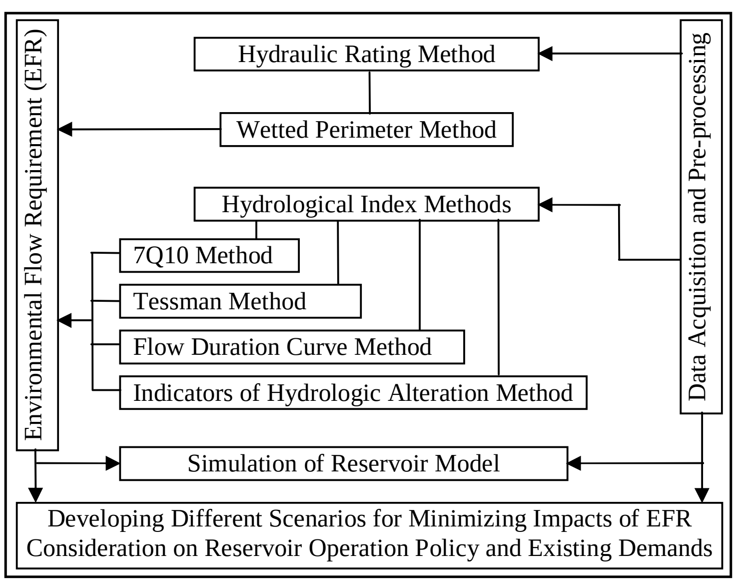

Figure 2: The methodological framework followed in the present study

2.3 Simulation of the Reservoir System

HEC-ResSim, a public domain model for reservoir system simulation developed by U.S. Army Corps of Engineers (USACE), is used for simulating the reservoir (proposed Salma dam reservoir) operation in the Hari Rod River basin. It utilizes reservoir inflow hydrographs, river networks and physical properties of the dam including reservoir characteristics and rule curves as necessary input data. Reservoir release, storage, spillway flow and downstream hydrographs are the model output (USACE, 2007). In the developed reservoir model, water is firstly allocated only for irrigation and hydropower demand and then EFR demand is considered along with them in sequence on a priority basis. The reservoir should meet the current irrigation demand of 42000 ha, and generate an installed capacity of 42 MW early after construction, and expected to meet the future irrigation demand for 74859 ha after development of irrigation facilities. Considering the present status of demand and storage behind the Salma dam, no shortage of water is observed for irrigation when reservoir is operating only for hydropower generation. Even EFR is also satisfied in this base condition. Thus, the present study attempts to evaluate the future impact when irrigation demand is expected to be very high, and reservoir will be operated in the interest of both irrigation and hydropower demand along with EFR demand. Twenty years daily time series data is used in the reservoir model for the simulation. Another publicly available decision support tool for computing crop water and irrigation water requirements, CropWat model, which is developed by the Food and Agricultural Organization (FAO) of the United Nations (UN), is used to estimate irrigation water requirement (IWR) for each crop (FAO, 1999) in the Herat province located downstream of reservoir. Mean monthly temperature, rainfall, evaporation and soil data are used to estimate net irrigation depth for each crop. Finally, three different scenarios are generated to evaluate the potential impacts of allocating EFR on the irrigation and hydropower demands and possible solutions are suggested to mitigate those impacts in the Hari Rod River basin of Afghanistan.

3. RESULTS AND DISCUSSIONS

3.1 EFR in the Hari Rod River

Major tributaries of the Hari Rod River are located in the lower part of the Hari Rod River basin. Thus, the present study considered the monitoring stations for the EFR assessment, which are located in the lower part of the basin. Tagab Gaza and Tirpul station is located in the middle and lower part of the basin respectively along with three more gauging stations (Pol-i-Pashtun, Pol-i-Hashimi and Rabat-i-Akhund) along the river between these two stations (Figure 1). Since the Tagab Gaza monitoring station is located immediately downstream of the Salma dam reservoir, EFR estimation in that station is more justified to be incorporated into the reservoir simulation. Estimated EFR in the Tagab Gaza monitoring station is presented in Figure 3.

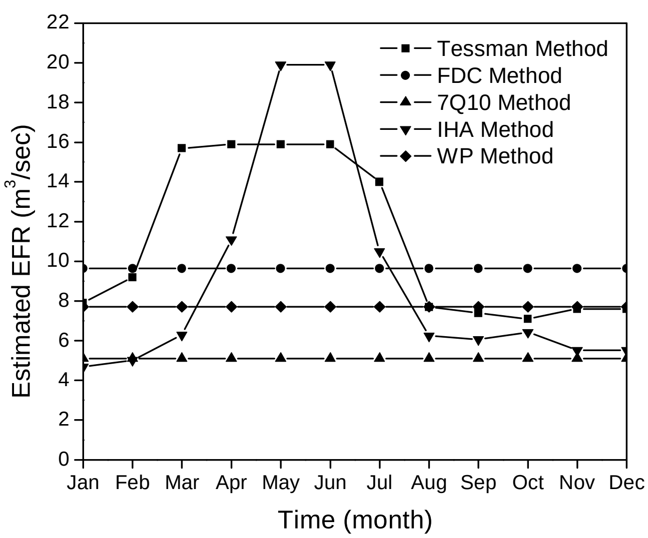

Figure 3: Estimated EFR in the Tagab Gaza monitoring station in the Hari Rod River

Tessman method (also called modified Tennant) uses a percentage of the average annual flow (AAF) for a monthly basis rather than two season (six months period) basis (Tennant method) to estimate EFR, which refers that this method considers seasonal variability. For selecting monthly minimum flow (MMF), this method applies the criteria such as: if average monthly flows (AMF) < 40% of average annual flow (AAF), then MMF equals the AMF, if AMF > 40% of AAF, then MMF equals 40% of AAF, and if 40% of AMF > 40% of AAF, then MMF equals 40% of AAF respectively. In FDC method, generally Q_{95} (% of time when a specified river discharge is equalled or exceeded 95% during the period of calculating EFR) and Q_{90} (% of time when a specified river discharge is equalled or exceeded 90% during the period of calculating EFR) indexes are used to define EFR, which indicate the extreme low flow conditions to protect the riverine ecosystem integrity. 7-day low flow with a 10-year return period is calculated to quantify EFR in 7Q10 method using 20 years daily discharge data. This method depicts that the resulted minimum stream flow should be maintained to protect river water quality. IHA method quantifies the hydrologic sequential variability of flow regime into 32 biological parameters based on statistical evaluation. These parameters were calculated from 20 years daily flow records (1961-1980) at Tagab Gaza station by considering single period analysis with nonparametric statistics using IHA (Indicators of Hydrologic Alteration) public domain software released by Nature Conservancy, USA. The result shows that there is an alteration between the recent (1971-1980) time and pre-settlement (1962-1970) time periods. The monthly low flow median in pre-impact period is 6.4 m^3/s, which is decreased to 5.3 m^3/s in the post-impact period. There is significant variation in April and May median flows of post-impact period compared to pre-impact period. The results from IHA analysis are used for range of variability analysis (RVA) technique and the management targets are identified. First, the management rules are developed based on the estimated ecological information needed to accomplish the target flows on annual basis. Then, biological goals are identified to achieve through generated flow regime. The nonparametric monthly low flow analysis for Tagab Gaza monitoring station indicates that the post-impact period peak discharge and volume increased by 4% compared to pre-impact period. The positive and negative changes in water conditions are increased from the pre-impact period than the post-impact period. In the wetted perimeter method, EFR is estimated as 7.71 m^3/s. Since it is difficult to estimate the point of maximum curvature in the wetted perimeter to discharge relationship, EFR is obtained based on the breaks in slope of wetted perimeter verses discharge diagram.

Tessman method generally provides good results at initial level of analysis. However, most often it gives unexpected under or over estimated results due to seasonal condition. EFR by this method is considerably higher than other methods for dry season. Estimated EFR by 7Q10, wetted perimeter and FDC approach are lower than that estimated by Tessman method. FDC method concise the entire flow distributions and shows the range of extreme low flow conditions in the river and can be used for primary screening of the complete series of river discharges from low flow to flood events. However, when the question of seasonal variability arises, 7Q10, FDC and wetted perimeter methods cannot fully include EFR in the management level. It is recognized that intra and inter-annual variability of hydrologic regimes are needed to maintain and restore the natural forms and functions of the aquatic ecosystems. Thus, IHA considering RVA is the most sophisticated form of hydrological index approaches to quantify EFR in a basin. It generally considers the magnitude, duration and frequency of the flow regime. Therefore, the estimated EFR using IHA method is incorporated into the reservoir simulation model to define its EFR demand during simulation of the developed reservoir model and evaluating the impacts of EFR on the other existing demands.

3.2 Scenario-based Simulation of the Salma Dam Reservoir

The proposed multi-purpose Salma dam reservoir is under construction in the Hari Rod River basin of Afghanistan, which is to be used for water supply and power generation. The hydrological characteristics of Salma dam reservoir is given below:

- Reservoir gross capacity is 633 Mm^3
- Live storage capacity is 514 Mm^3
- Average annual inflow to the river is 1217 Mm^3
- Existing average annual irrigation demand for 42000 ha of land is estimated as 246.57 Mm^3
- Average annual irrigation demand in future for 75000 ha of land is estimated as 587.62 Mm^3
- Average annual evaporation is estimated as 12.2 Mm^3
- Average annual gross EFR demand is calculated as 277.73 Mm^3
- Specific EFR demand (four months) is found as 45.67 Mm^3

In the proposed project, EFR demand has not been considered which is essential for the riverine ecosystem integrity and its long run sustainability. Therefore, there must be shortages of water for the existing irrigation and hydropower demand, if allocation for EFR demand is ensured in the basin. Thus, different alternative scenarios based on with or without EFR demand following the existing or proposed guide curve, were generated to evaluate the possible impacts of incorporating EFR into the reservoir operation policy as well as on the other existing (i.e. irrigation and hydropower) demands. All the scenarios from the simulation are described step-by-step in the following sections.

3.2.1 Scenario I: Simulation of Reservoir System without Considering EFR

The intention of developing this scenario at base conditions is to elaborate the dam status immediately after construction as the reservoir is being designed to operate for both the irrigation and hydropower demand. The simulation shows that there are about 18 fully satisfied years for irrigation within 20 years of analysis. The irrigation shortage is observed in only two dry years in specific months of dry season. Available water in the reservoir is very less in the year 1966 and from 1970 to 1971. Thus, these two periods are highlighted in this study. The shortage of water due to the irrigation demand from August to November in 1971 is presented in Figure 4 and in that year total annual inflow is 281 Mm^3. However, the average annual irrigation demand and average annual inflow (AAI) to the river are found as 584 Mm^3 and 1217 Mm^3, respectively. This implies that there is no shortage of irrigation demand for the normal and wet years, if the reservoir inflow is equal to or more than the estimated AAI. The result also shows that total reservoir inflow in year 1969 to 1970 was almost same as year 1971, but there was no shortage observed. This is due to the reservoir storage during 1968 to 1969 period when the reservoir inflow is more than the estimated average inflow. Thus, besides meeting the demands, much water was stored in the reservoir and released in the year 1970 and no shortage was observed in that specific year. Observation shows that annual inflow in 1971 was extremely low which is almost equal to one-fifth of AAI. Although there was less inflow into the reservoir, simulation result shows that a total of 8 months (January to end of July) was fully satisfied. However, irrigation demand was 20% satisfied in August and 45% in September and October, assuming that there is no irrigation demand in December. The average energy simulated is reasonably more than the proposed capacity (186.13 MWh). This is because the release for both hydropower and irrigation is from the hydropower gate in order to optimize both demands. It is considered as a key reference to compare with the other scenarios.

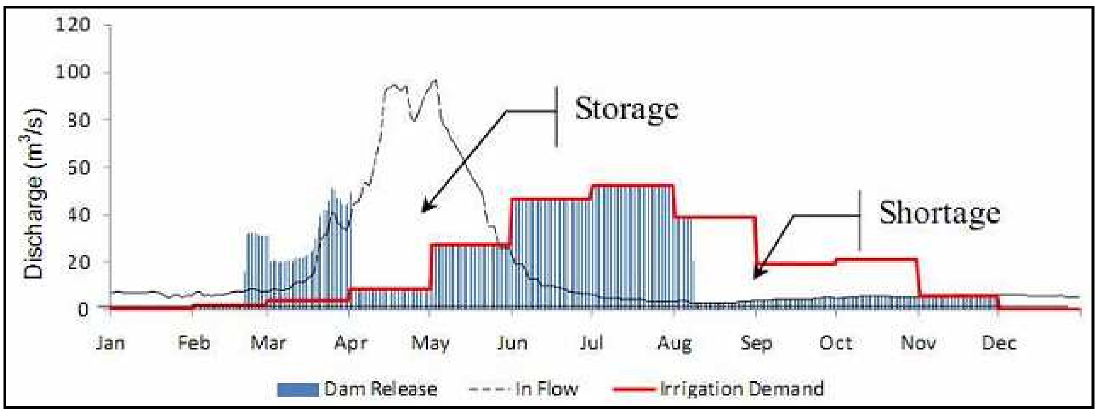

Figure 4: Dam release without considering EFR to satisfy irrigation demand in 1971

3.2.2 Scenario II: Simulation of Reservoir System by Considering EFR

This scenario evaluates the impacts of incorporating the EFR demand into reservoir operation policy on the other existing demands. The simulation shows that there will be more stress on the irrigation demand in the specific dry years and this is presented in Figure 5.

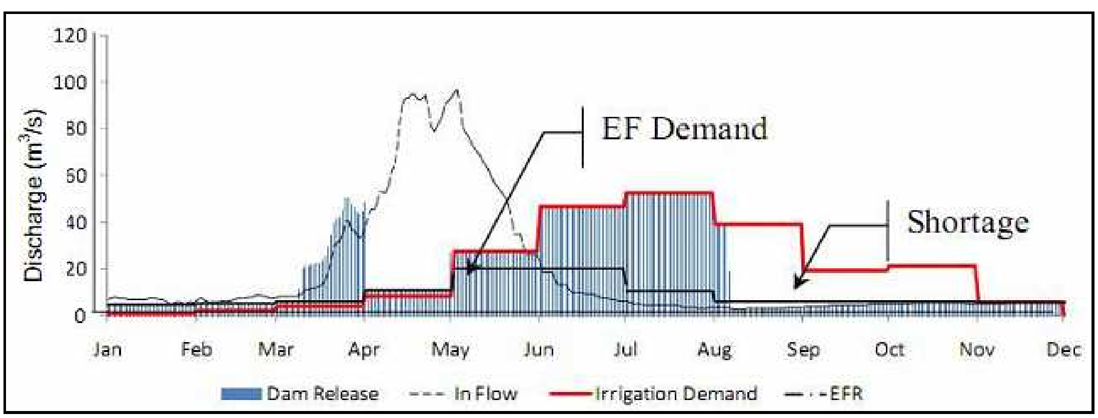

Figure 5: Shortage of irrigation demand in 1971 by considering EFR into reservoir operation

The release decision to satisfy EFR was defined as a minimum reservoir supply on monthly basis. This decision is made in reservoir hydropower gate and then EFR is evaluated downstream of the river at Tirpul station. River routing is carried out to calibrate the estimated EFR downstream of the river. Table 1 presents the impacts of EFR consideration into reservoir operation policy on irrigation demand. The shortage is observed from August to October of 1966, 1970 and 1971 out of 19 years of simulation. In addition, severe shortage is noticed in 1971, where reservoir is meeting almost 22% of the irrigation demand. However, much difference is not observed in average energy produced after adding EFR demand in reservoir simulation model.

Table 1: Shortage to satisfy irrigation demand due to EFR consideration for scenario II

Description	Aug-66	Sep-66	Oct-66	Sep-70	Oct-70	Aug-71	Sep-71	Oct-71
Release (Mm^3)	59.0	14.2	16.5	37.7	19.2	24.3	10.9	13.9
Irrig. Demand (Mm^3)	101.0	49.8	54.8	49.8	54.8	101.0	49.8	54.8
% Satisfied	58	29	30	76	35	24	22	25

3.2.3 Scenario III: Developing Proposed Guide Curve for Reservoir Operation

The simulation result shows that the reservoir starts to storing water from April to end of June, and releases water from July to next April. This release decision is applicable for the normal and wet years, where the dam inflow is almost equal to AAI. It may also help to decrease the possibility of flooding while meeting the irrigation demand and generating the hydropower energy. However, during drought period, when the dam inflow is almost one-fifth of AAI, the existing conservation zone will not fully meet the irrigation, hydropower and EFR demands. Thus, a new conservation zone is developed based on dry year circumstances to meet all three demands during dry years. The result also shows that no shortage will be observed after applying the new rule curve of conservation zone, which is presented in Figure 6.

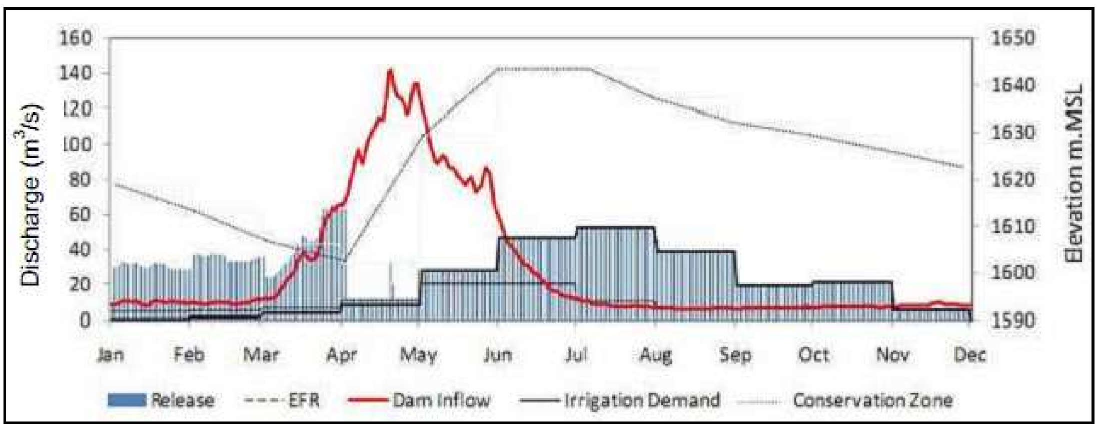

Figure 6: Proposed guide curve of conservation zone for reservoir operation

Results obtained by all scenarios are summarized to evaluate the impacts of EFR consideration on the irrigation and hydropower demand (Table 2). It shows that when reservoir releases for only irrigation and hydropower, there is shortage for years 1966 and 1971. However, after EFR inclusion into the reservoir operation policy along with other demands, shortage is noticed for the years 1966, 1970 and 1971. Afterwards, by applying the proposed new conservation rule curve estimated in scenario III simulation, it can be shown that there is no shortage for irrigation, hydropower and EFR demand. At the same time, the energy generated for those specific years for scenario III is more then that for scenarios I and II.

Table 2: Comparison of the results obtained by all three scenarios

Description	Unit	Scenario I Irrig.+Pwr/Existing Conservation Zone		Scenario II Irrig.+Pwr+EFR/Existing Conservation Zone			Scenario III Irrig.+Pwr+EFR/Newly Proposed Conservation Zone	
	Years	1966	1971	1966	1970	1971	1966	1971
Irrigation Shortage	Mm^3	114	155	116	86	157	0	0
	%	54	73	56	54	76	0	0
Power Gen.	MWh	154.5	121.6	156	-	120	197.4	140.9

3.3 Scenario-IV: Impact Mitigation by Improving the Irrigation Efficiency (IE)

Improving the current irrigation practices could be another potential solution for minimizing the irrigation shortage. Before the conflict period, total cultivated area was about 97000 ha, which has been drastically decreased to 42,141 ha. Thus, there is a potential scope of expanding the current irrigation scheme in future. Scenario II shows that there will be shortage of water in dry years for 74859 ha irrigable areas if EFR demand is considered into reservoir operation policy. Therefore, improving IE can be a good option to mitigate this impact. For this, average IWR per ha was estimated to find the total demand from the future irrigable area.

Table 3: Improving irrigation efficiency (%) for mitigating impact for the whole cultivable area

Description	Unit	IE	Wet Season	Dry Season	Total
Total cultivated area	ha	-	52402.00	22458.00	74859
Estimated avg. CWR per ha	L/s/ha	-	2.42	4.13	-
Total irrigation water supply (winter)	Mm^3	-	361.01	**226.00**	587.01
Existing IWR (winter)	Mm^3	35%	328.44	240.26	568.70
IE increased by 5% IWR (winter)	Mm^3	40%	317.24	**225.09**	542.33
IE increased by 10% IWR (winter)	Mm^3	45%	309.59	221.09	530.68
IE increased by 15% IWR (winter)	Mm^3	50%	303.11	211.54	514.65

Table 3 demonstrates that improvement of IE only by 5%, there will not be any shortages of water for irrigation hydropower and EFR demands. During dry season, total supply and demand from the reservoir is simulated as 226 Mm^3 and 240.26 Mm^3, respectively. However, the demand is more than the supply, which means that there is shortage having 35% existing IE. The result also shows that the irrigation demand is decreased almost about 15 Mm^3 for improving IE only by 5%, and hence the supply is meeting the full irrigation demand. Besides, further options will be created for cultivating more irrigable area during wet years with the surplus water.

4. CONCLUSIONS

This study was an attempt for establishing a framework to estimate EFR in the Hari Rod River basin of Afghanistan by using different well-known hydrological and hydraulic methods. After estimating EFR, a reservoir simulation model was developed for the proposed reservoir (Salma dam reservoir) system applying the HEC-ResSim model to estimate the impact of incorporating EFR into the reservoir operation policy as well as on the irrigation management. It was found that the hydrological based methods were suitable to establish EFR in the Hari Rod River basin of Afghanistan. Estimated EFR using the IHA method considering the RVA approach was preferred for simulating the reservoir model, as it considers natural flow variability, magnitude, frequency and duration of flow during EFR estimation. The study concludes that there will be shortage of water during dry years if the Salma dam reservoir operates for meeting all the three demands such as irrigation, EFR and hydropower production. Impact of EFR consideration on the reservoir operation policy can be mitigated by applying the developed conservation zone for dry years. This can save about 2000 ha land that was supposed to be affected. The present study conclusively proves that impacts of water shortages can be minimized significantly by a little improvement of the existing irrigation efficiency in the Hari Rod River basin of Afghanistan.

ACKNOWLEDGEMENTS

This research was conducted at the Water Engineering and Management field of study in the Asian Institute of Technology (AIT), Thailand with the financial assistance from the Government of the Netherlands through a fellowship grant. The authors are thankful to the Government of Netherlands for providing the financial supports as well as to AIT for providing the technical and educational supports during the study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SURGE PHENOMENA AND FACTORS INFLUENCING STORM SURGES

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ABSTRACT

A storm surge is an abnormal rise of the sea level caused by a tropical cyclone moving over a continental shelf. In fact, high amplitude waves generate continuously in the sea and move towards the coast. Surges are caused by the interactions of air, sea and land. The cyclone provides the driving force in the form of a very high horizontal atmospheric pressure gradient and very strong surface wind. As a result, the sea level rises and continues to rise as the cyclone approaches the shallow water, and reaches a maximum along the coast near the point of landfall. The waves inundate vast stretches of coastal area and sweep away everything. The period of the main surge at a place varies from few hours to more than a day depending on the speed of the cyclone and other related factors.The sea level gradually tends to return to the normal state in this stage. It includes, however, various kinds of oscillations due to topographic and other effects. It was verified that the storm surge is very sensitive to pressure drop, radius of maximum wind, bathymetry and coastal geometry of perspective coastal zone of the country.This paper is to design a model which looks into the surge phenomena and influencing parameters for storm surges. For testing sensitivity of the different parameters, the IIT storm surge model has been simulated through 9 different experiments using synthetic data of Cyclones. Moreover the model was also tested with the actual data and the results obtained from the model are good agreement with reported data.

Keywords: *Cyclone, topography, forerunner, resurgence, shallow water*

1. INTRODUCTION

Cyclones develop over warm seas near the equator. Air heated by the sun rises very swiftly, which creates areas of very low pressure. As the warm air rises, it becomes loaded with moisture which condenses into massive thunderclouds. Cool air rushes in to fill the void that is left, but because of the constant turning of the earth on its axis, the air is bent inwards and then spirals upwards with great force. The swirling winds rotate faster and faster, forming a huge circle which can be up to 2,000 km across. The centre of the storm is calm and cloudless which is called the eye, where there is no rain, and the winds are fairly light. When warm air rises from the seas and condenses into clouds, massive amounts of heat are released. The result of this mixture of heat and moisture is often a collection of thunderstorms, from which a tropical storm can develop.

A storm surge is an abnormal rise of the sea level caused by a tropical cyclone moving over a continental shelf. In fact, high amplitude waves generate continuously in the sea and move towards the coast. Surges are caused by the interactions of air, sea and land. The cyclone provides the driving force in the form of a very high horizontal atmospheric pressure gradient and very strong surface wind. As a result, the sea level rises and continues to rise as the cyclone approaches the shallow water, and reaches a maximum along the coast near the point of landfall. The waves inundate vast stretches of coastal area and sweep away everything. Storm surges at any place may be divided into three stages: forerunner, the main surge and resurgence. A forerunner is the gradual rise of sea level preceding the main storm. Even when the cyclone is far from the coast, some large-scale disturbances seem to produce variations in coastal sea level. Thus a forerunner can be used as an indicator of the arrival of a cyclone. The main surge follows after the storm surge rises to its peak. The period of the main surge at a place varies from few hours to more than a day depending on the speed of the cyclone and other related factors. Resurgence follows the main surge. The sea level gradually tends to return to the normal state in this stage. It includes, however, various kinds of oscillations due to topographic and other effects. The resurgence may sometimes continue for two or three days.The factors influencing storm surges may be classified into major four categories. These are: 1) Meteorological; 2) Oceanographic; 3) Hydrological and 4) Geographical.

2. METHODOLOGY

The IIT storm surge model has been simulated through different experiments using synthetic data of Cyclones. Moreover the model was also tested with the actual data of the severe cyclone Sidr-2007 at the central coastal zone of Bangladesh.

2.1 Governing Equations

For numerical treatment, it is convenient to express the governing equations in flux form as

$$\frac{\partial \zeta}{\partial t} + \frac{\partial \tilde{u}}{\partial x} + \frac{\partial \tilde{v}}{\partial y} = 0 \qquad (1)$$

$$\frac{\partial \tilde{u}}{\partial t} + \frac{\partial}{\partial x}(u\tilde{v}) + \frac{\partial}{\partial y}(u\tilde{v}) - f\tilde{v} = -g(\zeta + h)\frac{\partial \zeta}{\partial x} - \frac{1}{\rho}(\zeta + h)\frac{\partial p_a}{\partial x} + \frac{T_x}{\rho} - \frac{c_f \tilde{u}}{(\zeta + h)}(u^2 + v^2)^{\frac{1}{2}} \qquad (2)$$

$$\frac{\partial \tilde{v}}{\partial t} + \frac{\partial}{\partial x}(u\tilde{v}) + \frac{\partial}{\partial y}(u\tilde{v}) + f\tilde{u} = -g(\zeta + h)\frac{\partial \zeta}{\partial y} - \frac{1}{\rho}(\zeta + h)\frac{\partial p_a}{\partial y} + \frac{T_y}{\rho} - \frac{c_f \tilde{v}}{(\zeta + h)}(u^2 + v^2)^{1/2} \qquad (3)$$

Where $\tilde{u} = (\zeta + h)u$ and $\tilde{v} = (\zeta + h)v$ are new prognostic variables and $(\zeta + h)$ gives the total depth of the basin.
The equation of continuity (1) along with the two momentum equations (2) and (3) form the three basic equations of the numerical model. It consists of a set of three coupled equations for the three unknowns $\tilde{u}, \tilde{v}$, and ζ. If the terms in these three equations arise out of (i) Coriolis terms, (ii) the inverted barometric effect i.e. $\frac{\partial p_a}{\partial x}$ and $\frac{\partial p_a}{\partial y}$ due to fall in atmospheric pressure, (iii) the component of wind stress (T_x, T_y), and (iv) the bottom stress component (F_x, F_y) could be specified by meteorological data and the geometry of the continental shelf then the problem would be solved by numerical integration. The response in the sea at any instant $t > 0$ then determines the surge heights.

2.2 Boundary and Initial Conditions

Theoretically the only boundary condition needed in the vertically integrated system is that the normal transport vanish at the coast, that is ,

$$u\cos\alpha + v\sin\alpha = 0 \quad \text{for all } t \geq 0 \quad \text{for all } t \geq 0 \qquad (4)$$

where α denotes the inclination of the outward directed normal to the x-axis. It then follows that $u = 0$ along the y-directed boundaries and $v = 0$ along the x-directed boundaries.
At the open- sea boundary, the normal currents across the boundary may be prescribed, yielding a condition such as (4) modified by a non-zero term on the right hand side of the equation. Alternatively, a radiation type of condition may be applied, which leads to (Heaps, 1973)

$$u\cos\alpha + v\sin\alpha + \left(\frac{g}{h}\right)^{\frac{1}{2}}\zeta = 0 \qquad (5)$$

Application of a radiation type of condition (4) at the open sea boundary of a model allows the propagation of energy (disturbances) only outwards from the interior in the form of simple progressive waves. It also helps to eliminate the transient response more quickly as a result of the frictional dissipation in the system. Flather (1976) noted that application of a radiation condition in the numerical model may remove the unrealistically large currents and grid scale oscillations in the vicinity of the open boundary, which may be produced by the application of conventional open-sea boundary condition (i.e., $\zeta = 0$ at y = 0).
As usual it is assumed that the motion in the sea is generated from an initial state of rest, so that

$$\zeta = u = v = 0 \quad \text{everywhere for } t = 0 \qquad (6)$$

2.3 Meteorological Factors

Meteorological Factors are: (a) pressure drop; (b) the maximum sustained wind; (c) radius of maximum wind; (d) surface wind distribution and (e) forward motion of the storm. The difference of pressure between the eye of the cyclone and of the surroundings is known as pressure drop. Due to low pressure in the eye of the cyclone, there is a rise in the sea level. From the hydrostatic consideration, a fall in pressure of 1 mb would lead to a rise in the sea level of approximately 1 cm. Also, the pressure drop influences to some extent the location of peak surge on the coast. The maximum sustained wind is mainly due to very strong circulatory wind around the eye of the cyclone. Generally this strong wind ring is at a distance between 40 to 60 km or even higher from the eye. This distance is the radius of maximum wind, which is responsible for overall extent of the storm as well as surge along the coastal belt.

Case study: (Testing sensitivity of meteorological factor considering the Bay of Bengal as model domain)

The effect of wind –stress forcing like Pressure drop:
The objective of these experiments is to study the role of wind-stress in the overall computation of storm-surges. For experiments, the various parameters set during these experimental phase based on the effect of wind-stress forcing like pressure drop, are as follows:

No. of East-West Grid Points	: 331
No. of North-South Grid Points	: 154
Track's Starting Point	: 18° N, 89.20° E
Track's Ending Point	: 22° N, 89.80° E
Simulation of model run	: 18 hours
Time Step (DT)	: 60 sec.
No. of iterations to execute (KS)	: 1080 (KS= DT × Hrs)
Radius of Maximum Wind (Rmax)	: 40 km

Table 1: The effect of pressure drop

Experiment No	Presseure Drop (HPa)	Maximum Surge Height (m)
1	64	6.14
2	54	5.34
3	74	6.83

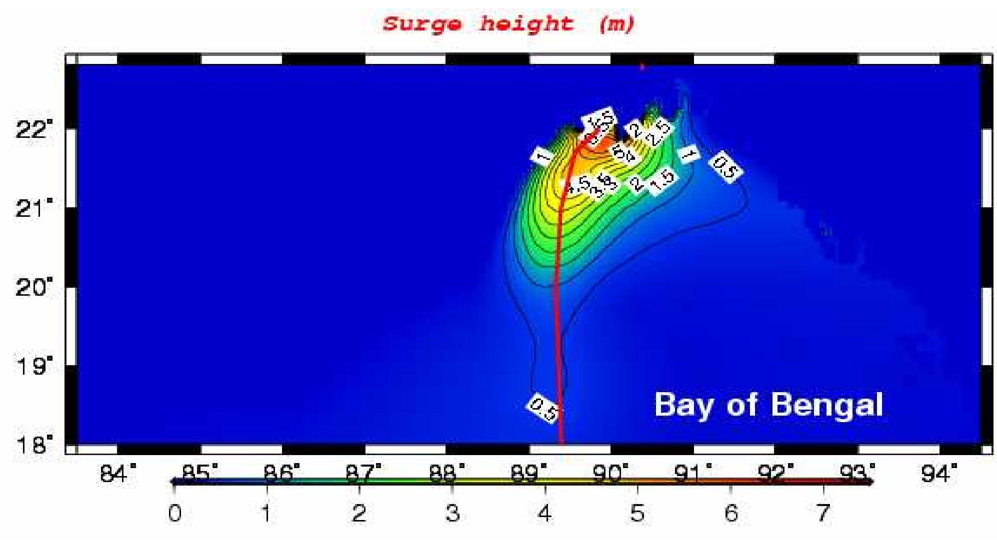

Figure 1: Cyclone land-falling at a coast at actual depth with pressure drop 64 H Pa.

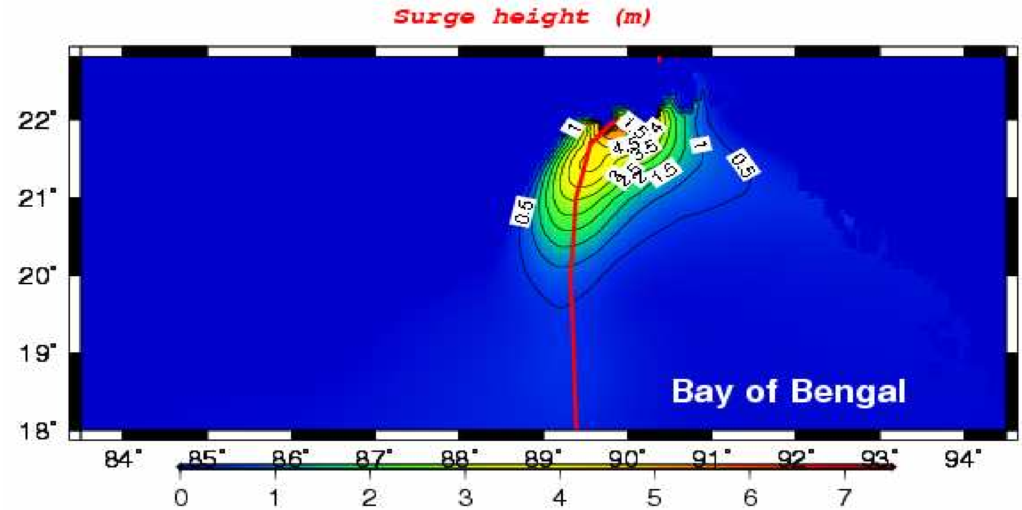

Figure 2: Cyclone land-falling at a coast at actual depth with pressure drop 54 H Pa.

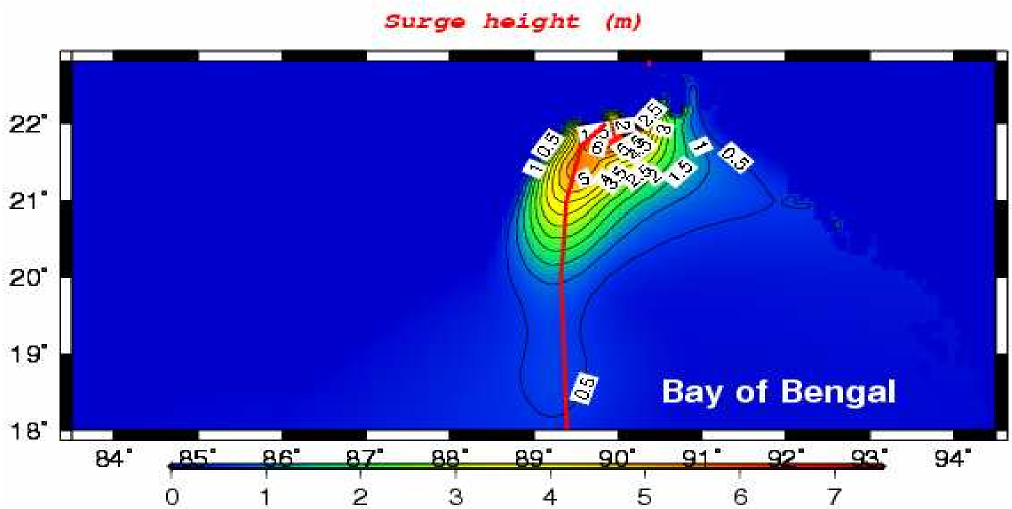

Figure 3: Cyclone land-falling at a coast at actual depth with pressure drop 74 H Pa.

The effect of Radius of maximum wind:
For experiments, the various parameters set during these experimental phase based on the effect of wind-stress forcing like radius of maximum wind, are as follows:

No. of East-West Grid Points	: 331
No. of North-South Grid Points	: 154
Track's Starting Point	: 18° N, 89.20° E
Track's Ending Point	: 22° N, 89.80° E
Simulation of model run	: 18 hours
Time Step (DT)	: 60 sec.
No. of iterations to execute (KS)	: 1080 (KS= DT × Hrs)
Pressure Drop (DP)	: 64 HPa

Table 2: The effect of Radius of Maximum wind

Experiment No	Radius of Max. Wind (Km)	Maximum Surge Height (m)
4	50	6.809232
5	45	6.501345
6	35	6.058701

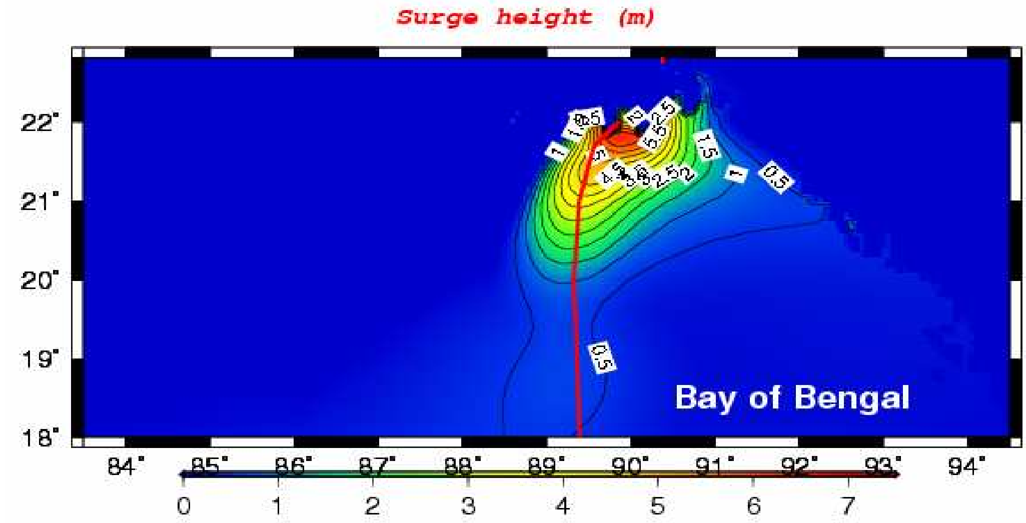

Figure 4: Cyclone land-falling at a coast at actual depth with Rmax 50 km

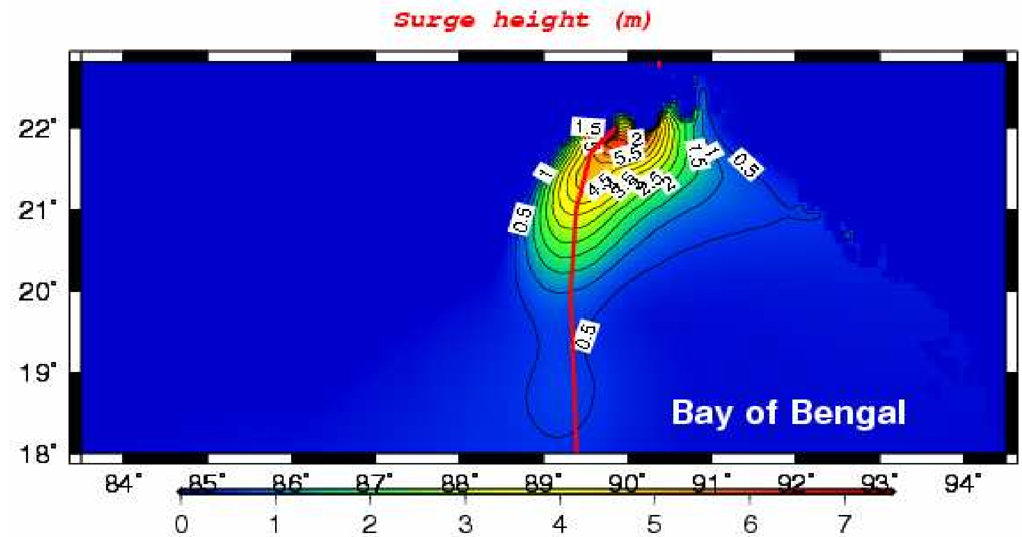

Figure 5: Cyclone land-falling at a coast at actual depth with Rmax 45 km

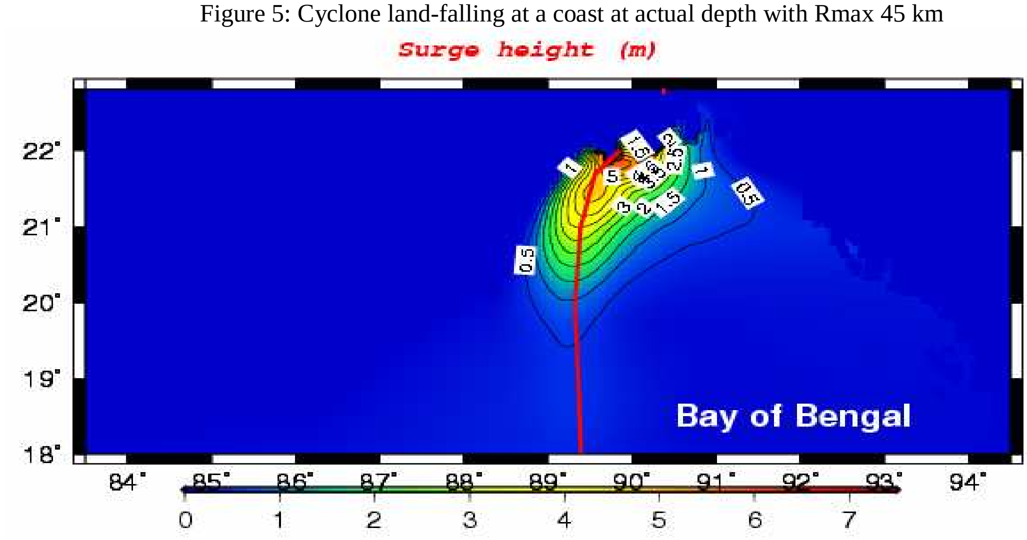

Figure 6: Cyclone land-falling at a coast at actual depth with Rmax 35 km

2.4 Oceanographic Factors

The most important oceanographic factors which may modify the surge amplitude are (a) bathymetry, (b) astronomical tides and (c) inshore currents in closed regions. Modeling experiments show that the surge response is very sensitive to the near coastal bathymetry. The head of the Bay of Bengal is very shallow (less than 10 meters deep) and is characterized by sharp changes in sea bed contours. The shallowness of water and the consequent large bottom friction retard any current underneath and may considerably modify the surge heights in this region. Astronomical tide is another kind of rise of sea level which occurs as a result of minor gravitational imbalance due to attraction of the celestial bodies (mainly moon and sun) upon the earth surface. Tide amplitude varies with the periodical movements of these heavenly bodies relatively to the earth. The rise due to tide may be as high as 4.8 meters above the mean sea level at some parts of Bangladesh coast. If the arrival of the surge coincides with the time of high tides, worst devastation may takes place. The amplification of surge becomes very complicated through its nonlinear interaction with tide in shallow water (Proudman, 1955).

2.5 Hydrological Factors

The main hydrological factors, which may modify surge development, are (a) river discharge in sea and (b) rainfall distribution.The discharge of fresh water carried by the rivers may modify the surge amplitude, for example especially in the Head Bay of Bengal where a large number of inlets including a major river system Ganga- Brahmputra – Meghna joins the sea. The dynamic effect of these inlets and estuaries is the potentially deep inland penetration of surge originating the Bay.The impact of heavy precipitation, associated with tropical cyclone, on surge height has not been studied for the Bay of Bengal. However, it is expected to influence the surge amplitude marginally.

2.6 Geographical Factors

Geographical factors correspond to (a) coastal geometry; (b) offshore island; and (c) the topography or spot heights of the surge affected areas. The location of highest surge depends on the coastal geometry of the basin (Nikhil et al , 2011). Experiments suggest that the curving coast not only shift the peak surge position but also affect its height (Dube et al, 1982). It is seen that conventional method of coastline representation by small straight-line segments ends to over reflect the wind driven water. For example, because of the northward converging nature of the Bay of Bengal, storm blowing straight into it may funnel the water towards the north (depending upon the track of the cyclone).

Case study: (Testing sensitivity of geographical factor considering the Bay of Bengal as model domain)

The effect of coastal geometry: The objective of the experiments based on this factor was to understand the sensitivity and the relationship of coastal shape to storms on water surface, and the resulting storm-surge as concave coast will experience more surge than the convex coast. For experiments, the various parameters set during these experimental phase based on the effect of coastal geometry, are as follows:

Table 3: The effect of coastal geometry perspective to Bay of Bengal

Experiment No	Starting Point	Ending Point	Maximum Surge Height (m)
7	18 °N, 89.20 °E	22.50 °N, 91.80 °E	3.84
8	18 °N, 89.20 °E	22.20 °N, 90.70 °E	4.20
9	18 °N, 89.20 °E	21.90 °N, 89.20 °E	3.85

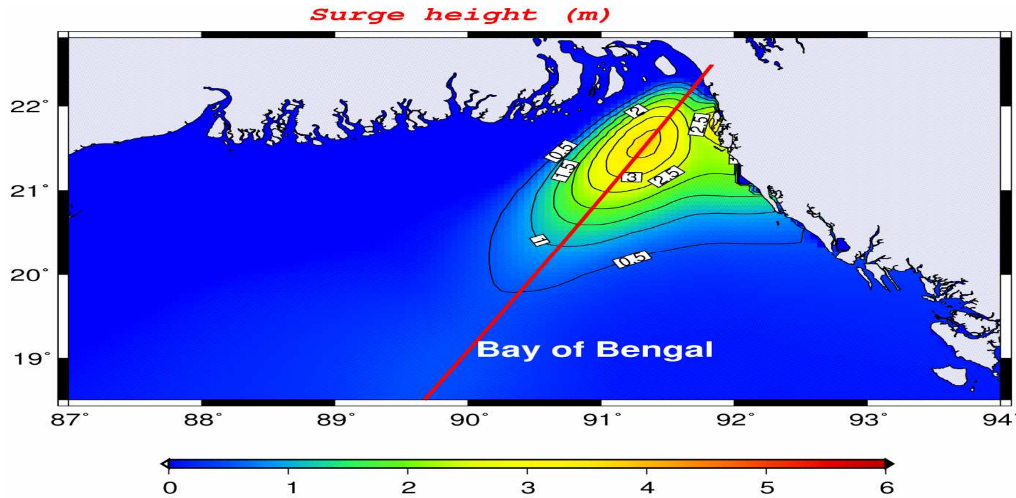

Figure 7: Cyclone land-falling in eastern coastal zone at actual depth

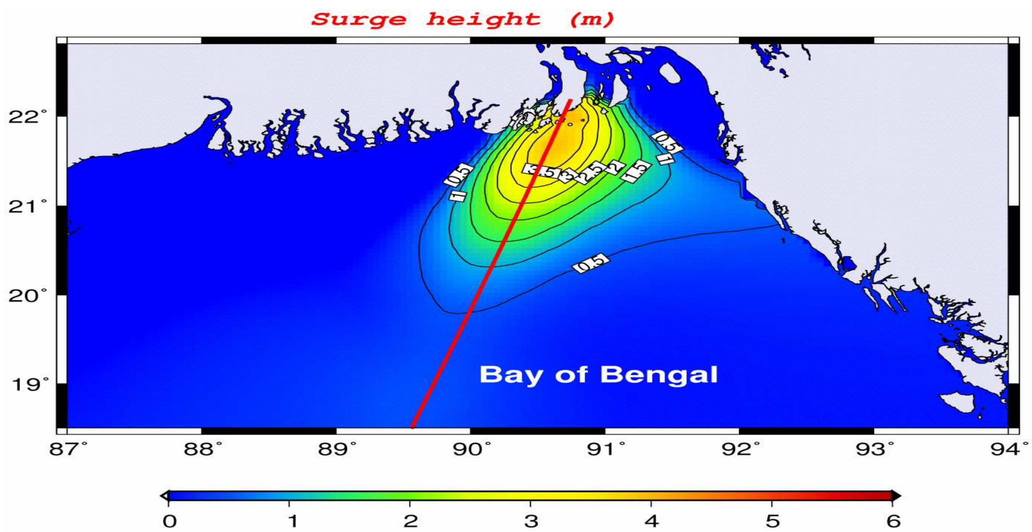

Figure 8: Cyclone land-falling in central coastal zone at actual depth

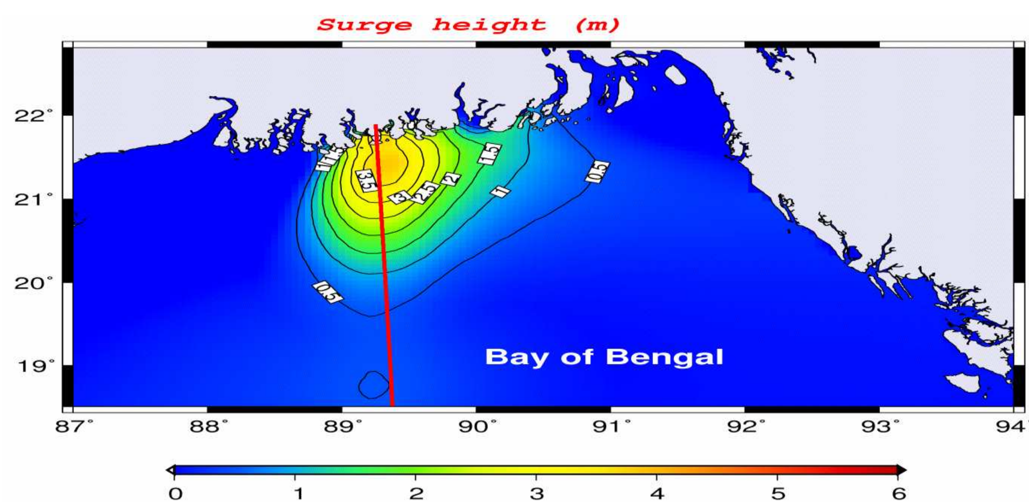

Figure 9: Cyclone land-falling in western coastal zone at actual depth

3. RESULTS AND DISCUSSIONS

For testing sensitivity of the meteorological parameters, we consider the real bathymetry data and differences in the pressure drop; it was found that higher surges are basically associated with lager differences in the pressure drop. The three different values for R-max (Radius of Maximum Wind) were considered which being 50 km, 45 km and 35 km. The overall computation choosing there three different values show small variation in the computed surge height. The cyclone having R-max of 50km produces height maximum value of peak surge.
Experiments 7 – 8 were carried for cyclones coming straight to three different coastal zones of Bangladesh. The tracks are assumed to be straight and at actual water depths. The maximum value of peak surges were different and if was found that cyclone land falling at central coastal zone perspective to Bangladesh is showing the highest maximum value of peak surge as compared to the other two coastal zones.

4. CONCLUSION

The IIT storm surge model has been simulated through 9 different experiments using synthetic data of Cyclones. Moreover the model was also tested with the actual data and the results obtained from the model are good agreement with reported data. Therefore maximum peak surge of any cyclone are depend on the meteorological parameters, oceanographic parameters, hydrological input as well as geographical parameter of perspective country of the world.

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TSUNAMI HAZARD ASSESSMENT IN THE NORTHERN BAY OF BENGAL ALONG THE BANGLADESH COAST

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ABSTRACT

This study identifies the major tsunamigenic fault sources in the Bay of Bengal and presents tsunami inundation maps under different scenarios. In this study, four potential tsunamigenic fault-sources (FS1 ~ FS4) were identified from geophysical and geological data along the India-Burma Plate faultlines from secondary information of geophysical and geological data. In addition, a fifth case was considered to analyze the impact of the continental shelf on tsunami attenuation. Two sets of numerical models were employed to estimate the tsunami height and intrusion extent at the coast. Maximum wave heights along four transects from the shoreline in the five cases indicate that in most cases shoaling occurs at the continental shelf margin. The maximum wave height at the shoreline would occur in Cox's Bazar for Case FS 1. Model results indicate that wave speed decreases from approximately 31 m/s at the edge of the continental shelf to 9-19 m/s along the shoreline. The maximum inundation is found at Nijhum Dwip in the range of 3-4 m, and in the Sundarbans area, Moheskhali and Cox's Bazar coast in the range of 1-3 m. Small islands and part of the Manpura island in the Meghna Estuary get inundated by 1-3 m.

Keywords: *Tsunami, Bay of Bengal, continental shelf, wave speed, inundation*

1. INTRODUCTION

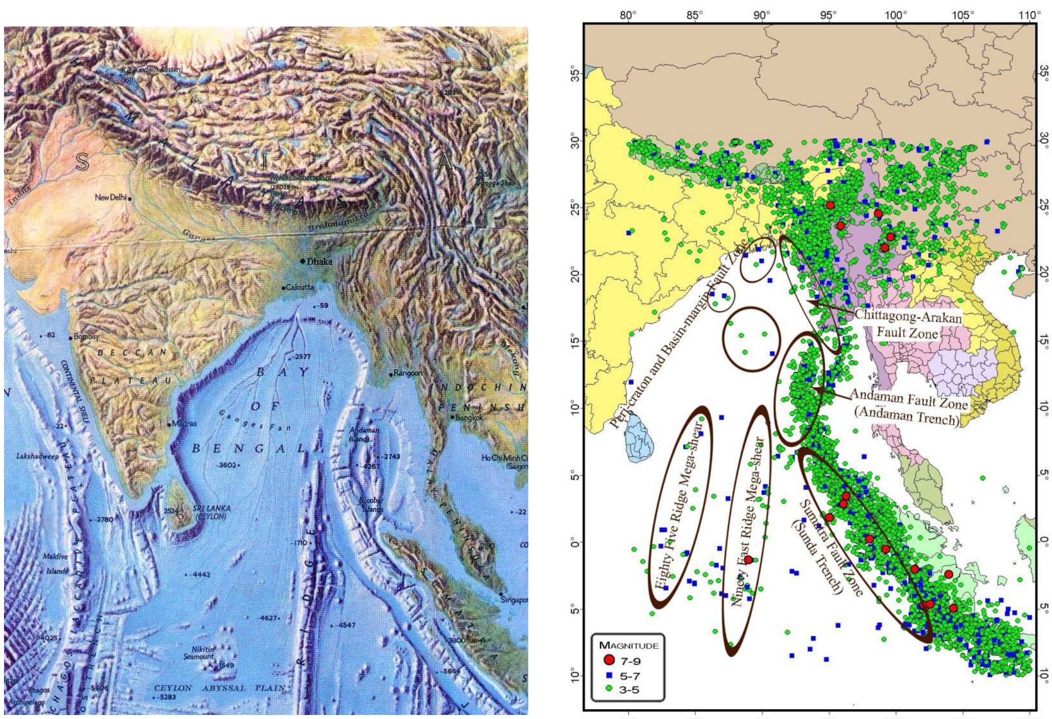
Tsunamis gained world-wide attention after the devastating tsunami of December 26, 2004 that originated off the coast of Sumatra and resulted in death of approximately 310,000 people and widespread damages in several countries around the Bay of Bengal and Indian Ocean including Indonesia, Thailand, Sri Lanka, Eastern India and Myanmar. The damage caused by this event to Bangladesh coast was limited. There were reports of 2 deaths and several capsized fishing trawlers resulting from 5 to 6-ft high waves in the coastal areas of Bangladesh. There were also reports that the shore waterline receded by about 50 meters in Saint Martin's island, and the water level fluctuated for about 3 hours, which are usual signs of a tsunami. The coastal fisheries were affected since the coastal water was very rough for a long time. Direct damage to fish habitat and ships in the Bangladesh coast was unknown.

Since damages to the northern Bay of Bengal and Bangladesh coast were very limited from the Sumatra tsunami, a popular hypothesis was that the northern Bay of Bengal coast was protected from tsunamis by the relatively long and shallow continental shelf. It was supported by the fact that the most wave energy from the nearly North-South aligned India-Burma plate subduction faultline would be released in the East-West direction, and relatively less wave energy would affect the northern Bay of Bengal coasts. However, historical accounts indicate that tsunamis have hit Bangladesh and adjoining coasts in the past, including a major tsunami that originated in the Arakan coast (Myanmar) in 1762 and inflicted significant damages. Therefore it was felt necessary that the threats of tsunami be investigated through a scientific research. This paper presnts the results from a collaborative research project initiated under an MoU between Institute of Water and Flood Management (IWFM) of Bangladesh University of Engineering and Technology (BUET) and Institute of Water Modelling (IWM). Other partners -Bangladesh Water Development Board (BWDB), Geological Survey of Bangladesh (GSB), Department of Geology, University of Dhaka, and Jadavpur University, Kolkata, India, were later invited to join the collaboration. The main objectives of this research were to: (i) devise scenarios, in terms of origin of tsunami, sea-level displacement and energy release orientation, which may have significant impact on the coasts around the northern Bay of Bengal; and (ii) estimate the wave height and inland intrusion extent of tsunami water in Bangladesh coast under different scenarios.

2. METHODOLOGY

2.1 Tsunamigenic Fault Sources in the Bay of Bengal and Scenarios

Possible origins of tsunami and maximum vertical displacements of the sea-floor along the India-Burma Plate faultlines were identified from secondary information. The orographic map of the Bay of Bengal (Figure 1,a) clearly demonstrates that the continental shelf-break occurs along the entire coast of the bay close to shoreline, starting from Sumatra to Sri Lanka, except Bangladesh where the shelf-break occurs about 200 km ocean-ward from the coast. In addition, the extended shallow bathymetric profile of the continental shelf plays a key role in flattening the waveform through a defocusing process (Ioualalen *et al.*, 2007) that would greatly reduce the impact of any trans-oceanic tsunami.



(a) (b)

Figure 1: (a) Orographic map of the Bay of Bengal and (b) fault source map of the region derived from seismological data

The earthquake distribution (Figure 1,b) in the Bay of Bengal clearly identifies various fault source zones that are intrinsically associated with the locations of earthquake epicenters. Further, the source zones are distinctly correlated with the occurrence of known tectonic elements such as Ninety East Ridge Mega-shear, Eighty Five Ridge Mega-shear (Raiverman, 1986), Sunda Trench, Andaman Trench, Chittagong-Arakan Fault Zone, and Peri-craton and Basin-margin Fault Zone (Khan and Akhter, 1999). Sumatra fault zone and Chittagong-Arakan fault zone are already geologically established fault zones. Andaman fault zone has also been documented as in Figure 2 as a major fault zone (Paul and Lian, 1975).

Figure 2: Andaman Fault zone derived from seismic section.

The regional structural configuration of the Bay of Bengal as envisaged from the vertical component magnetic anomaly map (Figure 3,a) is quite consistent with the regional crustal features of the Bay of Bengal like Ninety East Ridge, Eighty Five East Ridge, and the elevated crust in the northern Bay of Bengal.

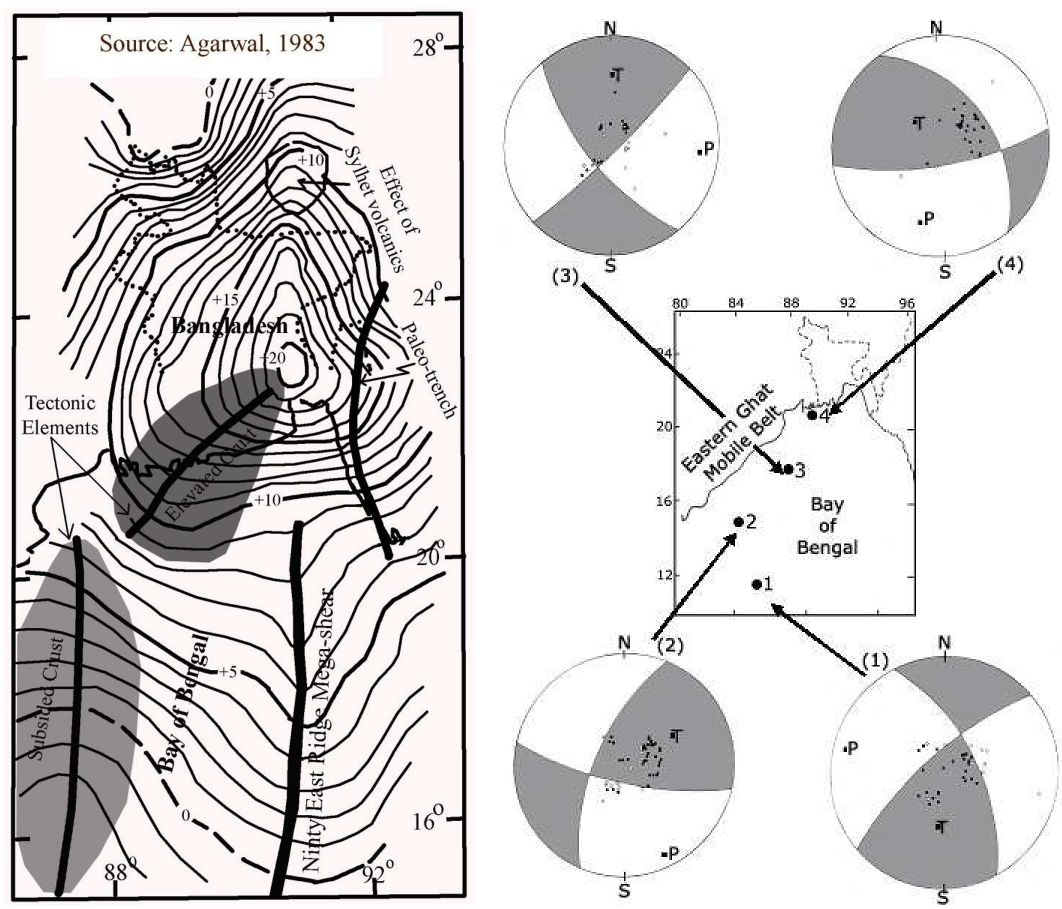


Figure 3: (a) regional crustal features of the Bay of Bengal derived from vertical magnetic data and (b) western region of the Bay of Bengal exhibiting a clear strike-slip fault mechanism.

East-West seismic section along 19^{o}N latitude between 87^{o}E and 91^{o}E longitudes (Figure 4) clearly exhibits a regionally long wavelength crustal feature representing an elevated and subsided older surface (Curray and Moore, 1971). Further, the normal faults occurring along the location of "Swatch of No Ground" in the 19^{o}N seismic section suggest that the zone occupied by the "Swatch of No Ground" is a faulted zone resulting from the upward bulge of the older crust. The western part of the Bay of Bengal along the Eastern Ghat Mobile Belt, the focal mechanism solutions of four earthquake events, also show pure strike-slip faulting (Khan, 1991) (Figure 3,b).

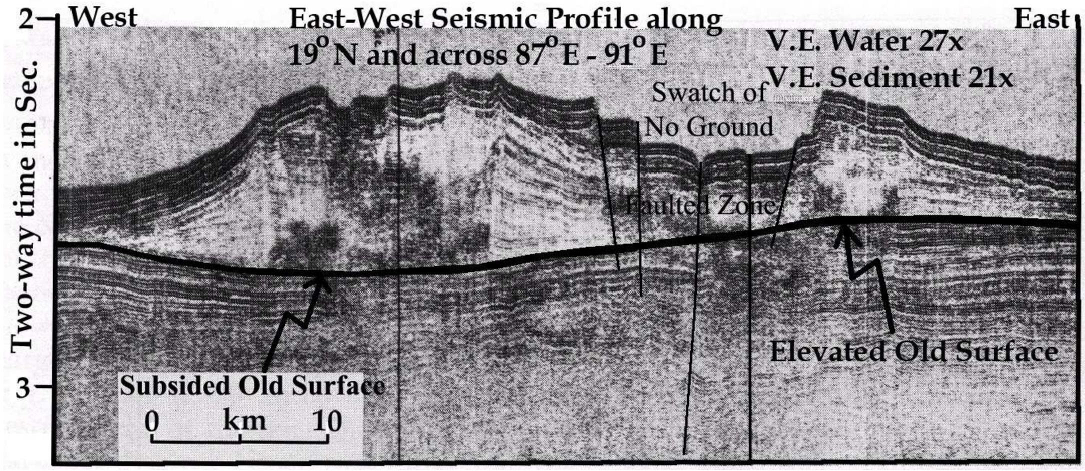


Figure 4: Seismic section showing crustal features of the Bay of Bengal.

Based on the aforesaid geophysical and geological data, the potential fault source map of the Bay of Bengal has been prepared and presented in Figure 5. It shows four potential tsunamigenic fault-sources. Four scenarios for tsunami modeling are also developed from the information on fault source parameters. These parameters of the fault sources such as the rupture length, slip offset, dip angle, slip angle, strike angle and the moment magnitude have been calculated from geophysical and seismological data. Parameters of the four scenarios or cases are presented in Table 1.

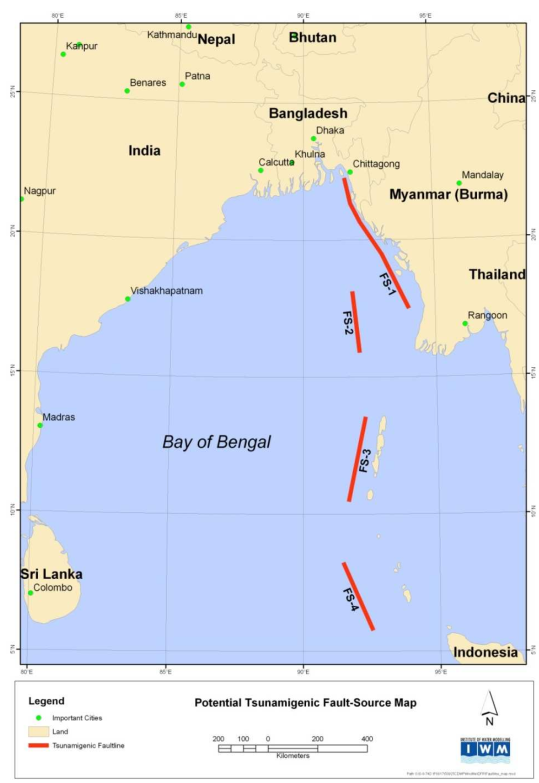


Figure 5: Potential tsunamigenic fault source map.

Table 1: Tsunami scenario/case parameters.

Potential sources of tsunami	Fault location	Fault segment's lenght (km)	Max fault slip(Δ) (m)	Initial rupture time	Fault dip angle (δ)(deg)	Fault slip angle (λ)(deg)	Fault strike angle (α)(deg)	Focal depth (d)(km)	Moment magnitude (Mw)
FS 1	(22N,91.7E)-(18.5N,93.5E)	410	3-7	0	30-40	45-65	340	10	8 (Potential)
FS2	17N, 92E	250	5	0	50	40	20	10	8
FS 3	12N, 92E	350	5	0	50	45	30	10	8(Potential)
FS 4	07N, 92E	300	9	0	40	50	320	10	8

* FS 4' (Sumatra tsunami without continental shelf)

2.2 Numerical Modelling of Tsunami Propagation

Two sets of numerical models were employed to estimate the tsunami height and intrusion extent at the coast. In the first set, a 3D hydrodynamic model simulated 'tsunami generation' and provided initial water surface as an input to a 2D 'tsunami propagation' model that simulated wave propagation. The propagation model was calibrated and validated for the 'Sumatra' tsunami of December 2004 at the Sri Lankan coast by Welhena (2006). This model was also applied to calculate tsunami propagation at the Cox's Bazar coast of Bangladesh by Hussain *et al.* (2008). The widely used ETOPO 2 global data by NOAA (2005) were used to generate the model bathymetry.

In the second modeling set, tsunami model was set-up using MIKE 21 modeling tool, which comprised four nested levels. In this set, a geological model, called QuakeGen, has been used for calculation of deformation in bed level based on the geophysical and seismological data. The initial surface has been generated using the hydrodynamic module of MIKE 21 modeling system based on the output from the QuakeGen model. The model describes the deformation of the bed level due to a double coupled model developed by Okada (1985). The tsunami model comprises four nested levels with the following grid sizes: Regional model having grid size of 16200m, Coarse model having grid size of 5400m, Intermediate model having grid size of 1800m and Fine model having grid size of 600m. The Meghna Estuary is resolved on a 600m grid resolution. The fine grid model domain covers the whole coastal region except Ukhia and Teknaf upazilas, while the coarse grid model covers Ukhia and Teknaf upazilas. The model is two-way nested and is driven through the release of the applied initial surface elevation only. The Regional model has been used to absorb energy at the boundary and to avoid reflection from internal boundaries. All the boundary conditions at the regional model have been set to zero. The Coarse grid model, which covers the Bay of Bengal, is the actual domain where initial surface deformation due to sub-sea earthquakes has been applied. The intermediate grid model serves only as a transition to the local fine grid model. The fine grid model is used for detailed mapping of the inundation and flood risk due to the tsunami.

The Regional grid model of tsunami was calibrated with the tsunami of December 26, 2004, which originated at the West coast of Sumatra. Manning number (M), reciprocal of Manning's roughness coefficient (n), is the main calibration parameter for this model. M was selected based on the values suggested by Aida (1977) and CERP (2000). The possible range of M found in the literature is rather wide. A value on the lower side was chosen to take into account the resistance by buildings, trees, roads, etc. Mangrove forests are known to give a very high energy loss. The Manning number for the mangrove forests were estimated on the basis of basic hydraulics combined with crude estimates of involved length scales. A land use map for the coastal area of Bangladesh was developed for estimation of M. It is found that the coastal area is covered mainly by agriculture, settlement and reserved forest. For simulation of tsunami, two categories of the land use have been considered - one is for agriculture and settlement, and the other is for the Sundarban reserved forest. A Manning number of 25 m1/3/s (n=0.04 s/m1/3) for the agriculture and settlement area, and 15 m1/3/s (n=0.07 s/m1/3) for the Sundarban reserve forest have been considered.

3. RESULTS AND DISCUSSION

3.1 Travel times and maximum wave height

From the first set of model simulations, travel times of tsunamis to reach Bangladesh coast in the four cases were computed from the 2D propagation model results. In addition to these four cases, a fifth case, FS 4': Sumatra tsunami without continental shelf, was considered to analyze the impact of the continental shelf on tsunami attenuation. Table 2 gives the calculated tsunami travel times in these five cases. The travel time is the minimum in Case FS 1 for Cox's Bazar, whereas it is the maximum in Case FS 3 for the Borguna coast and Meghna estuary. The travel time is almost the same for the entire coast in Cases FS 3 and FS 4. Travel time for Case FS 4' indicates that if there were no continental shelf, the tsunami waves from Sumatra would reach the Bangladesh coast 20 to 40 minutes earlier.

Table 3 gives the maximum wave heights of tsunamis in the five cases mentioned above. Wave heights have been presented at two locations: continental shelf margin and shoreline, along four transects from the shoreline. In most cases along a given transect, wave heights are higher at the continental shelf margin than at the shoreline. This indicates that shoaling occurs at the continental shelf margin.

Maximum wave heights vary in different cases and also at different locations along the coast (Table 3). The maximum wave height at the shoreline would occur in Cox's Bazar for Case FS 1. Comparison of wave heights and travel time for different cases indicates that a tsunami originating at the Myanmar coast would be the highest and would arrive the earliest. For Case FS 1, a tsunami having a maximum wave height of 0.98 m would arrive Cox's Bazar 10 min after originating at the Myanmar coast.

Table 2: Tsunami arrival time at Bangladesh Coast (minutes)

Sources	**Locations**			
	Swatch of No Ground	**Borguna Coast**	**Meghna Estuary**	**Cox's Bazaar**
FS 1	80	90	40	10
FS 2	100	110	100	70
FS 3	130	140	140	110
FS 4	120	130	110	100
FS 4'	90	90	90	80

Table 3: Maximum wave height (meters)

Locations	**On/off shore**	**Sources**				
		FS 1	**FS 2**	**FS 3**	**FS 4**	**FS 4'**
Swatch of No Ground	Cont. Shelf	0.11	0.17	0.06	0.01	0.14
	Shoreline	0.09	0.03	0.02	0.01	0.05
Borguna Coast	Cont. Shelf	0.33	0.16	0.27	0.06	0.14
	Shoreline	0.14	0.05	0.04	0.02	0.06
Meghna Estuary	Cont. Shelf	0.83	0.21	0.11	0.07	0.01
	Shoreline	0.33	0.14	0.11	0.06	0.02
Cox's Bazaar	Cont. Shelf	0.85	0.23	0.09	0.06	0.03
	Shoreline	0.98	0.12	0.04	0.02	0.02

3.2 Inundation mapping

From the second set of model simulations, inundation map for the coastal region of Bangladesh has been prepared based on MIKE 21 model results of the four cases or scenarios. The map has been prepared considering the land level from digital elevation model and the existing polders in the coastal region of Bangladesh. This was done following a previous study by IWM (2009).

Initially all the tsunamis generated from the potential sources were simulated using the MIKE 21 modeling system. Simulations were carried out for Mean Sea Level (MSL) condition and for Mean High Water Spring (MHWS) level conditions. In all the simulations only the MHWS condition shows the influence of tsunami along the coast of Bangladesh. Maximum inundation maps for all of the tsunami events (i.e. four scenarios) have been generated from the simulation results under MHWS condition. Finally the inundation map has been generated based on the maximum inundation maps using GIS tools.

To determine the MHWS level for the coast of Bangladesh, a map has been produced for the Bay of Bengal. The MHWS data at different locations have been taken from the Admiralty Tide Tables (1995). The MHWS level along the coast of Bangladesh has been calculated, which is 3 m MSL at the western coast and higher in the Sandwip channel. In this study 3.46 m PWD has been considered for the simulation of tsunami at the coast of Bangladesh.

Maximum inundation maps for four scenarios of tsunami show insignificant influence in the coastal region of Bangladesh under the MSL condition. However, some influence has been found under the MHWS tide condition. Figures 6 to 9 show the maximum inundation maps for the four scenarios of tsunami under the MHWS tide condition.

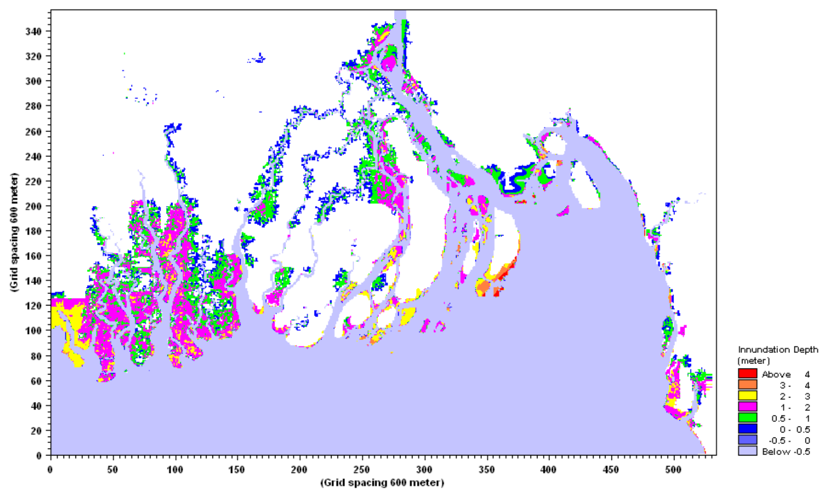


Figure 6: Maximum inundation map for scenario FS 1.

Tsunami scenario FS 1 has the source in front of Chittagong coast and propagates in all directions away from the coast. The tsunami inundates Nijhum Dwip and the floodplain of South Hatia (outside polder) in opposite direction in the range of 3-4 m, Noakhali coast, Urirchar and Sandwip in the northern direction in the range of 1-2 m, Cox's Bazar coast in the southern direction in the range of 1-3 m, and Sundarban coast in the range of 1-3 m. Small islands in the Meghna Estuary also get inundated due to this tsunami.

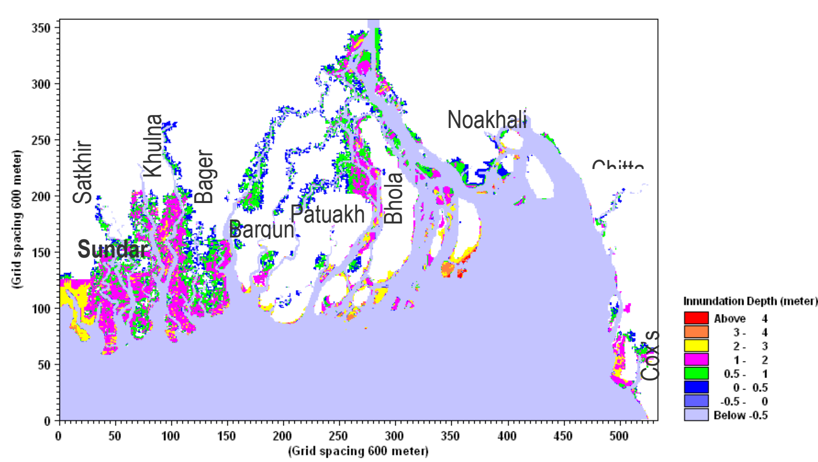


Figure 7: Maximum inundation map for scenario FS 2.

Tsunami under scenario FS 2 originates in front of the Myanmar coast and hits mainly the coast between Kuakata and Noakahali. The highest inundation (2-4 m) has been found at Nijhum Dwip. Other coasts like Sundarban and Cox's Bazar get inundated by about 1-3 m.

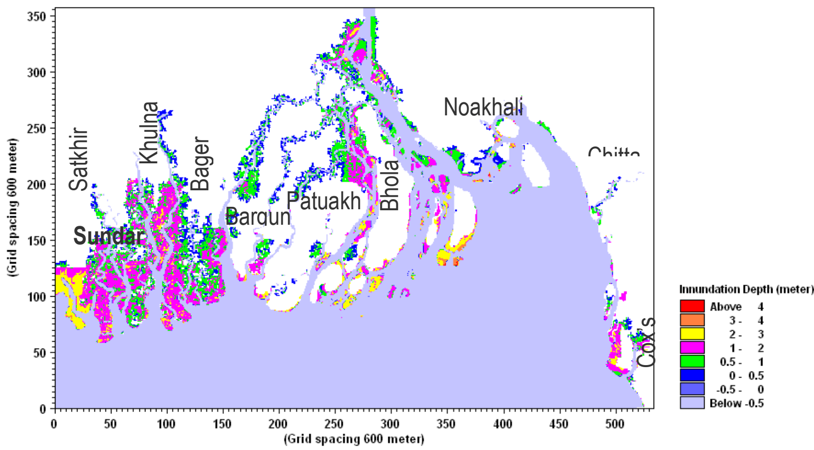


Figure 8: Maximum inundation map for scenario FS 3.

Tsunami under scenario FS 3 originates north of Anadaman island and propagates towards the coast of Bangladesh. It mainly affects the coast between Kuakata and Nijhum Dwip. Other areas have been found less affected in the simulation results.

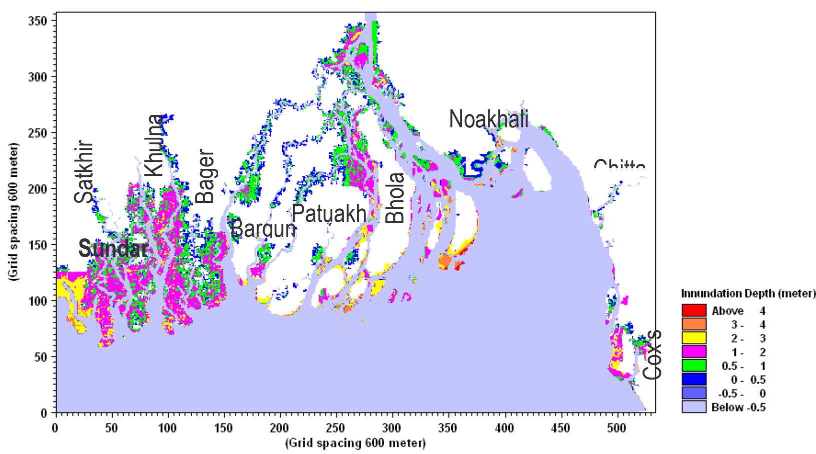


Figure 9: Maximum inundation map for scenario FS 4.

Tsunami under scenario FS 4 originates on the north-west side of Nicobar islands and hits the entire coast of Bangladesh. The highest range of inundation (2-3 m) has been found at Nijhum Dwip.

All the maps show that maximum inundation takes place in the Sundarbans area, Nijhum Dwip, Shonadia island, Bauphal upazila of Patuakhali district and small islands in the Meghna Estuary. The maximum inundation is found at Nijhum Dwip in the range of 3-4 m, and in the Sundarban area, Moheskhali and Cox's Bazar coast in the range of 1-3 m. Small islands and part of the Manpura island in the Meghna Estuary get inundated by 1-3 m. The maximum inundation at other locations remains within 1-2 m, which are mainly due to the MHWS tide. Bauphal upazila under Patuakhali district is one of them.

Tsunami level with travel time at five different locations of the Bangladesh coast has been extracted from the simulation results of four scenarios and presented in Table 4.

Tsunami level at Nijhum Dwip varies from 4.15 to 5.00 m PWD with a minimum travel time of 1 hour 45 minutes for scenario FS 1. Tsunami level at Sandwip varies from 3.65 to 4.5 m PWD with a minimum travel time of 20 minutes for scenario FS 1.

Table 4: Tsunami level and travel time under different scenarios.

Sources	Hiron Point		Kuakata		Nijum Dwip		Sandwip		Cox's Bazar	
	Tsunami level (mPWD)	Travel Time (Hr:Min)	Tsunami level (m PWD)	Travel Time (Hr:Min)	Tsunami level (m PWD)	Travel Time (Hr:Min)	Tsunami level (mPWD)	Travel Time (Hr:Min)	Tsunami level (mPWD)	Travel Time (Hr:Min)
FS 1	3.57	06:30	4.40	04:35	5.00	01:45	4.50	00:20	5.60	00:05
FS 2	3.64	02:45	4.79	03:20	4.29	04:15	3.70	04:40	4.08	02:30
FS 3	3.6	03:45	4.11	04:30	4.15	05:20	3.65	05:45	3.69	03:40
FS 4	3.8	04:00	4.45	04:35	4.45	05:40	3.89	06:30	3.91	04:10

The combined tsunami inundation map has been generated based on the maximum inundation maps of the four scenarios (Figure 10). The Sundarban area, Nijhum Dwip, south of Hatia (outside polder) and Cox's Bazaar coast are likely to be inundated during tsunami. The maximum inundation is seen at Nijhum Dwip in the range of 3-4 m, and in the Sundarban area and Cox's Bazar coast in the range of 1-3 m. Small islands and part of the Manpura island in the Meghna Estuary get inundated by 1-3 m. Bauphal upazila of Patuakhali district is a low lying area which may experience inundation of 1-2 m during MHWS tide. In this area the influence of tsunami wave is insignificant. The inundation map also shows that the Teknaf beach is likely to be inundated by a maximum depth of 4.5 m.

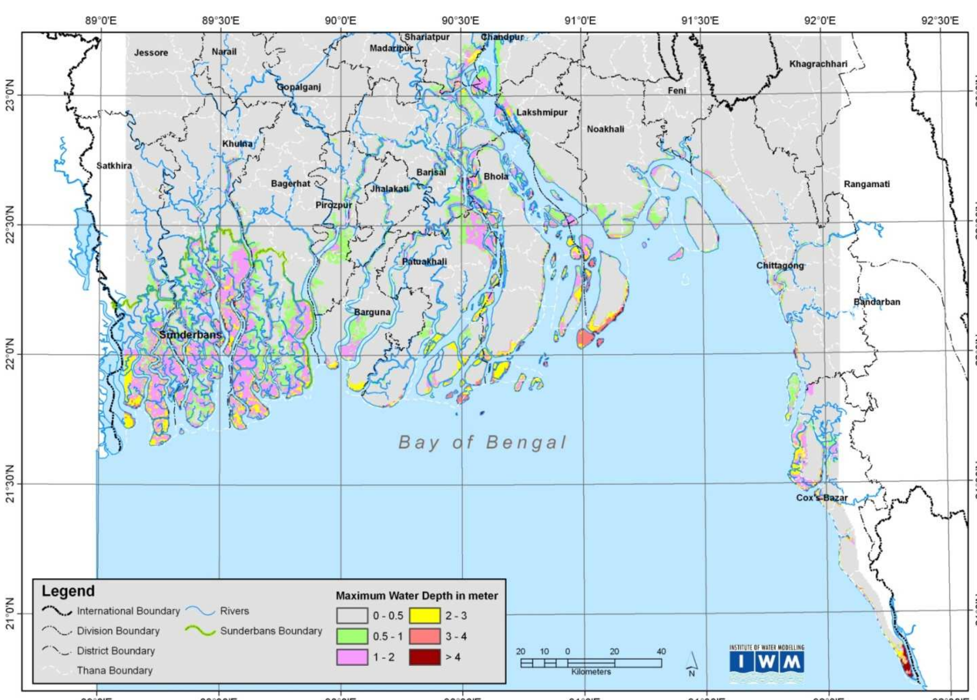


Figure 10: Combined inundation map of tsunami in the coastal region of Bangladesh.

4. Conclusions

This study identified the major tsunamigenic fault sources in the Bay of Bengal and presented tsunami inundation maps under different scenarios. This study also presented two sets of numerical models that would be useful to predict tsunami hazard under different conditions along the coast. The models may be used in a tsunami warning system in future, and in further detailed hazard mapping and risk analysis. It is expected that the findings of this study will be useful for planning disaster risk reduction measures in the coastal zone.

ACKNOWLEDGEMENTS

The collaboration from Mr. Jalaluddin M. Abdul Hye of Bangladesh Water Development Board (BWDB), Mr. Reshad Md. Ekram Ali of Geological Survey of Bangladesh (GSB), Professor Dr. Mahmood Alam of Department of Geology, University of Dhaka, and Dr. Gupinath Bhandari of Jadavpur University, Kolkata, India are gratefully acknowledged. We sincerely appreciate the cooperation of Professor Shinji Sato, University of Tokyo, for providing the 2D propagation model for the present research.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

MODELLING PROPOSED WATER DISTRIBUTION NETWORKS FOR WATER SUPPLY IN LOHAGORA MUNICIPALITY OF NARAIL, BANGLADESH

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ABSTRACT

In this study, three alternative models of water supply pipe networks (model-1: surface water source with treatment plant, model-2: groundwater source without treatment plant, model-3: groundwater source with treatment plant) in Lohagara municipality of Narail district have been developed based on the collected data and information. The simulation result of model-1 shows that one high lift pump having capacity of 335.54 m^3/hr and delivery head of 10m (abstracting water from clear water reservoir) is sufficient to maintain minimum residual pressure of 5m water column at any point of the distribution network up to 2040. The model-2 result shows that the daily water demand of 8,053 m^3/d at the end of design period (at 2040) can be met by only three production wells assuming a safe yield of 102 m^3/hr from each 150mm diameter production well. The analysis also suggests that location of these wells should be evenly distributed within the municipality area. The study also suggests that in case of model-3, location of production wells should be closer enough to the water treatment plant as well as minimum distance should be maintained between the production wells with a view to reduce the cost. Since the pipe network for study area has been modelled for different combinations of surface and groundwater sources with and without treatment plant, it is emphasized that attention should be given in selecting appropriate water source based on socio-economic aspects of the locality, environmental issues, taste of the consumers and cost effectiveness of the system.

Keywords: *Lohagara municipality, Narail, Water distribution network, Water supply, Water treatment plant*

1. INTRODUCTION

Over the last half-century, there has been an increasing trend of population settlement in urban areas especially in developing world. It has been predicted by United Nations that about 56% of the people in developing countries will be resided in urban areas (Karim and Mohsin, 2009; UNEP, 2002), which will certainly cause many problems due to increasing water demand by the growing populations. Therefore, supply of adequate safe water in the urban areas is a challenging task for the urban development authorities in the developing countries of the world. In the past, efforts were given only for the establishment of supplementary sources to meet the rising demand without considering the quality aspect. However, the most significant aspect of any water resources planning and management strategy is to ensure adequate supply of water with acceptable quality (Nobi and Das Gupta, 1997). Every year, a large amount of budget is invested around the world for providing or upgrading the piped water supply facilities. Even then, a vast population of the world is without safe piped water facilities. Nearly 80% to 85% of the cost of a water supply project is used in the distribution system (Swamee and Sharma, 2008). Therefore, design of water distribution system has attracted many researchers due to its involvement with huge cost. The purpose of water distribution design is to size and configure a system so that it meets existing and future demands while providing pressures above a minimum level for service (Filion et al., 2007; Mays, 2001; Mays and Tung, 1992). The distribution system is a vital component of every drinking water utility. Its primary function is to provide the required water quantity and quality at a suitable pressure, and failure to do so is a serious system deficiency (NRC, 2006). Modelling existing and future water demand remains the most challenging task in water distribution system design. To facilitate the process somewhat, geographic information systems (GIS) are increasingly being co-opted to assign water demand to network nodes based on user classifications such as residential, commercial, industrial, institutional, etc (Filion et al., 2007).

In Bangladesh, urban population is also rapidly increasing as a result of natural urban growth and migration from rural areas. The current urban population is about 38 million and will be reached to about 74 million by 2035 (BBS, 2005). Such growth of population in urban area will certainly impose a huge burden on urban water supply facilities, which may cause a large number of people to be lived without access to safe water supply in urban area of the country. The declining trend of available water supplies is one of the most important environmental concerns faced by the country at present. Department of Public Health Engineering (DPHE) in cooperation with City Corporation or municipality has installed distribution networks for water supply necessary to deliver water to the urban dwellers in major cities of Bangladesh (Karim and Mohsin, 2009). However, some additional projects of DPHE were also conducted to establish the water distribution networks in many other municipalities of the country. The present study was conducted in Lohagara municipality of Lohagara upazilla under Narail district of Bangladesh, where there was no distribution networks available at present for water supply to the residents of municipality (IWM, 2011). The main objective of this study is to establish a proposed water supply network in the Lohagara municipality based on the available demand and supply scenarios.

2. MATERIALS AND METHODS

2.1 The Study Area

Lohagara municipality is the only urban area of the Lohagara upazila of Narail district. It consists of nine wards and occupies a total area of 16.13 sq. km (Figure 1). It was established on 2000 and presently it is classified as a 'C' category municipality. Its population have been increasing since its inception. The relative importance of the town has ever been growing as a regional centre of trade and commerce. The water supply in Lohagara municipality mainly depens on groundwater as the source of water supply. The municipality is divided into 9 wards (Figure 1), which has no existing pipe networks for water supply purpose at present (IWM, 2011). It is situated on the bank of the Upper Nabaganga River in the south west region of Bangladesh.

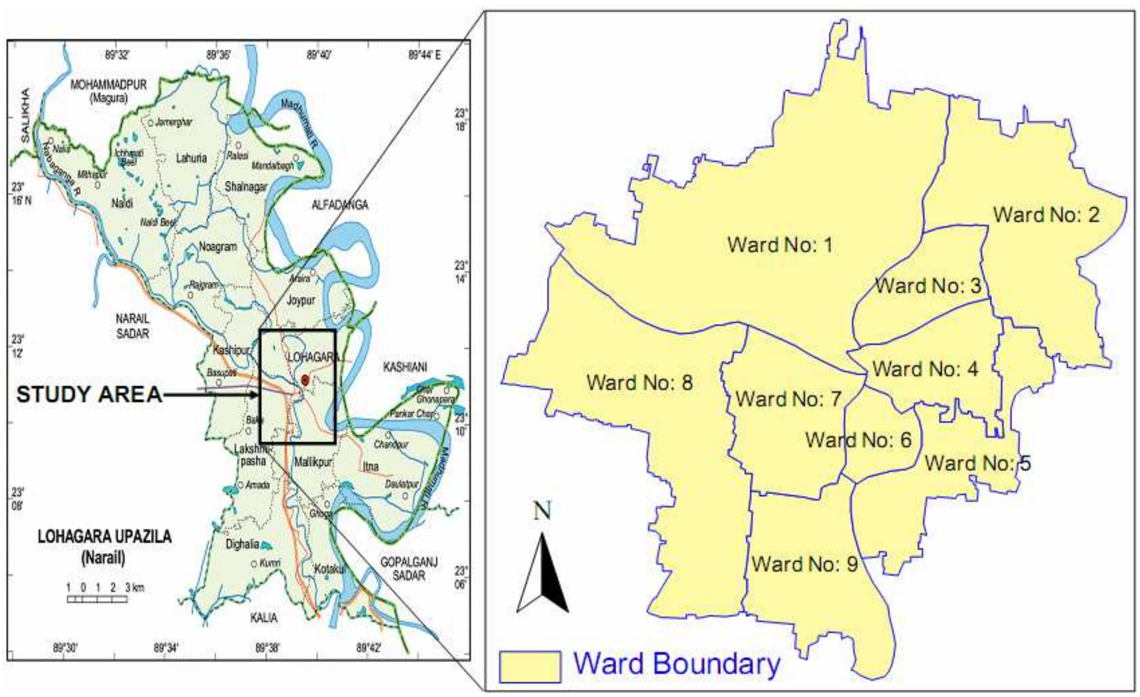

Figure 1: Location of Lohagara municipality in Lohagara upazila of Narail district

2.2 Data Collection

Study of water distribution network modelling requires input data from many sources. The data and information includes population projection, land use plan, assessment of water sources potentials and topographic information etc. Development and use of water distribution models comprises of many activities and processes. Extensive surveys were conducted to collect these data from the field. Input imprecision is propagated to uncertain model predictions. However, some of the data such as present water consumption and present population cannot be measured directly and it is rarely known precisely. In such cases, several established techniques are employed for estimating and quantifying the data and information, which are not directly available.

2.3 Projection of Population

For population projection, the geometric progression method is the most widely used method which is also known as empirical method. Population projection on the basis of 2.78 growth rate (Figure 4) is assumed natural increase rate with allowances for internal migration. As the rate of natural increase is expected to continue to be reduced and therefore, reduce population growth rates are considered for the projection periods and is presented in Table 1. From the known population /dwellings of at least two last censuses, growth rate can be estimated by method using the equation:

$$P_f = P_P\left(1+\frac{r}{100}\right)^n$$

Where, P_f = Future population or dwellings
P_P = Present population or dwellings
r = Growth rate of population or dwellings and
n = Design period.

Table 1: Estimated population in all wards of Lohagara municipality

Name of Locality	**Projected Population**							
	2001	**2010**	**2015**	**2020**	**2025**	**2030**	**2035**	**2040**
Pourashava	23,028	29,347	33,537	38,277	44,626	50,807	59,342	67,398
Ward No-01	3,617	4,559	5,220	5,987	6,871	7,872	9,128	10,504
Ward No-02	2,215	2,781	3,173	3,615	4,119	4,656	5,263	5,820
Ward No-03	2,344	2,965	3,431	3,939	4,531	5,176	5,904	6,654
Ward No-04	2,863	3,430	3,821	4,252	4,732	5,260	5,902	6,554
Ward No-05	2,451	2,948	3,313	3,736	4,222	4,740	5,377	6,038
Ward No-06	1,019	1,228	1,373	1,533	1,714	1,913	2,134	2,368
Ward No-07	3,035	3,632	4,042	4,488	4,988	5,519	6,163	6,837
Ward No-08	2,537	3,337	3,931	4,605	5,406	6,297	7,324	8,408
Ward No-09	3,416	4,465	5,234	6,123	7,171	8,381	9,896	11,661
Extension Area	-	-	-	-	872	993	2,250	2,556

According to GIS based Pourashava map, the area is estimated as 16.16 sq. km. and the population is 27,132. Comparing all the stated population figures with the predicted population figures for the year 2010 which is 29,347 seems to be on the right track and considered acceptable. Error of the report in case of feasibility level study should be around ±15% and in case of detail design, it is ±5%. This standard is being applied in many of the donor aided water supply projects of World Bank (WB) and Asian Development Bank (ADB) in Bangladesh. To arrive at this target, at least ward wise assessment of population growth rate is necessary for applying in population projection. In Lohagara Municipality, wardwise population for the year 2001 is available as this Pourashava is created after 1991 census. In absence of the previous two census (1981 and 1991) population, assessment of population growth rate is quite impossible. Hence, the present study picked up the population figures of 1991 and 1981 from the census reports of 1991 and 1981 for the existing mouzas /localities of the Pourashava. Area and population plays an important role for the assessment of population growth rate. To figure out the baseline (2001) growth rate in each ward, same localities of census 2001, 1991 and 1981 have been sorted out and placed in parallel to the wards of the Pourashava. Unfortunately, some of the localities could not be traced out probably for locality name change or the split of locality into more parts. However, ward wise growth rates have been calculated on the basis of the same area and same locality. Thus, a growth rate model of the Pourashava has been developed and are presented in Figure 2 and 3.

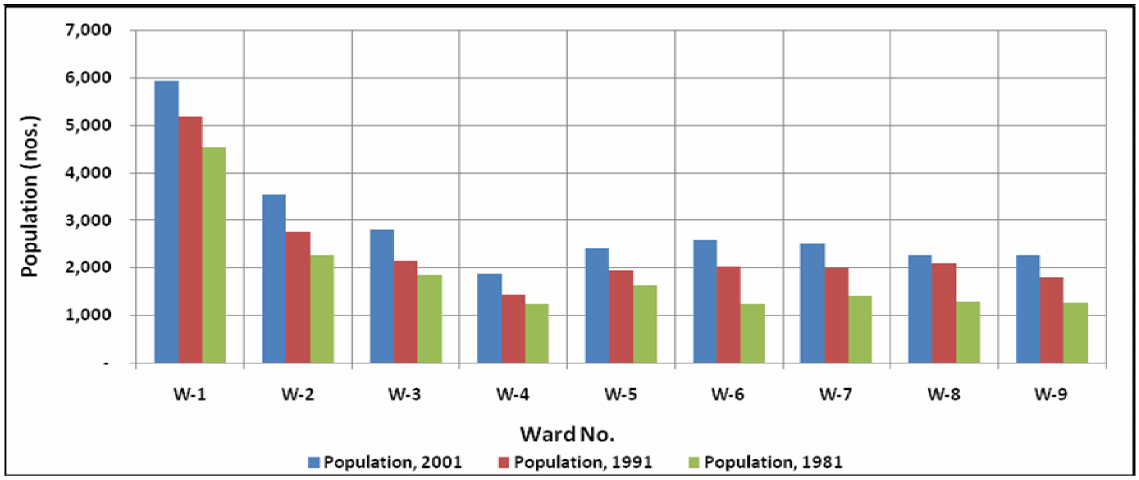

Figure 2: Ward-wise adjusted population for 1981, 1991, 2001 census in Lohagara Municipality

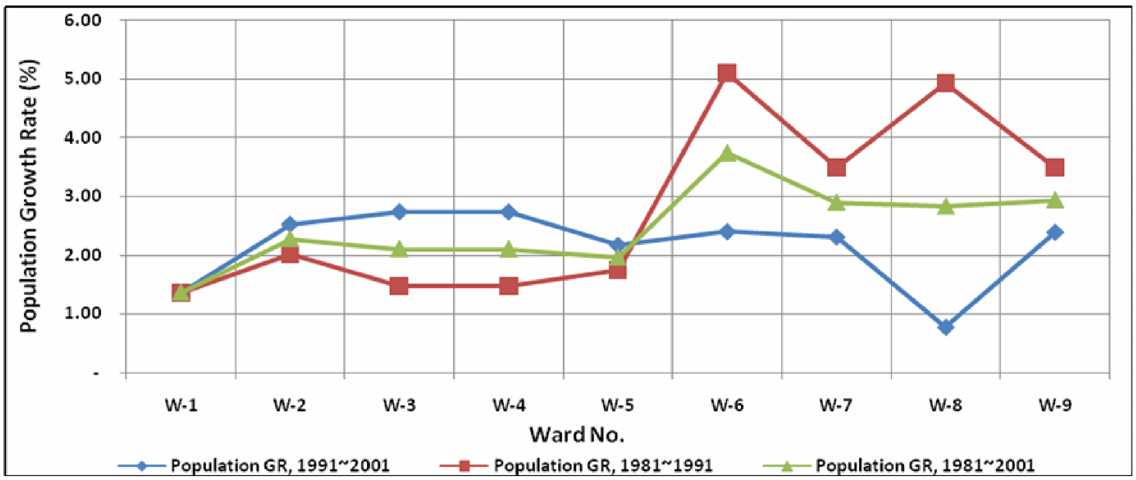

Figure 3: Ward wise population growth rate in Lohagara Municipality

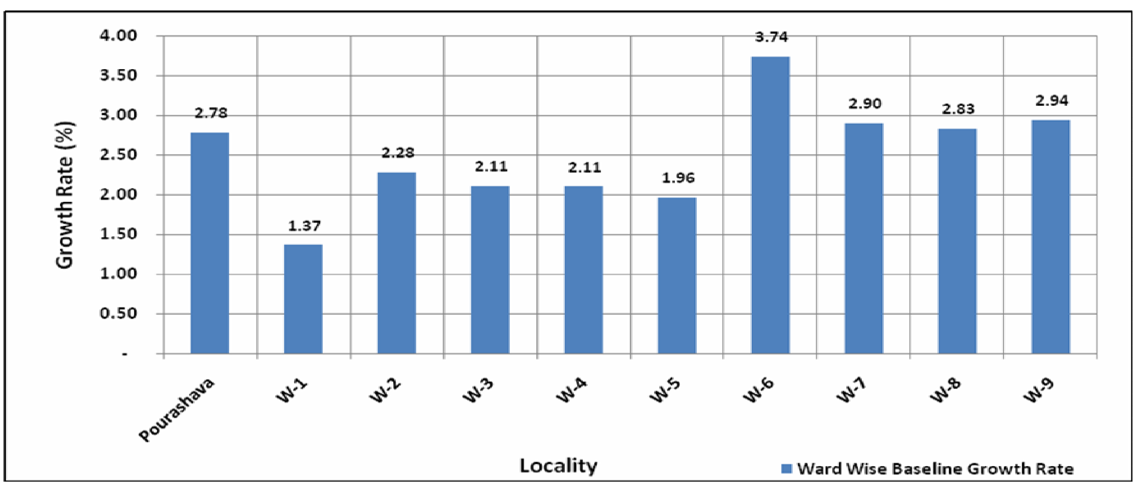

Figure 4: Wardwise baseline growth rate (%) in Lohagara Municipality

It reveals from the analysis that the growth rate varies significantly within the Municipality area and the urban area. Figures within the parenthesis indicate the negative growth rate. There is also found extremely high growth rate in some of the wards. These occur mainly due to frequent changes of Pourashava area, inclusion of new mouzas in different wards and inter change of mouzas within the wards. According to the census reports, the average population growth rate during the periods of, 1991-2001, 1981-1991and 1981-2001 are 2.06, 2.48 and 2.27 respectively whereas ward wise growth rate for the selected localities are in very high side. But growth rate calculated for 2.78 from census report to be acceptable for base line growth rate. There is a general consensus that the population growth rate 2.78 is expected to continue to reduce influencing the household size and households as well. The achievement of such low natural increase rates is attributed to the success of Family Planning Programs throughout the country. This is thought to be conservative due to the probability that higher education levels are associated with a greater awareness for the need of family planning. It has also been suggested that there is still significant scope for ongoing family planning impact. On the basis of the normal growth rate, highest growth of the wards and past experience, the baseline (2001) growth rates listed in Figure 4 has been assumed as applied in projecting the future growth rates up to the design period (2040).

2.4 Estimating Water Consumption and Demand

Assessment of domestic water consumption is not possible where there is no piped water supply as the consumers uses their daily water requirement from both groundwater (GW) and surface water (SW) sources even sometimes from rainwater source. Quantifying use of SW is just a guess. Here, designer judgement is predominant. Domestic water consumption has been proposed on the basis of the published literatures on water supply and water consumption analysis. Estimated domestic water demand and consumption are presented in Figure 5. Water demand up to the end of design period (2040) has been assessed considering intended population coverage by piped water supply system, consumers per connection for different types of service connections, population served through each types of service connections, non-domestic demand including the fire demand, water losses and backwash water (in case of water treatment plant).

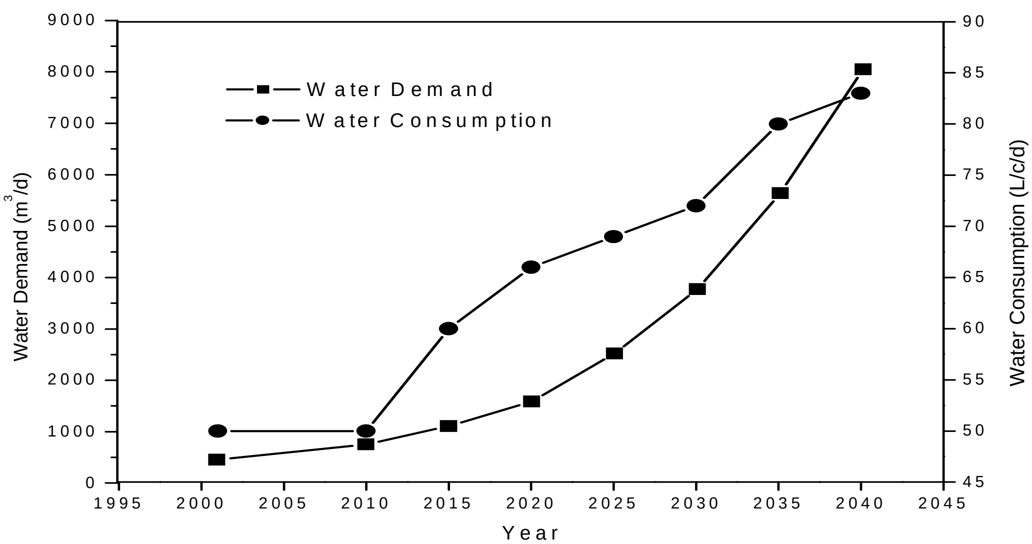

Figure 5: Estimated average domestic water demand and consumption in Lohagara Municipality

2.5 Assessment of Water Source Potentials

The flows in the Upper Nabaganga River can be used as a sustainable surface water (SW) source for a water treatment plant (WTP) for surface water, which can supply drinking water to the consumers of the municipality. Lohagara municipality intake point is located conveniently on Upper Nabaganga River, which is a schematized river of the South West Region Model (SWRM) developed earlier by IWM. The Upper Nabaganga is basically a spill channel from the Padma River, which drains the Gorai River basin. It has significant discharge, which reduces during the dry season. However, flows in Upper Nabaganga River were reported to be continuous all throughout the year. The river has linkages with the Ganges through other spill channel but carries lesser discharge. Considering the requirement for a WTP for surface water, the water demand of the municipality is estimated as 0.1001 m^3/s (100.1 lps) based on the dependable flow analysis using flow duration curve of the Nabaganga River for the period of 1986 to 2009. It is obvious from the result that the surface water is sufficient to satisfy the water demand for WTP. The municipality is characterised by a prominent aquifer at varying thickness of more or less 36m below the aquiclude at 200 m to 235 m. The estimate on the design year 2003 shows that the upper aquifer is semi-confined and lower aquifer is confined in nature. Based on the water balance studies, it can be concluded that the groundwater (GW) recharge is about 390 mm for the deeper aquifer in 2003, for which the corresponding annual aquifer storage volume is estimated as 7.25 Mm^3. It is also estimated that GW resources in the shallow aquifers of Lohagara municipality at May 01, 2003 for 6 m and 7 m depths are 7.48 Mm^3 and 10 Mm^3, respectively and at November 01, 2003, it is estimated as 11.29 Mm^3 and 13.81 Mm^3 for 6 m and 7 m depths, respectively (IWM, 2011).

2.6 Adopted Design Criteria

Adopted design criteria have been assumed (IWM, 2011) on the basis of the adopted design criteria of previous DPHE projects, field surveys, available literatures on water supply system and past experience and judgement. The adopted design criteria followed in the present sudy is presented in Table 2.

Table 2: Adopted design criteria

Parameters	**Mathematical Modelling of Water Distribution Networks**							
	2001	**2010**	**2015**	**2020**	**2025**	**2030**	**2035**	**2040**
(a) Design Period								
Design period (year)	30 from baseline year 2001							
(b) Population Coverage (%)								
Population coverage, % of total population of Pourashava	30.00	35.00	40.00	45.00	55.00	65.00	75.00	85.00
Population coverage by individual service connections, % of total population coverage	94.00	94.50	95.00	95.50	96.00	96.50	97.00	97.50
Population coverage by stand post, % of total population coverage	6.00	5.50	5.00	4.50	4.00	3.50	3.00	2.50
(c) Population served by each type of service connections (#/connection)								
Multi tap (people per tap)	9.63	9.84	9.96	10.07	10.19	10.31	10.44	10.56
Single tap (people per tap)	4.82	4.92	4.98	5.04	5.10	5.16	5.22	5.28
Shared tap (people per tap)	14.45	14.76	14.93	15.11	15.29	15.47	15.65	15.84
Street Hydrant (people per tap)	96.32	98.39	99.56	100.74	101.93	103.14	104.36	105.60
(d) Water Demand								
(i) Domestic Demand								
Multi tap (l/c/d)	70	75	80	85	90	95	100	105
Single tap (l/c/d)	40	45	50	55	60	65	70	75
Shared tap (l/c/d)	30	35	40	45	50	55	60	65
Street Hydrant (l/c/d)	15	17	19	21	23	25	27	29
(ii) Non-Domestic Demand (%)								
Total non domestic demand (NDD) (% of total domestic demand)	5.000	6.55	7.92	9.39	10.96	12.64	14.45	16.41
Peak day demand (% of average daily demand)	10.00	9.950	9.900	9.850	9.800	9.750	9.700	9.650
(iii)Unaccounted Water including fire demand								
Water Losses (% of domestic demand + non domestic demand)	10.00	12.36	14.15	16.09	18.19	20.50	23.04	25.86
(iv) Backwash Water								
Water requirement for backwashing of treatment units (% of TDD)	5.00	5.39	5.71	6.04	6.38	6.72	7.06	7.42
(e) Production & Treatment Hours								
Supply (hour)	0.00	0.00	8.00	11.00	14.00	17.00	20.00	23.00
Production (hour)	0.00	0.00	8.50	11.50	14.50	17.50	20.50	23.50
Treatment (hour)	0.00	0.00	8.50	11.50	14.50	17.50	20.50	23.50
(f) Storage								
Ground tank,% of TD	15%							
Overhead tank,% of TD	15%							

Parameters	Mathematical Modelling of Water Distribution Networks							
	2001	2010	2015	2020	2025	2030	2035	2040
(g) Distribution Network								
Peak factor for 24 hours supply	2.5							
Peak factor for 12 hours supply	1.25							
Peak factor for 8 hours supply	0.83							
Maximum Pressure (m)	15.0							
Minimum Pressure (m)	5.0							
Minimum diameter (mm)	100							

2.7 Developing Water Distribution Network Model

2.7.1 Nodal Demand Assessment

For nodal demand assessment, the area under each node, population density, assessment of service connections of each type and flow through each type of connection is needed. Linking all these spatial and non-spatial data has been completed by using the analytical ability of ArcGIS toolkit. Nodal demand calculation has been done by a spreadsheet program developed in MS Excel.

2.7.2 Digitization of Network Modelling Data

There are three types of data and information, which are essential for assembling a water distribution model**.**

(i) Network Data: Network data describes all physical components of the water distribution system and defines the interconnected patterns of those elements. Networks are made up of nodes and links. Nodes represent water system features at specific locations and links define relationships between nodes. Network data can include traditional data, which is mainly composed of two primary types – pipe and node data. Network geometry data are now generally available in the Geographical Information System (GIS) format.

- Intake and Water Treatment Plant (WTP) for SW Source: For modelling with SW source, an intake has been digitized along with a WTP beside the Nabaganga River. Treated water from the clear water storage reservoir with a rate of 97.28 lit/s with a delivery head of 10 m is to be injected to the distribution system. Ground elevation at pumping location is +5 m.PWD and hydraulic grade line level will be maintained as +10 m.PWD.
- Pumping stations: Pumping stations are needed for modelling with GW source. Based on the aquifer characteristics, required flow to meet the existing demand and total head needed, pump has to be selected carefully.
- Water Treatment Plant (WTP) for GW Source: In case of groundwater quality is not acceptable according to the WHO/Bangladesh water quality standard (WHO, 2006; ECR, 1997) WTP is needed to be constructed. Pumps will be used as the sources for WTP, which should be spaced about more than 1 km. apart so that no overlapping of influence zone occurs.

(ii) Water Demand Data: Water demand data describes two basic components of overall demand – metered or non-metered consumption and water losses from the distribution systems. Total daily water demand data is assigned to nodes in the modelled network. Modelling demand (consumption rates) and its distribution throughout the network is one of the key elements of a water distribution model. As such, the spatial distribution of demand and its variation over time must be carefully modelled.

(iii) Operational Data: Operational data describes actual operational system characteristics at a given time. Operational data is required to model water levels in reservoirs and status of pump stations. Operational data could be generally obtained from water utilities operational staff.

2.7.3 Model Building and Data Entry

Water distribution network model is assembled by applying two basic data entry procedures. At first, data is manually created by typing it into the model and then data has been transferred between various files by simply importing the data from one file to another, which also requires some additional manual editing techniques.

2.7.4 Model Calibration and Validation

Calibration is the process of adjusting model input data so that simulated hydraulics and water quality results adequately reflect observed field data. Calibration is an extremely important part of the modelling process of distribution systems. The process of calibration can be difficult, time-consuming, and costly. An accurate representation of the system and componetns is a must adequately to perform the calibration process. Two of the major sources of error in simulation analysis for hydraulics are demands (loading distribution) and pipe-carrying capacity. The importance of each of these error sources will depend on the network simulation. Use the measured values in a simulation model and adjust pipe friction factors followed by a simulation. Continue this process untill friction factors are selected that provide simulated pressures and flows that reflect the measured data. Other adjustments that may be necessary are adjusting pressure-regulating valves, redistributing demands, adjusting pipe diameters, and adjusting pump lifts. Calibration can also be performed by comparing measured water surface elevations in tanks with simulated elevations in tanks. It is adjustment of model aprameters so that predicted results adequately reflect observed field data (Mays, 2001).

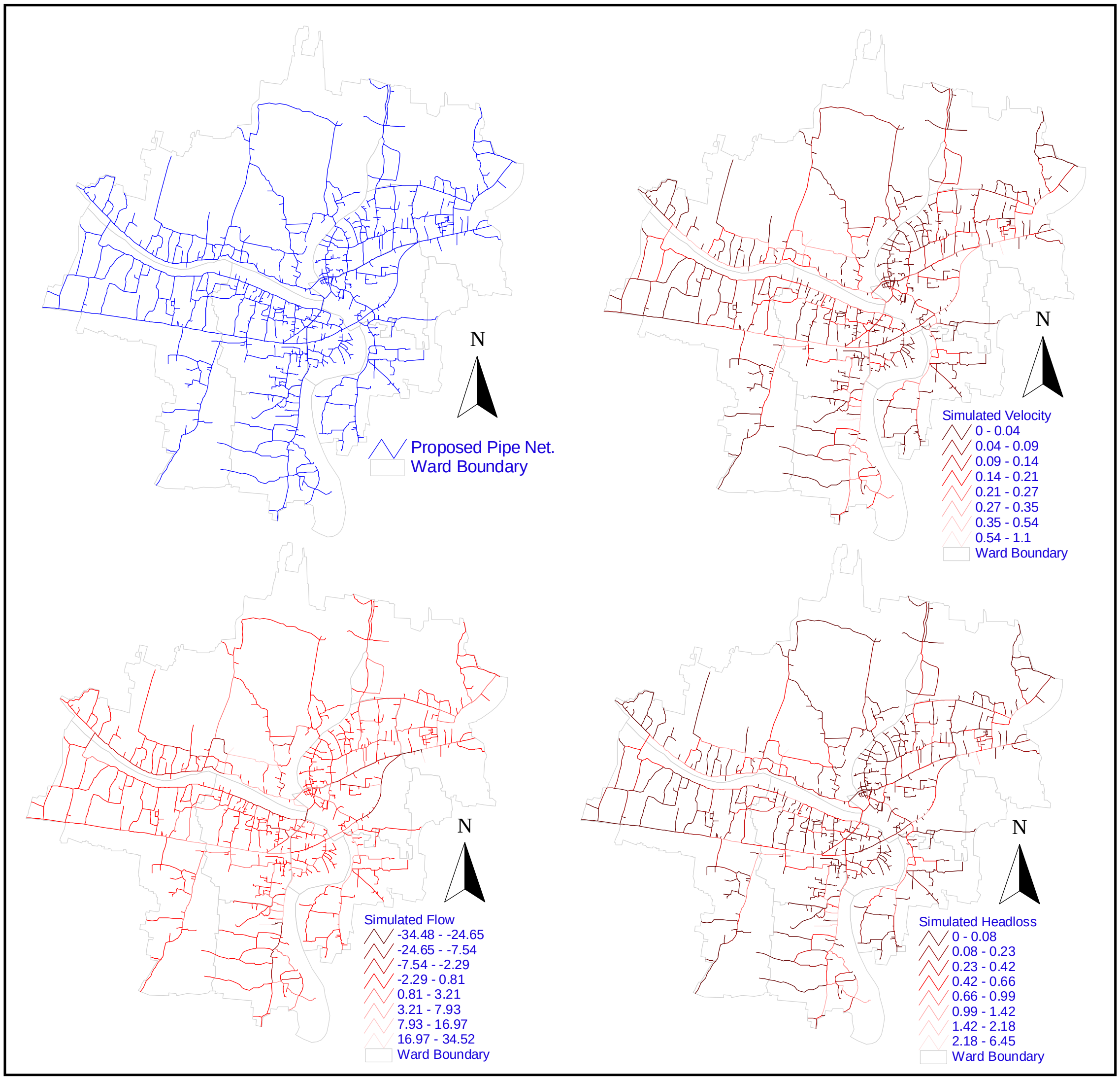

Figure 6: Simulated velocity, flow and headloss in proposed distribution network of Lohagara Municipality

Water Supply model calibration process involves adjustments of the following primary network model parameters: pipe roughness coefficients, spatial distribution of nodal demand, altering pump operating characteristics, pipe diameter and some other model attributes until the model results sufficiently satisfies the objectives. Model Validation is the process of independent comparison of model and field results to verify the adequacy of the model representation (Mays, 2001). Once a model is calibrated to match the project objectives, to gain full confidence in the model, model is validated using additional sets of data under different operational conditions. In performing validation, system demands, initial conditions and operational regimes need to be adjusted to match the conditions at the time the additional field data set was collected. The proposed water distribution networks and the simulated hydraulic parameters of those networks in Lohagara Municipality is presented in Figure 6.

3. RESULTS AND DISCUSSIONS

3.1 Water Sources for Model Simulation

Nabaganga River is flowing through Lohagara municipality of Lohagara upazilla under Narail district of Bangladesh. As both SW and GW sources are available, three models of water distribution pipe network for Lohagara municipality have been developed.

- the first model is built for SW source with water treatment plant,
- the second model is developed for GW source without water treatment plant and
- the third model is constructed by considering GW source with water treatment plant.

For both SW and GW sources with or without water treatment plant (WTP), one high lift pump (abstracting water from clear water reservoir) of capacity 335.54 m^3/hr at one bar (10 m) delivery head is sufficient to maintain minimum residual pressure of 5m water column at any point of the distribution system up to year 2040. For ground water source, the pipe network models have been built on the basis of the secondary data and the past experience in the field. Safe yield (abstraction) from one 150 mm diameter production well is 102 m^3/hr has been assumed and the location of these well are evenly distributed within the Lohagara Municipality area. On the basis of the above assumption, total of 3 numbers production wells will be required to meet the daily water demand of 8,053 m^3/d at the end of design period on 2040. It is estimated that selecting a pump of 102 m^3/hr at one bar (10 m) delivery head, will be sufficient to maintain minimum residual pressure of 5 m water column at any point of the distribution system up to the design year of 2040.

3.2 Surface Water Supply Model

In case of SW source for water supply, treatment of raw water is a must. Pipe network model for SW source starts from the clear water storage reservoir situated on the ground of the SW treatment plant. High lift pumping station will be needed to inject required quantity of treated water with certain discharge head to achieve the targeted minimum residual pressure (5.0 m water column) at any point of the distribution pipe network. Output of the model is presented in the form of junction data, pipe data and high lift pump data. High lift pump situated on the ground surface just after the clear water storage tank of the water treatment plant abstracting required quantity of treated water from the clear water storage tank will inject in the distribution system with certain discharge head to maintain the minimum residual head (5 m water column) at any point of the pipe network. The pipe network model is simulated by pipe network modeller. After each run of the pipe network model, the network is calibrated with pump discharge head and suitable pipe diameters to satisfy design criteria. The proposed pipe length for water supply in Lohagara Municipality is presented in Table 3.

Table 3: Wardwise proposed pipe length for SW supply model

Ward No.	**Pipe Length (km)**						**Grand Total (km)**
	100mm Dia	***150mm Dia***	***200mm Dia***	***250mm Dia***	***300mm Dia***	***350mm Dia***	
Ward No: 01	11.96	5.34	2.48	0.00	0.23	1.79	21.81
Ward No: 02	12.15	0.40	0.00	0.00	0.00	0.00	12.55
Ward No: 03	7.27	1.47	1.10	0.00	0.00	0.09	9.93
Ward No: 04	4.16	2.52	0.84	0.00	0.03	0.10	7.63
Ward No: 05	6.47	0.62	0.10	0.00	0.00	0.00	7.20
Ward No: 06	3.10	0.41	0.68	0.81	0.01	0.00	5.01
Ward No: 07	11.79	0.56	0.84	0.00	0.10	0.00	13.29
Ward No: 08	16.42	1.36	0.00	0.00	0.00	0.00	17.78
Ward No: 09	6.58	1.77	0.65	0.10	0.00	0.00	9.10
Total (km)	79.90	14.44	6.69	0.91	0.37	1.98	104.30

3.3 Ground Water Supply Model without WTP

A GW model generally consists of reservoirs, pumps, junctions and pipe network. In absence of GW source assessment result, model has been simulated by reservoirs only instead of pumps and reservoir combination as source. Like the SW supply model, similar representation approach has been followed in output for junction data and pipe data. Reservoir data is an additional output for GW supply model. The pipe network model is run by pipe network modeller. After the run, the network is calibrated with pump discharge head and suitable pipe diameters to satisfy design criteria. The proposed pipe length is presented in Table 4.

Table 4: Ward wise Proposed Pipe Length for GW Supply Model

Ward No.	Pipe Length (km)				Grand Total (km)
	100mm Dia	*150mm Dia*	*200mm Dia*	*250mm Dia*	
Ward No: 01	15.71	3.00	3.05	0.00	21.77
Ward No: 02	10.99	1.15	0.19	0.33	12.66
Ward No: 03	8.86	0.83	0.24	0.00	9.93
Ward No: 04	4.77	1.25	1.16	0.46	7.63
Ward No: 05	6.45	0.48	0.26	0.00	7.20
Ward No: 06	3.93	0.67	0.41	0.00	5.01
Ward No: 07	11.17	2.02	0.10	0.00	13.29
Ward No: 08	17.26	0.51	0.00	0.00	17.78
Ward No: 09	7.38	0.40	1.18	0.27	9.23
Total (km)	86.53	10.32	6.59	1.06	104.51

3.4 Ground Water Supply Model with WTP

In case of GW source, if the GW quality is not suitable for direct supply to the consumers, WTP is required to bring the water quality according to the WHO/Bangladesh water quality standard. As the source selection not finalized during the model built up, another model has been simulated for GW source with WTP. Pipe network model for GW source with treatment, starts from the clear water storage reservoir situated on the ground of the GW treatment plant. High lift pumping station will be needed to inject required quantity of treated water with certain discharge head to achieve the targeted minimum residual pressure (5.00m water column) at any point of the distribution pipe network. Output of the model is presented in the form of junction data, pipe data and high lift pump data. High lift pump situated on the ground surface just after the clear water storage tank of the water treatment plant abstracting required quantity of treated water from the clear water storage tank will inject in the distribution system with certain discharge head to maintain the minimum residual head (5.00m water column) at any points of the pipe network. The pipe network model is simulation by pipe network modeller and after each run, the network is calibrated with pump discharge head and suitable pipe diameters to satisfy design criteria. The proposed pipe length is shown in Table 5.

Table 5: Ward wise Proposed Pipe Length for GW Supply Model with WTP

Ward No.	Pipe Length (km)						Grand Total (km)
	100mm Dia	*150mm Dia*	*200mm Dia*	*250mm Dia*	*300mm Dia*	*350mm Dia*	
Ward no: 01	19.48	1.98	0.21	0.00	0.00	0.00	21.81
Ward No: 02	10.99	1.15	0.19	0.21	0.00	0.00	12.55
Ward No: 03	8.31	1.07	0.55	0.00	0.00	0.00	9.93
Ward No: 04	3.92	0.80	0.67	0.92	0.79	0.54	7.63
Ward No: 05	6.45	0.48	0.26	0.00	0.00	0.00	7.20
Ward No: 06	3.23	0.33	0.37	1.01	0.07	0.00	5.01
Ward No: 07	10.16	3.03	0.11	0.00	0.00	0.00	13.29
Ward No: 08	16.23	1.55	0.00	0.00	0.00	0.00	17.78
Ward No: 09	7.38	0.40	1.07	0.24	0.00	0.00	9.10
Total (km)	86.15	10.80	3.44	2.38	0.86	0.54	104.30

4. CONCLUSIONS

In the present study, three alternative models of water supply distribution networks have been developed for proposed water supply purposes in Lohagara municipality of Narail district have been developed based on the collected data and information. The first model refers to the combination of SW source with WTP, second model is relevant to the GW source without WTP, and third model is build by using GW source with WTP.

Based on the simulations and results obtained, a water distribution network is proposed for the Municipality area. Since the pipe network for study area has been modelled for different combinations of SW and GW sources with or without WTP, it is emphasized that proper attention should be given in selecting appropriate water sources based on the socio-economic aspects of the locality, environmental issues, taste of the consumers and cost effectiveness of the system. The simulation result of the first model shows that one high lift pump having capacity of 335.54 m^3/hr and delivery head of 10 m (abstracting water from clear water reservoir) is sufficient to maintain minimum residual pressure of 5 m water column at any point of the distribution network up to the end of design period 2040. The second model simulation result reveals that the daily water demand of 8,053 m^3/d at the end of design period (at 2040) can be met by only three production wells assuming a safe yield of 102 m^3/hr from each 150 mm diameter production well. The analysis also suggests that location of these wells should be evenly distributed within the municipality area. According to the third model simulation of water distribution networks, production wells should be located in such a way that they are closer enough to the WTP as well as minimum distance should be maintained between the production wells with a view to reduce the cost of water withdrawal and/or water production. As depicted in the design criteria section, minimum water pressure at the consumers end has been ensured not less than 5 m water column. This indicates that the owner of the multi-storeyed buildings need underground water reservoirs. However, for one storey buildings, no underground reservoir is necessary. From the analysis of the network, the hydraulic properties of each node and pipe are found from the software used for the modelling task. The detailed results of these node and pipes with selected hydraulic properties such as demand, hydraulic grade and pressure as well as pipe hydraulic properties such as diameter, velocity, head loss gradient and flow are provided for assisting in their calculation from time to time in future.

ACKNOWLEDGEMENTS

The authors wish to thank the Water Resources Planning (WRP) Division of the Institute of Water Modelling (IWM), Dhaka, Bangladesh, Lohagara municipality and Department of Public Health Engineering (DPHE) for extending all the necessary supports to complete the study. Special thanks to S.M. Mahbubur Rahman, Director of WRP Division, IWM for his continuous encouragements and supports for this study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

NUMERICAL SIMULATION OF DEAD ZONE FLOWS IN AN OPEN CHANNEL WITH A SIDE CAVITY AND SUDDEN ENLARGEMENT

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ABSTRACT

Three–dimensional numerical simulation is carried out to study the fundamental properties of flows in an open channel with a dead zone. In this paper, the flow field in two types of dead zone have been studied: one is rectangular side cavity and another one is the dead zone created at the downstream of channel with width enchroachment (i.e. sudden enlarged zone). A non-linear k-ε model is employed using finite volume method with a curvilinear coordinate system. It is found that the flow in the side cavity is characterized by three types of flow phenomena: the circulation inside the dead-zone, periodic coherent vortices at the interface of main stream and dead zone, and the water surface oscillation inside the dead zone. In this study all these characteristics are successfully reproduced by numerical simulation. The time averaged flow properties and temporal change of velocity profiles are well compared with previous experimental results. With increasing the length of the cavity, the circulation pattern and their numbers are changed along with velocity magnitude. For the open channel with sudden enlargment, although the upstream arrangemnet of flow domain is same as the cavity flow, the flow in the enlarged zone has the freedom to flow towards downstream in logitudinal direction. Due to this free downstream, the simulated flow field in the sudden enlarged zone is significantly different than that of open channel flows with side cavity. Beside the academic interests, present study contributes to understand the developement, extent and behaviour of flows in a dead zone for proper managemnet of river system and other water courses.

Keywords: *Open channel flow, dead-zone, side cavity, sudden enlargement, Unsteady RANS, non-linear k–ε model*

1. INTRODUCTION

The floodplain encroachment in a river is generally created due to construction of hydraulic structures such as embayment, spur-dykes etc. along the river for flood protection, navigation, bank protection, protection of bridges etc. The obstructed flow field in the downstream of a single spur-dyke or the flow field enclosed by two consecutive groynes are low velocity region compared to mainstream. This type of zone is generally termed as dead-zone. Flow velocity inside such kind of arrangement is lower than main channel. In addition to engineering applications, such structures increase the biodiversity of aquatic species by creating habitat and providing shelter for them. The main stream of a river, where the velocity is high, is not suitable for weak and small fishes. That's why the dead zone is a suitable shelter for them.

2. BACKGROUND OF THE STUDY

In this study the flow field in two types of dead zone have been studied: one is rectangular side cavity and another one is the dead zone created at the downstream of channel with width encroachment (i.e. sudden enlarged zone). They are briefly described below.

2.1 Rectangular Side Cavity

It is an enclosed portion in a side of open channel where there is no longitudinal flow from upstream and no downstream flow from the cavity; the flow enters laterally from the main stream and there is a lateral interchange of flow between mainstream and cavity. Figure 1 shows two field photographs of typical flow fields in a side cavity. Figure 2 shows a typical sketch of a rectangular side cavity, where the dead zone like flow is generated.

2.2 Dead Zone at the downstream of enchroached width in a open channel

In Bangladesh, the typical river encroachment means the earth filling of a portion of river course along the bank and pushing of river bank towards the center of river. Downstream of the fill, it creates a sudden enlarged portion along the river side. Such sudden enlarged zone due to width enhroachment are mainly observed in small rivers running through the cities or towns. Such filling may be continuous along the stream or discontinuous.

Figure 1 : An example of flow behavior in rectangular side cavity (Muto et al.., 2002)

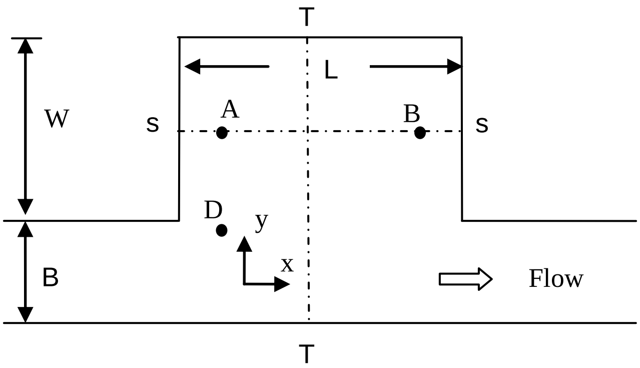

Figure 2 : Sketch of the flow domain with rectangular side cavity (L=length of the cavity, W=width of the cavity, B= width of the main channel)

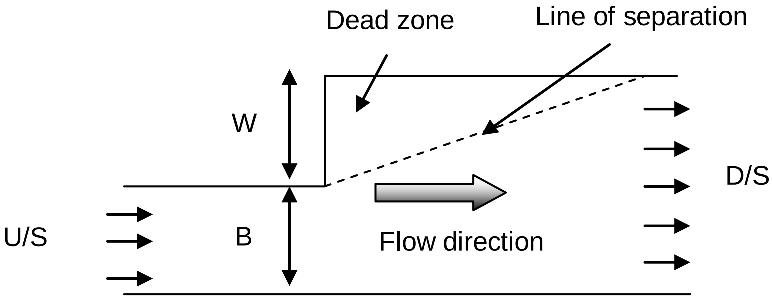

Figure 3 : Definition sketch of open channel with sudden enlargement
(B=Encroached width, W=Enlargement in width)


(a) Mayur River in Khulna


(b) Balu River in Dhaka

Figure 4 : Typical River encroachment in two city rivers in Bangladesh

This type of encroachment causes the reduction of channel cross-section, and thus the conveyance capacity of the river. It also causes the morphological changes of the river course; scouring occurred at the encroached section and silting or deposition of sediment are observed at the enlarged area. At the lee of the encroachment where the river suddenly enlarged, dead-zone like area is developed. Since the dead zone is a low velocity region, the sediment carried by the stream get deposited in that region. Therefore the downstream of the obstruction (earth fill) gradually becomes shallow compared to water depth at mainstream. Fig. 3 shows the definition sketch of open channel with sudden enlargement. The typical line of flow separation and dead zone is indicated in the figure. Fig.4 shows an example of river encroachment by earth filling in river in Bangladesh. Figure 4(a) shows that the vegetation is grown in the shallow depth region downstream of a obstruction in the river, and Fig. 4(b) shows the enchroached is river width due to sand filling inside the river along the bank.

In this paper, the unsteady flow fields in a rectangular side cavity and that of sudden enlarged zone are simulated by unsteady RANS to investigate the characteristics of unsteady flow behaviour in a dead zone.

3. NUMERICAL MODEL AND SIMULATION DETAILS

3.1 Non-linear *k–ε* Model in 3D Curvilinear Coordinate

The 3D flow equations in a *k–ε* model for an unsteady incompressible flow with contravariant components of velocity vectors on a curvilinear coordinate system can be written as follows.

Continuity Equation:

$$\frac{1}{\sqrt{g}}\frac{\partial V^{\alpha}\sqrt{g}}{\partial \xi^{\alpha}} = 0 \qquad (1)$$

Momentum equation:

$$\frac{\partial V^i}{\partial t} + \nabla_j\left[V^i(V^j - W^j)\right] + V^i\nabla_j W^j + V^j\nabla_j W^i = F^i - \frac{1}{\rho}g^{ij}\nabla_j p + \nabla_j\left[-\overline{v^i v^j}\right] + 2\nu\nabla_j e^{ij} \qquad (2)$$

k-equation:

$$\frac{\partial k}{\partial t} + \nabla_j\left[k(V^j - W^j)\right] + k\nabla_j W^j = -g_{il}\overline{v^l v^j}\nabla_j V^i - \varepsilon + \nabla_j\left\{\left(\frac{D_t}{\sigma_k} + \nu\right)g^{ij}\nabla_i k\right\} \qquad (3)$$

ε-equation:

$$\frac{\partial \varepsilon}{\partial t} + \nabla_j\left[\varepsilon(V^j - W^j)\right] + \varepsilon\nabla_j W^j = -C_{\varepsilon 1}\frac{\varepsilon}{k}g_{il}\overline{v^l v^j}\nabla_j V^i - C_{\varepsilon 2}\frac{\varepsilon^2}{k} + \nabla_j\left\{\left(\frac{D_t}{\sigma_k} + \nu\right)g^{ij}\nabla_i \varepsilon\right\} \qquad (4)$$

Here, V^j is the contravariant component of velocity vector of the flow and W^j is that of the grid motion. g_{ij} and g^{ij} are covariant and contravariant component of metric tensor. The covariant derivative is defined as

$$\nabla_i A^k = \frac{\partial A^k}{\partial \xi^i} + A^j\Gamma_{ij}\ ,\ \Gamma_{ij}{}^k = \left\{\begin{matrix} k \\ i\ j \end{matrix}\right\} = \frac{1}{2}g^{km}\left(\frac{\partial g_{jm}}{\partial \xi^i} + \frac{\partial g_{im}}{\partial \xi^j} - \frac{\partial g_{ij}}{\partial \xi^m}\right) = \frac{\partial \xi^k}{\partial x^p}\frac{\partial^2 x^p}{\partial \xi^i \partial \xi^j} \qquad (5)$$

here, Γ_{ij}^{k} is the Christoffed symbol.
The constitutive equation for 2nd-order non-linear k–ε model used in this study is as follows

$$-\overline{v'^i v'^j} = D_t S^{ij} - \frac{2}{3} k \delta_s^i g^{sj} - \frac{k}{\varepsilon} D_t [\alpha_1 Q_1 + \alpha_2 Q_2 + \alpha_3 Q_3], \quad D_t = C_\mu \frac{k^2}{\varepsilon} \tag{6}$$

here,

$$Q_1 = S^{i\alpha} g_{\alpha l} \Omega^{lj} + S^{j\beta} g_{\beta l} \Omega^{li}, \; Q_2 = S^{i\alpha} g_{\alpha l} S^{lj} - \frac{1}{3} S^{k\alpha} g_{\alpha m} S^{m\beta} g_{\beta k} \delta_l^i g^{lj}, \; Q_3 = \Omega^{i\alpha} g_{\alpha l} \Omega^{lj} - \frac{1}{3} \Omega^{k\alpha} g_{\alpha m} \Omega^{m\beta} g_{\beta k} \delta_l^i g^{lj} \tag{7}$$

Strain and rotation tensors are defined as

$$S^{ij} = g^{j\alpha} \nabla_\alpha V^i + g^{i\alpha} \nabla_\alpha V^j, \quad \Omega^{ij} = g^{j\alpha} \nabla_\alpha V^i - g^{i\alpha} \nabla_\alpha V^j \tag{8}$$

Considering the coefficients of non-linear terms as a function of strain and rotation parameters, their values can be determined as follows

$$\alpha_1 = -0.1325 f_M, \; \alpha_2 = 0.0675 f_M, \; \alpha_3 = -0.0675 f_M, \; f_M = \frac{1}{1 + m_{ds} S^2 + m_{d\Omega} \Omega^2} \tag{9}$$

Here, the functional form of c_μ is assumed as

$$c_\mu = \frac{c_{\mu o} (1 + c_{ns} S^2 + c_{n\Omega} \Omega^2)}{1 + c_{ds} S^2 + c_{d\Omega} \Omega^2 + c_{ds\Omega} S\Omega + c_{ds1} S^4 + c_{d\Omega 1} \Omega^4 + c_{ds\Omega 1} S^2 \Omega^2} \tag{10}$$

Here, the strain and rotation parameters are defined as

$$S = \frac{k}{\varepsilon} \sqrt{\frac{1}{2} S^{i\alpha} g_{\alpha j} S^{j\beta} g_{\beta i}}, \; \Omega = \frac{k}{\varepsilon} \sqrt{\frac{1}{2} \Omega^{i\alpha} g_{\alpha j} \Omega^{j\beta} g_{\beta i}}. \tag{11}$$

The model constants are given in Table 1(for details please see in Kimura et al., 2009 and Ali, 2008).

Table 1: Values for the coefficients of c_μ and c_β

Model Const.	$c_{\mu 0}$	c_{ns}	$c_{n\Omega}$	c_{ds}	$c_{ds\Omega}$	$c_{d\Omega}$	c_{ds1}	$c_{d\Omega 1}$	$c_{ds\Omega 1}$	m_{ds}	$m_{d\Omega}$
values	0.09	0.005	0.0068	0.008	-0.003	0.004	0.00005	0.00005	0.00025	0.01	0.003

3.2 Computational Schemes and Flow Domain

The governing equations for mean velocities and turbulent flows are discretized with the finite volume method based on full staggered boundary fitted coordinate system. For the momentum equation, convective and diffusive fluxes are approximated with Quick and central difference schemes respectively. The hybrid central upwind scheme is used for the k and ε equations. Time advancement is achieved by Adam-Bashforth scheme of second-order accuracy in each equation. The basic equations are discretized as fully explicit forms and solved successively with the time increment in step by step. The pressure field is solved using iterative procedure at each time step. The free surface elevation is solved by continuity equation integrated over the control volume of the surface layer. The wall functions are employed as the wall boundary conditions for k and ε. The frictions near the bed and side walls are estimated by log-law. An inflow velocity is prescribed in upstream end, and fixed depth with zero velocity gradient is used as boundary condition at downstream end of the flow domain.

The numerical simulations for 3-D unsteady flows are performed for three cases: two cases (C1 and C2) for open channel with rectangular side cavity and one case (E1) for sudden enlarged zone. The hydraulic conditions for Case C1 and C2 are shown in Table 2. Case C1 is simulated under the same conditions of laboratory experiments conducted by Kimura and Hosoda (1997). Case C2 is simulated to investigate the effect of length to width ratio (L/W) on the flow field. The plan view of the computational domain of side cavity is shown in Fig. 2, and that of sudden enlarged channel is shown in Fig. 3. The flow domain consists of 76 grids in longitudinal (stream-wise, x), 42 in transverse (width-wise, y) and 10 in perpendicular to bed (depth-wise, z) directions. The simulated results are presented in the following two sections.

Table 2: Hydraulic parameters for the simulations of open channel flows with a rectangular side cavity

Case no.	**B (cm)**	**L (cm)**	**W (cm)**	**L/W ratio**	Q_0 **(cm³/s)**	**ho (cm)**	**Fr no.**	**Bottom slope**	Δt **(sec)**
C1	10	22.5	15	1.5	747.0	2.02	0.83	1/500	0.0001
C2	10	45	15	3	747.0	2.02	0.83	1/500	0.0001

Table 3: Hydraulic parameters for the simulations of open channel flows with a sudden enlarged zone

Case no.	B (cm)	L (cm)	W (cm)	L/W ratio	Q_0 (cm^3/s)	ho (cm)	Fr no.	Bottom slope	Δt (sec)
E1	10	135	15	9	255	1.00	0.81	1/1000	0.0001

4. FLOW CHARACTERISTICS IN A SIDE CAVITY

4.1 Time Averaged Flow Properties

The unsteady flow in a dead zone is characterized by three important flow criteria: the circulation and oscillation inside the dead zone, and the coherent vortices at the interface of main stream and dead zone. However, the time averaged flow does not show instability characteristics. Figure 5 shows the time averaged (averaging for 10 sec) velocity vector, which mainly characterized by the circulation inside the dead zone. The simulated vector field and the circulation pattern are very similar to the previous experimental and 2D numerical studies (Kimura and Hosoda, 1997; Takemoto et al., 1984).

Figure 6 shows the time averaged profile of stream-wise velocity (u) along the transverse cross-section at centerline of dead zone (the section is shown in Fig. 2 as T-T). The comparison of simulated result with experiment shows good agreement.

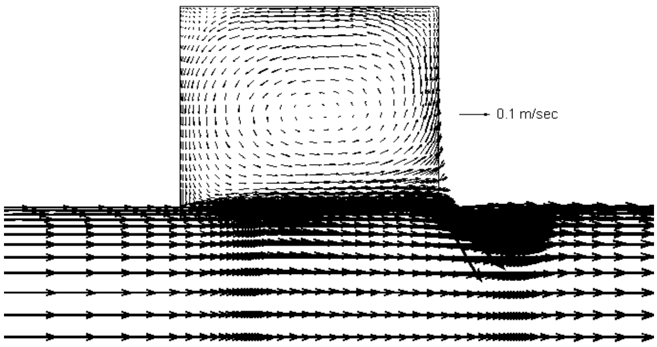

Figure 5 : Time averaged velocity vectors

4.2 Temporal Variation of Flows

The formation of instability vortices at the interface of main channel and dead-zone and its interaction with dead-zone circulation can be described by considering the temporal change of velocity vectors as well as the variations in water depth as shown in Fig. 7. Due to space limitation, only two vector plots and corresponding water depth distributions are shown in the figure for t=54.4 and 54.8 sec. It can be noted that the period of instability vortex for this flow is found about 0.8 sec. The deviation of velocity vectors from the main circulation near the interface shows the position of instability vortices. It is found that the instability vortex observed at upstream corner of interface moves downstream with gradual amplification with time, and the process continues repeatedly with a constant periodic interval. The figure shows that the instability vortex at upstream of dead zone interface for t=54.4 sec is moved downward at t=54.8 sec with a formation of new vortex at upstream corner.

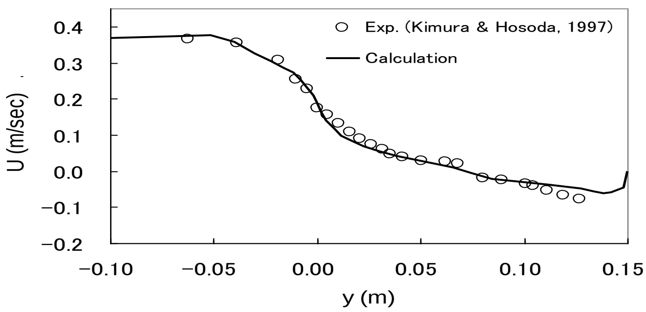

Figure 6 : Comparison of time averaged velocity profile along T-T Section of Fig 2.

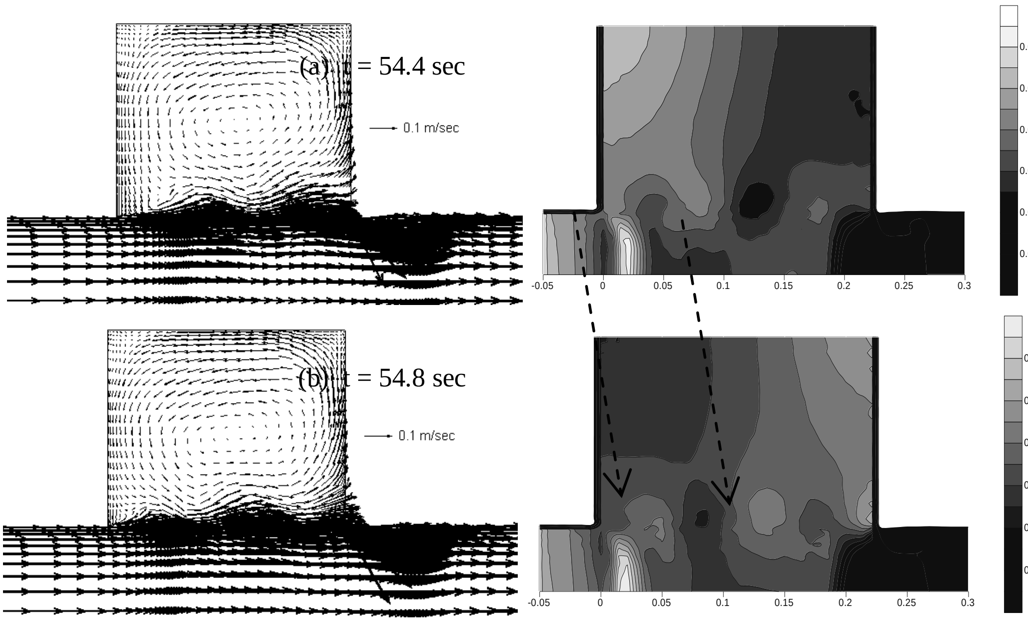

Figure 7 : Instantaneous velocity vectors (left side) and water depth contours (right side) at two different times with 0.4 sec interval.

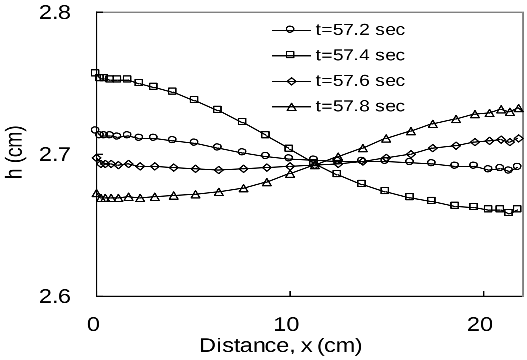

Figure 8 : Oscillation of water surface inside the dead-zone (S-S section).

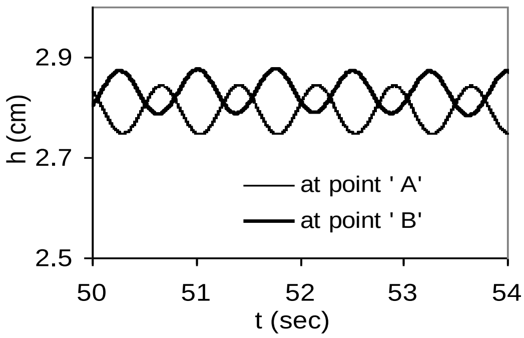

Figure 9 : Temporal variation of water surface at point A and B.

The temporal variations of water surface distribution demonstrate two important phenomena: one is seiche inside the dead-zone, and another is depression of water surface at the center of instability vortices. The depression in the free surface moves downward with time along the interface, as observed in the movement of large vortices in vector plots. In the present flow domain two to three depressions in water surface can be seen in an instantaneous flow field. Since, the vortices at the upstream corner are premature and those near the downstream corner are decayed due to merging with main circulation, the large vortices at the middle part of the interface are most matured and show highest depression in the surface.

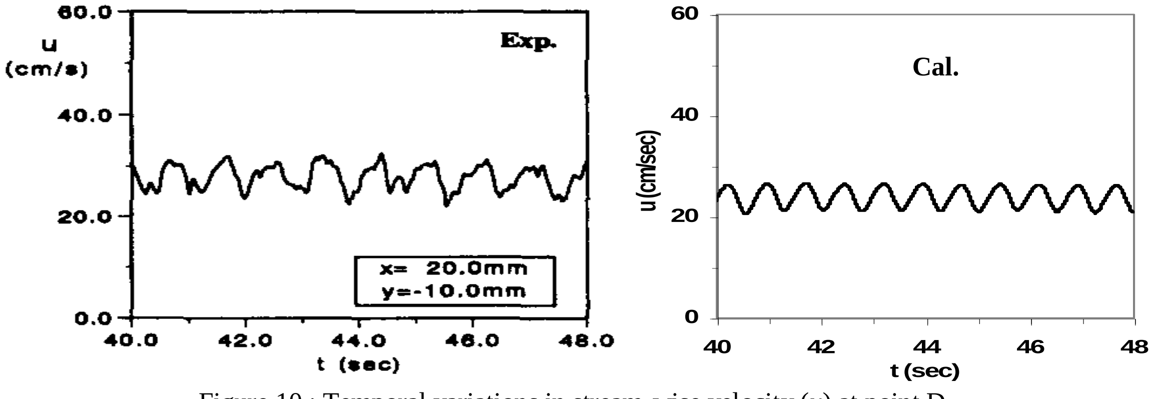

Figure 10 : Temporal variations in stream-wise velocity (*u*) at point D.

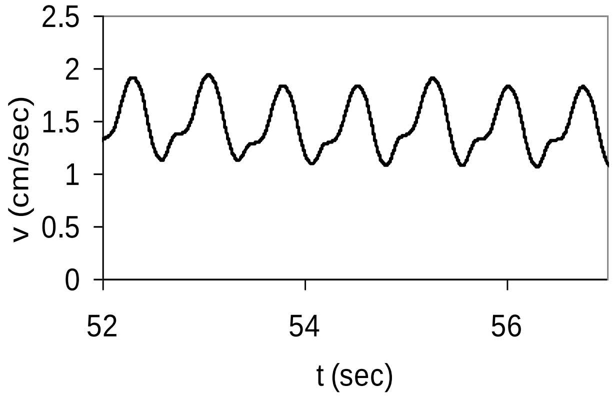

Figure 11: Temporal variation of transverse-velocity component (v) at the interface (x =5 cm).

The second important phenomenon demonstarted by the contours is the oscillation inside the dead zone. If we look the water surface distribution inside the dead zone, Fig. 7(a) shows minimum depth at downstream end and maximum depth in upstream end of dead zone; on the other hand, Fig. 7(b) shows reverse phenomena. A periodic change in the depth difference between upstream and down stream end of dead zone indicates the existence of oscillation in the cavity.

This oscillation of water surface is clearly demonstrated in Figs. 8 and 9. In Fig. 8, the spatial water depth distribution in the dead zone along the section S-S (location of section is shown in Fig. 2) is shown for different time intervals. It explains that the node of oscillation, that contains the lowest seiche mode with a direction parallel to the main flow, is located near the center of the circulation. The wavelength of the oscillation is about twice the dead-zone length. Comparing with Fig. 7, it is observed that the depth variation inside the dead zone is mainly due to the seiche, and near the interface it is governed by the instability vortices.

Figure 10 shows the calculated results for temporal free surface oscillation at point A and B. The period of oscillation is found as 0.75 sec, which is slightly smaller than the experimentally observed value of 0.87 sec. The calculated magnitude of velocity variation is well agreed with experiment. The period of oscillation in the temporal changes of u-velocity is found same with the periodic oscillation of water surface. The temporal change of transverse velocity component (v) at a point on the interface (x =5 cm, y =0) is shown in Fig. 11. It is observed that, each cycles of temporal velocity variations consists two components of oscillations. The long one is due to seiche and has a time period of 0.75 sec. The period of short oscillation component is about 0.15-0.25 sec, which is caused by the instability vortices. This phenomenon is also reported in previous experimental observations.

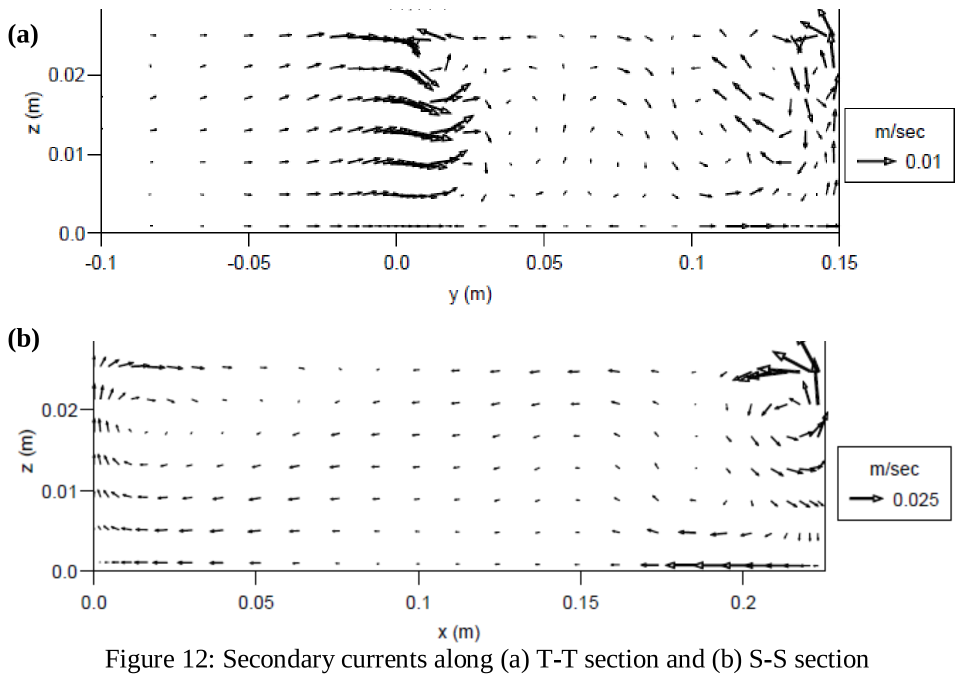

Figure 12: Secondary currents along (a) T-T section and (b) S-S section

4.3 Behaviour of Secondary Currents

Although the depth of the flow is small, strong secondary currents are found to be generated at inside the dead zone as well as near the interface area. Figure 12 shows the pattern of secondary current at T-T and S-S section in the time averaged flow field. The sectional locations are shown in Fig. 2. In the figure, Surface and corner circulation cells near the boundaries of dead-zone are clearly depicted. Although, several circulation cells are observed at the middle of dead-zone in T-T section with the size equal to the depth, the circulation cells near the interface and at the boundary region are found to be much stronger and seem to be a general feature of the dead zone flow. The flow exchange between main-channel and dead zone can be seen in the sections of an instantaneous flow field as observed in the plan view of flow vectors in Fig. 7.

4.4 Effect of Aspect Ratio (L/W) of the Cavity

For case C2, the ratio of the dimensions of the cavity is taken as L/W=3 to identify the change in flow behavior due to change in length. The simulated result of vector flow field shows that, with increasing the length of the cavity the circulation pattern and their numbers are changed along with velocity magnitude. In previous cases, only one circulation was observed, but in case 2 (Fig. 13) there are about three circulations, among them two are the main circulations and one is at left corner which is called corner vortex. The exchange of flow between mainstream and dead zone is very prominent comparing with the previous case. Figure 14 shows the fluctuation of water surface in the simulated flow field. The deep colour shows low depth of channel and the light colour shows higher depth in the contour map. The figure indicates that the water surface is highly fluctuating along the interface. Although the circulation pattern inside the cavity for case C2 is quite different than that of case C1, the fluctuation pattern in water surface is observed to be similar.

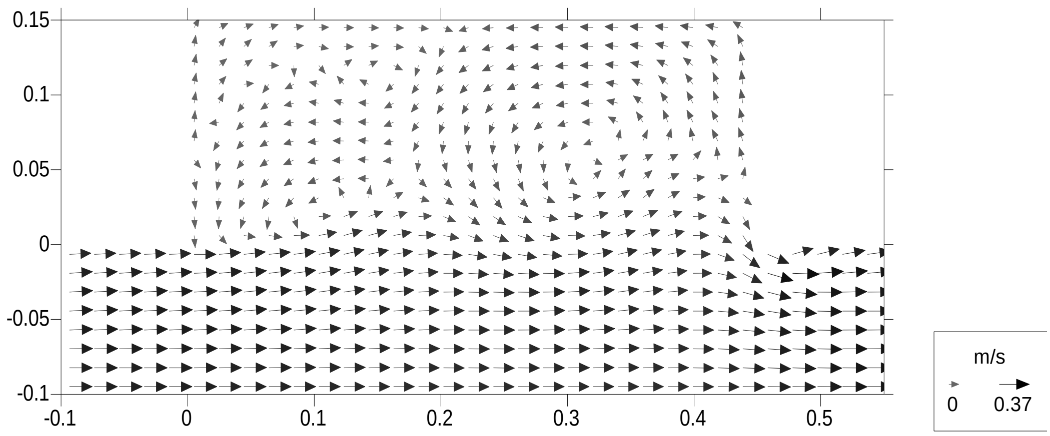

Figure 13: Instantaneous velocity vectors for case C2

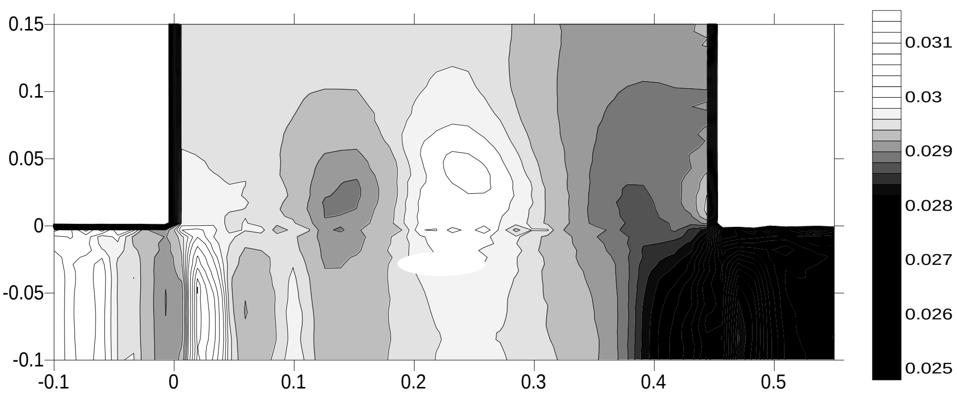

Figure 14: Fluctuation in water surface in the simulated flow field for case C2

5. FLOW CHARACTERISTICS IN A SUDDEN ENLARGED ZONE

The difference between flow domain of a sudden enlarged zone with that of side cavity is only the downstream boundary. In a cavity the downstream is closed and water cannot flow freely but to exchange the momentum in lateral direction with the mainstream flow. On the other hand, for the open channel with sudden enlargment, although the upstream arrangemnet of flow domain is same as the cavity flow, the flow in the enlarged zone has the freedom to flow towards downstream in logitudinal direction. Due to this free downstream, the simulated flow field in the sudden enlarged zone is significantly different than that of open channel flows with side cavity.

The simulated flow field of open channel flow with sudden enlarged zone for case E1 is shown in Fig. 15. Only one main circulation is observed in the dead zone of the enlarged portion of the channel. The main flow is observed to be deflected to wards the enlargement (left side). At the end of the main circulations a small circulation formed at the opposite site of the enlargement (right side) due to the movement of flow towards left side. After that the velocity vector is observed to be approached to the uniform flow. The contour of stream-wise velocity componant is shown in Fig. 16. White colour shows the high velocity region and the blakish indicates the lower velocity region. The deep black area is the negetive velocity region. The path of high velocity stream is clearly observed in the contour. Two circulation regions that contain negetive stream-wise velocity are also easily distinguishable from main stream flow.

Figure 17 shows the fluctuation of water surace in the simulated flow field. The deep colour shows low depth of channel and the light colour shows higher depth in the contour map. The figure indicates that the water surface is highly fluctuating along the interface. The water surface is observed to be depressed in the circulation with lowest depth at the center of the circulation. Since the circulation pattern in dead zone for case C1 and C2 is different than that of case E1, the fluctuation pattern in water surface is also observed to be different.

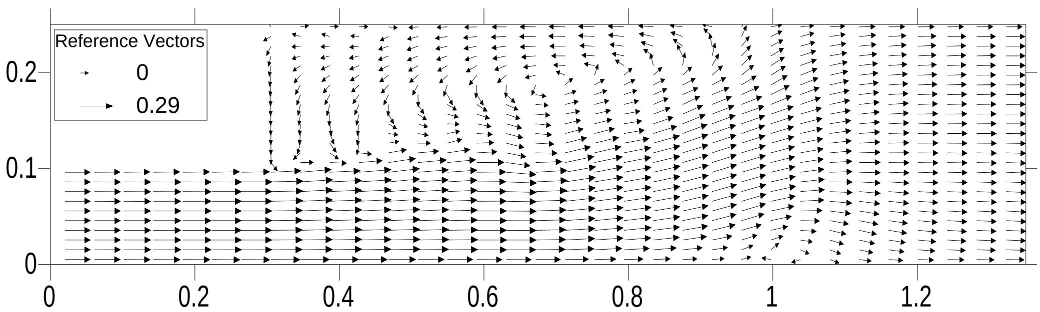

Figure 15: Instantaneous velocity vectors for case E1.

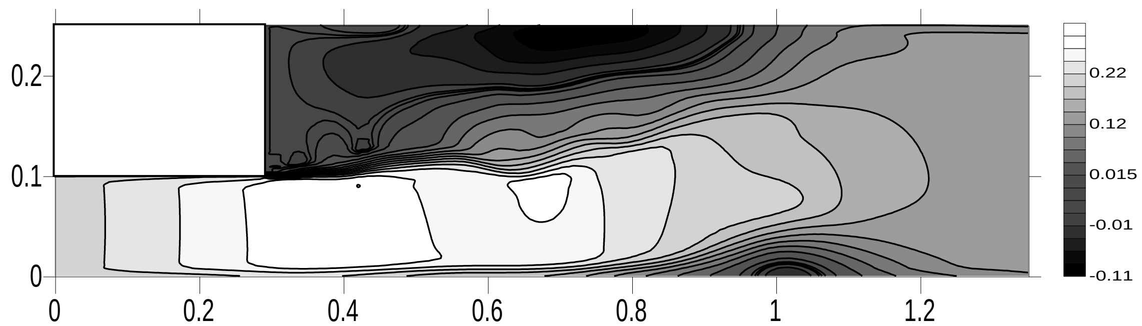

Figure 16: Contour of stream-wise velocity (U-velocity component in m/s) for case E1.

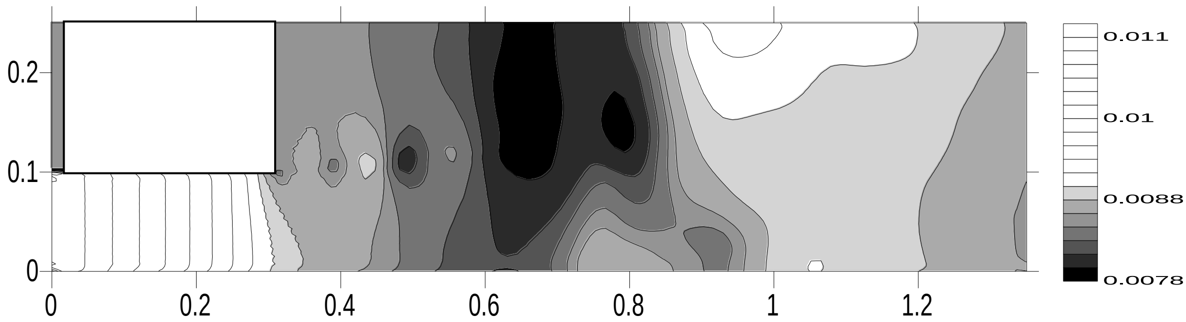

Figure 17: Fluctuation in water surface in the simulated flow field for case E1.

6. CONCLUSIONS

The flow fields in an open channel with two types of dead zone have been investigated by three-dimensional unsteady RANS computations: one is rectangular side cavity and another one is the dead zone created at the downstream of channel with width encroachment (i.e. sudden enlarged zone). The time averaged flow properties and temporal change of velocity profiles are well compared with previous experimental results. Based on the results following conclusions are made.

- It is found that the flow in the side cavity is characterized by three types of flow phenomena: the circulation inside the dead-zone, periodic coherent vortices at the interface of main stream and dead zone, and the water surface oscillation inside the dead zone. In this study all these characteristics are successfully reproduced by numerical simulation.
- The temporal variation of water surface shows a depressed water depth at the center of coherent vortices. This hollow of the free surface moves downward with time along the interface, as observed in the movement of large vortices in vector plots. In the calculated result, a periodic change in depth difference between upstream and downstream end of dead zone indicates the existence of oscillation in the cavity. The period of oscillation is found same as that of instability vortex. Although the depth of the flow is small, strong secondary currents are found to be generated at inside the dead zone as well as near the interface area.
- An exchange of flow between mainstream and dead zone is observed is observed. Due to the formation of circulation in the dead zone, the velocity is very small compared to main stream, and the velocity at the center of circulation is zero. For length to width ratio of the cavity as 1.5 (L/W=1.5), the center of the circulation is found to be situated at the middle of the cavity in both the directions.
- with increasing the length of the cavity, the circulation pattern and their numbers are changed along with velocity magnitude. In previous case, only one circulation was observed, but in the case with higher length to width ratio of the cavity (L/W=3), there are about two main circulations in the cavity, the first one is smaller than the second one due to the suppression by the prominent corner vortex. In this case the exchange of flow between mainstream and dead zone is very prominent compared to previous case.
- For the open channel with sudden enlargment, although the upstream arrangemnet of flow domain is same as the cavity flow, the flow in the enlarged zone has the freedom to flow towards downstream in logitudinal direction. Due to this free downstream, the simulated flow field in the sudden enlarged zone is significantly different than that of open channel flows with side cavity. Only one main circulation is observed in the dead zone of the enlarged portion of the channel. The main flow is observed to be deflected towards the enlargement (left side). At the end of the main circulations a small circulation formed at the opposite site of the enlargement (right side) due to the movement of flow towards left side. The water depth at the center of circulation is low compared to surroundings.

Finaly, it can be concluded that the dead zone is a low velocity region, and due to this low velocity the sediments carried by the stream may deposit there easily to form shallow depth region. The effect of width enchroachment due to earth filling or construction of any obstruction extending inside the river also creates dead zone that finaly may filled up by sediments. Beside the academic research interests, the present study contributes to understand the developement, extent and behaviour of flows in a dead zone for proper managemnet of river system and other water courses.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STUDY ON DRAINAGE CONGESTION IN DHAKA-NARAYANGANJ-DEMRA (DND) PROJECT AREA

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ABSTRACT

DND (Dhaka-Narayanganj-Demra) project is one of the major water resources project in Bangladesh. In 1989 a Flood Action Plan (FAP) was taken and flood protection was given the prime concern in the DND area. It is also taken into account to develop the irrigation facilities and solve drainage problem in DND area. But recent condition in DND area is not satisfactory. Drainage congestion has become the prime concern in DND area. The main purpose of the study is to find out the root causes of drainage congestion problem as well as to find out necessary solutions for the improvement of drainage condition. The study includes critical review of all major components of the project, analysis of existing situation, identify the major causes of drainage congestion and recommendation for the solution. The study calculates the peak discharge of the catchments by using rational method and then the estimation of discharge for different canals have been made by using 20-40 rule. The comparison shows the main cause of drainage congestion as the canal are less than effective in carrying the drainage water. Some recommendations has been provided in this study for the improvement of drainage congestion in DND area.

Keywords: *DND area, drainage congestion, 20-40 rule, canal capacity.*

1. INTRODUCTION

DND (Dhaka-Narayangonj-Demra) project is one of the major water resources project in Bangladesh. Since 1947, the project was a flood-deficit area. In 1987 and 1988 Bangladesh faced a disastrous flood. In this time the capital of Bangladesh Dhaka had faced a flood and drainage congestion problem. After the flood in 1989 with the help of World Bank a Flood Action Plan (FAP) was taken. In this FAP plan an area is selected under this project named DND area (Dhaka-Naraongonj-Demra). Flood protection was given the prime concern in the DND area. It is also taken into account to develop the irrigation facilities and solve drainage problem in DND area. The Project is financed by ADB, World Bank & Govt. of Bangladesh and technical support was provided by BWDB, JICA, DWASA, RAJUK, and Roads & Highway (Jahan & Ahmed 2002). The project was undertaken having its goal to the development of agricultural sector. But now days a rapid urbanization has found in the area, it causes some environmental, social issues as well as drainage congestion problem. Many parts of project area are free from flood after construction of embankment but there is still local drainage congestion problem. So further studies are necessary in order to find out the problems and its causes and try to find the solutions. Some recommendations also needed for proper management of the project. With the objectives of removing drainage congestions and to upgrade the drainage system of Dhaka City and DND Project area, a study was undertaken in 2006 by Institute of Water Modeling and Dhaka Water Supply and Sewerage Authority (IWM 2006). The integrated flood control plan in DND area has been prepared by Bangladesh Water Development Board (BWDB 2008). The main purpose of the study is to find out the drainage congestion problem and give necessary solutions for the improvement of conditions. To carry out this study, necessary data has been collected from the related organizations and field visit has made at the project area to observe the present condition. Theoretical analysis has made by using rational method and after that 20-40 rule and compare it with the present canal conditions to find the problem. At last some recommendations are provided for improvement of the drainage congestion problem.

2. PROJECT DESCRIPTION

The DND project area lies between latitudes 23°37'25"N to 23°43'10"N and longitudes 90°26'05"E to 90°31'30"E. The Sitalakhya River lies on the east and the Buriganga River on the west of the study area. The study area, which covers 59.16 km2, is located in Demra and Shampur Thana of Dhaka District and Fatullah, Siddirganj and Sadar Thana of Narayanganj District (Figure 1). The area covers Dhaka City Corporation Ward No. 32 and 33 and Shyampur Union of Shyampur Thana, Matuail Union of Demra Thana, Siddirganj, Shimulpara and Godnail Unions of Siddirganj Thana, Kutubpur and Fatullah Unions of Futulla Thana and Municipal Ward No. 1, 2, 3, 4 of

Narayangonj Municipality and Enayetpur Union of Narayanganj Sadar Thana. The climate of the study area is classified as tropical monsoon type, characterized by three seasons: pre-monsoon, monsoon, post-monsoon/dry season. Average temperature varies from about 20°C in December and January to about 30°C in March and April. The average daily evaporation varies from 80 to 130mm. It is the lowest in November and the highest in August. The surface water hydrology around the study area is complex. The rivers, around and influencing the study area, are the Dhaleswari-Buriganga-Sitalakhya-Balu river system. The Buriganga is a tributary to the Dhaleswari River, a major tributary to the Meghna. The Lakhya River joins the Dhaleswari at 11 km downstream of the Buriganga confluence. About 5 km below the Dhaleswari-Lakhya confluence, the Dhaleswari meets the Meghna River. The Dhaleswari-Buriganga-Sitalakhya-Balu River system is tidal and the effect is significant during the dry season when upstream inflows are minimal. Saline intrusion however, does not take place in the area. The area is criss-crossed by drainage khals and old irrigation canals. Also, borrow pits of the internal roads are working as part of drainage network. The system is presently working as one basin and discharging by the existing pump station at Shimrail into the Sitalakhya River. There are two main drainage khals which meet at Ghoshpara near Shimrail and the flow leads to the pump station. These two main khals are fed by six secondary and other minor khals.

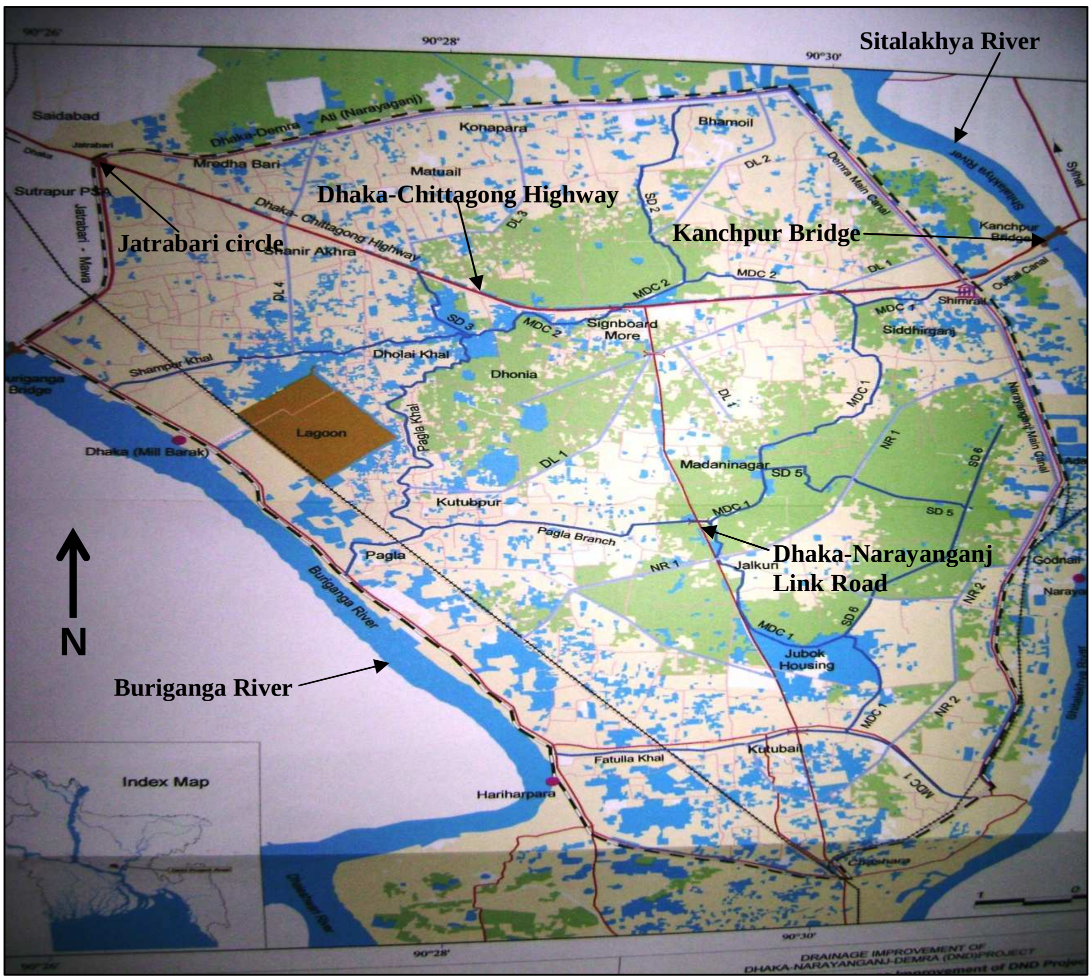

Figure 1: Plan area map of DND project area

The DND Project area has been developed rapidly during past two decades because of its proximity to Dhaka, the capital city and Narayangaj the large commercial and industrial town and being a flood free area from external flooding. It has progressively changed its land use pattern from agricultural development area to an urban development area. The present scenerio gives about 25% of agricultural land against 56% of the same in JICA (1991, 1992) Study. This reduction in agricultural land is mainly due to increase in urban development area. The area is compartmentalized into four zones by the Dhaka-Chittagong Highway and Dhaka-Narayanganj link Road (Figure 1). Zone-1 (13.44 km^2) is the northern part of area in the north side of the Dhaka-Chittagong Highway, Zone-2 (23.17 km^2) is the western part of the area in the west side of the Dhaka-Narayanganj Link Road, Zone-3 (5.06 km^2) is the southern part of the DND area in the south side of Fatulla Khal and Zone-4 (17.49 km^2) is the eastern part of the study area in the east side of the Dhaka-Narayanganj Link Road. Zone-wise land use pattern is shown in Table 1.

Table 1: Zone-wise Land use in DND area (CEGIS 2008)

Land use	Area (Sq. km)				
	Zone 1	Zone 2	Zone 3	Zone 4	Total
Agriculture land	2.80	2.75	0.12	7.93	13.60
Settlement	8.57	11.01	3.14	6.87	29.76
Road	0.53	0.67	0.21	0.47	1.88
Canal	0.46	0.20	0.02	0.43	1.12
Industry	0.13	1.54	0.35	0.10	2.13
Sewerage Treatment Plant	-	0.95	-	-	0.95
Water bodies	0.17	4.15	1.34	1.72	8.38
Others	0.28	0.65	0.16	0.25	1.34

3. SURVEY AND DATA COLLECTION

In this study, some reconnaissance surveys have been carried out throughout the project area by several field visits. The objectives were to get the existing scenario of the drainage congestion problem and to identify the root causes behind it. The causes as identified are summarized below:

- The area is near to Dhaka- the capital city and Narayangaj - the large commercial and industrial town and being a flood free area from external flooding encouraged construction of dwelling house, industries and workshop on low lands in unplanned manner.
- Lack of proper planning for the use of project land created scope for unplanned urbanization of the project area.
- Placing of artificial barriers in low-lying areas and cross dams on the mouth of drainage khals for pisciculture.
- Dumping of sewage and solid wastes into the drainage canals from newly established industries, houses and factories.
- Construction of internal road networks in unplanned manner without adequate structures at crossings over drainage khals.
- Reduction in pumping capacity of the existing pump station to drain out additional rain water along with waste water from industries and factories.
- No provision of retention ponds for storage of excess runoff in the monsoon.
- Lack of proper coordination between various government agencies active in the area and the district administration.
- Non-operation of pumps due to power failure and unscheduled repair and maintenance.
- Absence of proper sewerage and wastewater management for the area.
- Poor maintenance of drainage system.

Other than field visit and reconnaisance survey, required data has been collected from BWDB to conduct this study. These data include area map, rainfall data, watershed data, drainage network map, topographic data, discharge and water level data, data on existing canals, regulator and embankment, land zoning map, data on existing pump station and capacity.

The area is divided into nineteen sub-catchments and they are numbered from C-1 to C-19 (CEGIS 2008). There are two main drainage canals (MDC-1, MDC-2) and nine secondary drainage canals (Pagla khal, Pagla branch khal, Fatulla khal and SD-1 to SD-6). Figure 2 shows the catchment area and drainage network map. The catchment details are gven in Table 2.

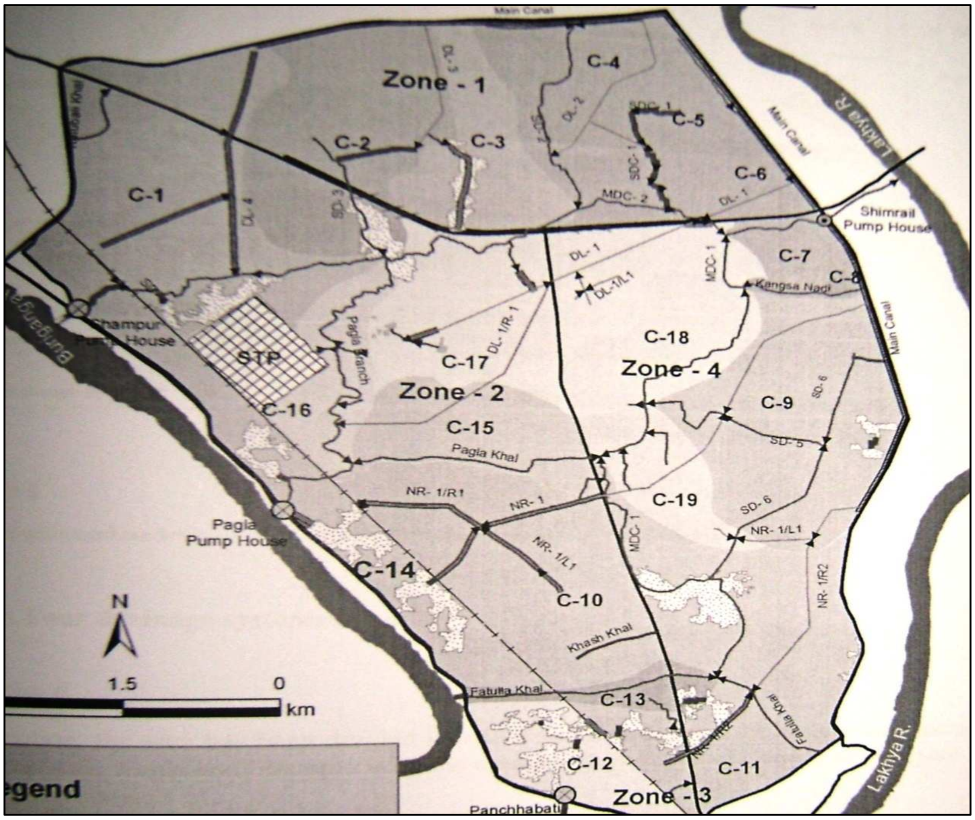

Figure 2: Catchmnet area and drainage network map in DND area (CEGIS 2008)

Table 2: Drainage sub catchments in DND area (CEGIS 2008)

Drainage sub-catchments	Area (Km^2)	Conveyance Khal	Lowest Land Level(m PWD)	Highest Land Level (m PWD)
C-1	7.06	Shampur Khal	1.85	9.11
C-2	4.56	Secondary drainage -	1.71	8.85
C-3	2.56	Main drainage-2	1.61	6.01
C-4	2.80	Secondary drainage-	1.76	6.55
C-5	1.92	Secondary drainage-	2.14	6.10
C-6	1.80	Main drainage-2	1.72	7.60
C-7	0.86	Main drainage-1	1.76	6.84
C-8	0.30	Kangsho Nadi	3.31	6.53
C-9	5.87	Secondary drainage-	2.30	6.92
C-10	8.30	Main drainage-1	1.62	6.38
C-11	3.83	Fatulla khal	0.18	7.12
C-12	1.97	Fatulla khal	0.23	6.88
C-13	0.80	Fatulla khal	2.89	6.76
C-14	2.75	Pagla khal	2.64	9.66
C-15	2.34	Pagla khal	1.78	5.52
C-16	1.56	Pagla branch	1.92	4.58
C-17	5.40	Main drainage-2	1.75	6.49
C-18	2.29	Main drainage-1	1.96	6.62
C-19	1.66	Main drainage-1	1.49	5.52

4. ANALYSIS AND RESULTS

4.1 Estimation of Runoff

In order to estimate the required discharge of a drainage canal it is needed to find out the watershed area related to drainage canal. A drainage canal may carry water from different catchment area. So the first step is to find out the catchment area from which water comes to drainage canal. Here rational method is used to calculate peak discharge. The rational method is suggested for planning and design purpose for drainage basin that are uncomplicated and not larger than 50 km^2. By this method discharge is estimated as (Subramanyam 1994):

$$Q = 0.278CIA \quad \ldots\ldots (1)$$

Where, Q=Maximum rate of runoff or peak discharge in cumec, C=Run-off coefficient, A= Area of catchment in km^2 and I is rainfall intensity (mm/hr).

4.2 Rainfall Intensity for Dhaka Station

The rainfall intensity corresponding to a duration (time of concentration) and the desired probability of exceedence P (return period T=1/P) is found from the rainfall frequency duration for the given catchment area. This study includes fitting of Gumbel extreme value distribution and analysis involving daily rainfall, number of thunder storm days and rainy days and short duration rainfall. According to WMO a thunder storm day is a local calendar day on which thunder is hard. A thunder storm does not have to have precipitation, nor thus one have to see lighting, just hear the thunder. Data collected of annual rainfall, annual maximum daily rainfall, annual number of rainy days, annual number of thunder storm days for Dhaka rain gauge station are presented in Table 3.

Table 3: Annual Rainfall, Annual Maximum daily rainfall, Annual number of rainy days, annual number of thunder storm days for Dhaka rain gauge station (Ishtiaq 2008).

Year	Annual Rainfall (mm)	Annual Maximum Daily Rainfall (mm)	Annual Number of Rainy Days	Annual Number of Thunder Storm Days
1985	2053	92	128	61
1986	2500	176	126	57
1987	2187	138	112	54
1988	2482	135	137	60
1989	1627	118	115	43
1990	2130	94	129	59
1991	2850	123	131	69
1992	1169	90	96	29
1993	2819	140	142	76
1994	1540	74	103	46
1995	1751	83	91	45
1996	2044	150	116	55
1997	1896	121	112	56
1998	2310	122	130	56
1999	2374	141	115	54
2000	2124	158	116	47
2001	1679	71	126	47
2002	1875	88	124	53
2003	1693	93	121	51
2004	2347	341	112	58

4.3 Rainfall Intensity at DND area

In this study 5 years return period is selected for taking the value of rainfall intensity as it is a more common phenomena and economic in design purpose. The obtained value for 5 years return period for Dhaka station is 77.57 mm/hr. Rainfall intensity at Shimrali station is very close to rainfall intensity at Dhaka. Prior to the double mass analysis, correlation analysis has been carried out to find out the relative dependency of available records among the target and index stations. Target stations are the individual stations whose statistics has to be compared, whereas index station are those which are located within 30 km radius from the target stations and have significant correlation with the available data of target station. Using the correlation method a relation is established between Dhaka station and Shimrail station which value is 0.91. By this correlation factor, the rainfall intensity at Shimrail is found 85.31 mm/hr at 5 years return period, which is used as the rainfall intensity at DND area.

4.4 Selection of Runoff Coefficient

First at the time of JICA study the main plan of development of DND area as agricultural land. Now a new plan established for removing water logging and in this new plan the land development is based on residential zone. For urban area in residential zone runoff coefficient varies between 0.30 to 0.75. Here for DND area the value of runoff coefficient is taken as 0.60.

4.5 Estimation of Canal Discharge

The DND area has 19 catchments which is shown in the Figure 2. The drainage runoff of each catchment has been calculated by rational method, which are presented in Table 4.

Table 4: Catchment name, calculated discharges and conveyance canals

Catchment name	Calculated discharge (cumec)	Conveyance canal
C-1	100.38	Secondary drainage-4
C-2	64.84	Secondary drainage-3
C-3	36.39	Main drainage-2
C-4	39.81	Secondary drainage-2
C-5	27.299	Secondary drainage-1
C-6	25.59	Main drainage-2
C-7	12.23	Main drainage -1
C-8	4.27	Secondary drainage-6
C-9	83.46	Secondary drainage-5
C-10	118.01	Main drainage-1
C-11	54.46	Fatulla khal
C-12	28.01	Fatulla khal
C-13	11.37	Fatulla khal
C-14	39.10	Pagla khal
C-15	33.27	Pagla khal
C-16	22.18	Pagla branch
C-17	76.78	Main drainage-2
C-18	32.56	Main drainage-1
C-19	23.60	Main drainage-1

4.6 Estimation of Drainage Discharge

Drainage discharge is calculated from the obtained peak flow in different catchments. Every catchment has conveyance canals, so water comes from the catchment area to a canal. For more than one catchment from which water comes to a canal 20-40 rules are used for calculating discharge.

20-40 Rule

Runoff is determined above and below the outlet of contributing ditches and streams, at points of change in the channel slope, at culverts and bridges, and at the outlet. Runoff calculations generally begin at the upper end of the drain and proceed downstream. An empirical procedure, termed the 20-40 rules, should be used in computing the required capacity for a drain below a junction with a lateral. For large drainage areas, the application of the procedure may have considerable effect on the drain design. In small areas the change in required drain capacity may be so small that the procedure need not be applied. Experience in applying the 20-40 rules will guide the designer in its use. The rules for computing the required capacity for a drain are (Water Management Guide 2007):
Rule 1—Where the watershed area of one of the ditches is 40 to 50 percent of the total watershed area, the required capacity of the channel below the junction is determined by adding the required design capacity of each drain above the junction. This is based upon the assumption that the flows from two watersheds of about the same size may reach the junction at about the same time, and that therefore the drain capacity below the junction should be the sum of the two flows. This rule should be used in all cases for watershed areas of less than 300 acres.
Rule 2—Where the watershed area of a lateral is less than 20 percent of the total watershed area, the design capacity of the drain below the junction is determined from the drainage curve for the total watershed area.
Rule 3—Where the watershed area of a lateral is from 20 to 40 percent of the total watershed area, the discharge is proportioned from the smaller discharge at 20 percent to the larger discharge at 40 percent. In this range the discharges should be computed by both methods and the difference in cubic feet per second obtained. The design discharge for the channel below the junction should then be obtained by interpolation. See the following example.

By using the value of rainfall intensity, run-off coefficient and the above mentioned rule, the drainage discharge for different canals are estimated and is presented in Table .

Table 5: Estimated discharge for different canals

Canal Name	Estimated discharge (cumec)
Main drainage canal 1	121.49
Main drainage canal 2	100.1
Pagla khal	72.37
Pagla branch	22.18
Secondary drainage 1	27.99
Secondary drainage 2	39.82
Secondary Drainage 3	64.84
Secondary Drainage 4	100.38
Secondary Drainage 5	83.46
Secondary Drainage 6	4.27
Fatulla khal	79.94

4.7 Drainage Canal Conditions

This study collects the data for existing drainage capacity of main drainage canal 1 and 2, which are compared with estimated values and presented in Table 6.

Table 6: Existing and estimated drainage capacity of the main drainage canals of DND area

Station No.	Station Name	Khal Name	Existing drainage capacity (cumec)	Estimated drainage (cumec)
1	Painadi	Main drainage canal -1	15.53	121.49
2	Shanarper	Main drainage canal -2	7.67	100.1

The comparison shows the existing canal capacity is very less for safe disposal of excess water from catchment area which causes drainage congestion problem. Root causes of present adverse conditions of canal includes dumping of waste in the canal, artificial barrier in the canal, encroachment of canals, inadequate design, poor maintenance etc.

5. CONCLUSIONS AND RECOMMENDATIONS

DND (Dhaka-Narayanganj-Demra) project was developed as an irrigation project by construction flood embankment after 1988 flood. As the project area has become safe from flood, people gradually started to move there and have constructed houses there. Most part of this area has become residential area without proper planning, drainage system and residential facilities during last two decades. Drainage congestion is a major problem in this area. This study investigates the root causes of the drainage congestion in this area. The study also include calculation of peak discharge of the catchments by using rational method and then the estimation of discharge for different canals by 20-40 rule. A comparison of discharge of canals with estimated discharge has made at two points and found the canals are very less effective in their main purpose of conveying of drainage water. The estimated discharge for safe disposal of water from surrounding catchment areas by main drainage canal-1 is found 121.49 cumec but the present canal capacity is found very less (15.53 cumec) than the estimated value. For main drainage canal-2, the estimated value is 100.1 cumec, which is much larger than the present capacity of the canal for disposal of excess water (7.67 cumec). It has found from field survey that artificial barrier, unplanned land zoning, lack of EIA and SIA are the main causes of drainage congestion. Inadequate main pump capacity is also one of the main causes of drainage congestion. Based on the critical review of past and present studies, analysis of data , field survey and public opinion some recommendations has provided for the improvement of drainage congestion in DND area. Recommendations include a detail planning of the area including the land zoning, redesign of canal section, re-excavation of the canals, making waste dumping station, improving the puming capacity of the drainage pumps and proper maintenance. This study may help the concerned authority to improve the drainage congection in DND area.

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SUSTAINABILITY OF PERIPHERAL RIVERS OF DHAKA AS SOURCES OF WATER SUPPLY

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ABSTRACT

Dhaka is one of the most rapidly growing megacities in South Asia. Supplying water for its huge population is a great challenge for the government. Groundwater contributes 83% of the total supply while contribution of surface water is 17 %. Because of indiscriminate extraction of groundwater water table of Dhaka city is experiencing a depletion of 2 to 3 meters per year offering a potential threat of land subsidence. So it is the demand of the time to evaluate the suitability of surrounding surface water bodies as sources of water supply. Dhaka city is surrounded by a circular river system. The river system includes the rivers Buriganga, Turag, Shitalkhya, Balu and Dhaleswari. Sufficient quantity of water remains in the rivers during the five months of monsoon season but the flows are practically nil during dry period except tidal backflow from the Meghna river. The water quality of the river system is deteriorating tremendously. It is observed that required dissolved oxygen for sustaining aquatic lives (5 mg/l) prevails only in the Dhaleswari and very short downstream reach of Shitalakhya throughout the year. Ammonia level in the different reaches is well above the permitted value specified in the USEPA to avoid toxic effect. Concentrations of Nitrate, Phosphate, Zinc, Chromium, and Lead are well below the allowable limits in different Environmental Quality Standards (EQS). The assessment of this study identifies that water of some least polluted river reaches like the Upper and Lower reaches of the ShitaLakhya river (Demra to Rupganj and Siddhirganj to Kalagachia) and the Dhaleswari river (Nabinagar to Kalagachia) could be used in city water supply providing required treatment.

Keywords: *Water quality, Dissolved oxygen, Ammonia level, water treatment*

1. INTRODUCTION

The present water demand in Dhaka city is 2370 million liter per day. Dhaka Water Supply and Sewage Authority (DWASA) tries its best to afford sufficient water for meeting up demand of the city. By the year 2025 its population will be 21 million. At present supply deficit is 40% of the total demand of Dhaka & the deficit will be 60% by 2025. Depletion of ground water table is very alarming. It is very crucial to diversify the sources of water supplied for the city. Diminution of dependency on the groundwater and to make provisions to meet demand of the city from surface water is now very essential. Wastewater being discharged from major drains/khals contains domestic as well as industrial wastes and is ultimately disposed into the peripheral rivers. Since the peripheral river system gets no upstream inflows during dry season, the high waste loads deteriorate the water quality tremendously. A thorough assessment of water quality in the peripheral rivers is essential for their sustainability and also to rescue the city water supply system from every possible upcoming adversity. The objective of the study is to assess the sustainability of peripheral river system of Dhaka as potential surface water sources for Dhaka city water supply to cope up the increasing water demand of the city.

2. METHODOLOGY

The assessment of water supply scenario comprises evaluation of peripheral river system around Dhaka city including its extend, topography and hydraulic condition; groundwater resource, depletion of groundwater table underlying the city; present water supply practice and treatment facilities; the levels of content of different elements, compounds and materials co-existing with water. It also necessitates dealing with characteristics of water body (extend, source, storage volume and hydraulics), characteristics of pollutants (source, volume and type), climatic condition for supplementing the evaluation.

2.1 River Network

Five river reaches encompasses the Dhaka city which include Buriganga, Shitalakhya, Dhaleswari, Turag and Balu river. There are some drainage channels inside the city like Dholai khal, Begunbari khal, Norai khal and Kallayanpur khal which are linked with the peripheral rivers. Few of them are controlled through several structures.

2.2 River Topography and Hydraulics

The river reaches have general slopes from north to south. The river system experience flood flows during monsoon from upstream inflows of the Turag, Dhaleswari and Shitalakhya river. The river system gets no inflows during dry season. The rivers experience semi tidal influence in downstream reaches during flood seasons. Whereas tidal back flows from the Upper Meghna river penetrate far upstream during dry periods.

2.3 Groundwater Depletion

The surface and subsurface geologic formation under Dhaka city are not favorable for replenishment and storage of groundwater against withdrawal. The records of groundwater level observed by Bangladesh Water Development Board (BWDB) indicate that the rate of annual declinations of groundwater level varies from 2 to 3 meters at different locations.

2.4 Water Supply in Dhaka City

Dhaka is a one of the most rapidly growing megacities in South Asia. Supplying water for its huge population is a great challenge for the government. Historically piped water supply in the Dhaka city started in 1876.The population of Dhaka city is around 12.27 million and the water demand is about 2485 MLD (million liters per day) considering per capita consumption 160 liter/person/day. DWASA is capable of supplying only 1500 MLD. Groundwater contributes 83% (441 deep tube wells) of the total supply while contribution of surface water (Sayedabad and Chadinaghat treatment plant) is 17 %. As a result during the last few years the groundwater level has been declining drastically which offers a potential threat of land subsidence and earthquake. By the year 2025 population of the city will be 21 million. At present supply deficit is 40% of the total demand of Dhaka & the deficit will be 60% by 2025. (Source: DWASA).

2.5 Data Collection

Measurement of water quality and storage volume is very expensive, requires skilled manpower, sophisticated instruments and well organized laboratory facilities. As a government organization, Department of Environment (DoE) measures some water quality parameters at some selected locations in the peripheral rivers around Dhaka city. DoE has which include DO, BOD5¬ (at 20°C), COD, turbidity, pH, Electrical conductivity, alkalinity, total dissolved solid (TDS), total suspended solid (TSS), total coliform (TC) and fecal coliform (FC). Besides some water quality parameters had been measured for "Forth Dhaka Water Supply Project" which include DO, BOD5 (at 20°C), COD (dichromate value), NH3-N, NH4-N, NO3-N, PO4, chromium (Cr), lead (Pb), mercury (Hg), zinc (Zn) and coliform (TC & FC).

2.6 Stations of Data Collection

Department of Environment has established some water quality monitoring stations at different reaches of the rivers system. Stations for collecting samples of Buriganga are Hazaribag, Kamrangir char, Chadinaghat, Forasgonj, Sadarghat, Dholai khal, Bangladesh China Bridge and Pagla. Stations of Shitalakhya are Demraghat, Narayanganj and Siddirganj. Stations of Dhaleswari are at Nabinagar, Reckabibazar and Kalagachia.Stations of Balu river are at Keodala and Balurpar while those for Turag are at Mirpur Bridge and Bashila.

2.7 Method of Assessing Suitability of Sources

The assessment of suitability of peripheral rivers as potential sources of water supply for Dhaka city has been based on existing water quality and storage volume of the different reaches of the circular water system. Water quality is evaluated on the basis of water quality standards according to Bangladesh Standard.

3. ILLUSTRATIONS

3.1 Stream Water Quality

Stream water quality has been assessed keeping attention on the drinking standard as well as sustainability of aquatic lives. Water quality observed in different measurements in the river system is illustrated in the following articles.

3.1.1 Dissolved Oxygen

Dissolved oxygen concentration remains low in the dry seasons (i.e. January to April) and it maintains the level more than 5 mg/l (required for sustaining aquatic habitats) from May to December in the entire peripheral river system. Minimum dissolved oxygen required for drinking is 4 mg/l. Minimum dissolved oxygen in the river system is summarized in Table1.

Table 1: Minimum Dissolved Oxygen

River	**Minimum DO (mg/L)**
Turag river	Below 1.0
Buriganga river (Hazaribag to Keraniganj)	Nearly 0.0
Buriganga river (Keraniganj to Fatulla)	Below 1.0
Dhaleswari river (Fatulla to Nabinagar)	Around 2.0
Dhaleswari river (Kashipur to Kalagachia	Around 5.0
Shitalakhya river (Kalagachia to Mosinabanda)	Above 4.0
Shitalakhya river (Siddirganj to Rupganj)	Below 1.0
Balu river	Below 1.0

3.1.2 Biological Oxygen Demand (BOD)

High BOD generally occurs at Kamrangirchar in the Buriganga river (50 to 60 mg/l). BOD hardly crosses 10 mg/l in the Dhaleswari and Shitalakhya river. In Balu river it varies from 5 mg/l to 70 mg/l while that for Turag river varies from 5 mg/l to 40 mg/l. Maximum allowable BOD5 is 3 mg/l. Maximum BOD5 (at 20°C) in the river system observed in the dry season is summarized in Table 2.

Table 2: Maximum BOD_5 at 20°C

River	**Maximum BOD_5 (mg/L)**
Turag river	Around 30
Buriganga river (Hazaribag to Keraniganj)	35 to 55
Buriganga river (Keraniganj to Fatulla)	Around 12
Dhaleswari river	Around 12
Shitalakhya river (Kalagachia to Mosinabanda)	Around 10
Shitalakhya river (Siddirganj to Rupganj)	Around 07
Balu river	Around 08

3.1.3 Chemical Oxygen Demand (COD)

COD in the Turag river and upper reaches of BuriGanga river remains high varying from 50 mg/l to 60 mg/l except Hazaribag having COD of around 90 mg/l. COD is drastically reduced to below 20 mg/l from Pagla in Buriganga to entire Dhaleswari river. Along the Shitalakhya River COD varies from 20 to 45 mg /l. Maximum COD observed in the dry season is stated in Table 3.

Table 3: Maximum COD (Dichromate value)

River	Maximum COD (mg/L)
Turag river	50 to 60
Buriganga river (Hazaribag to Keraniganj)	60 to 90
Buriganga river (Keraniganj to Fatulla)	Around 20
Dhaleswari river	Below 20
Shitalakhya river (Kalagachia to Mosinabanda)	30 to 40
Shitalakhya river (Siddirganj to Rupganj)	Below 20
Balu river	40 to 60

3.1.4 Nitrogen

Nitrogen content at three states i.e., NH_3-N, NH_4^+-N and NO_3-N have been evaluated in the river system. High ammonia nitrogen concentration (around 10 mg/l) is observed at Kamrangirchar and Keraniganj on Buriganga river. Same is case with the Balu River. In the Shitalakhya River its concentration ranges from 3 to 8 mg/l. Maximum NH_3-N concentration in the river system is above the USEPA guideline (0.02 mg/l) to avoid toxic effect on fishes. Ammonia content is low in the Dhaleswari River. Ammonium concentration reaches nearly 20 mg/l in the Turag river and Buriganga river. Dhaleswari and Shitalakhya River experience ammonium concentration of 2 mg/l or less except at Sarulia. The concentration ranges between 4 to 12 mg/l in Balu River. Nitrate content is below 10 mg/l in the entire river system except at Hazaribag. Nitrogen concentrations during dry season are stated in Table 4.

Table 4: Nitrogen concentrations

River	NH_3-N (mg/L)	NH_4^+-N (mg/L)	NO_3-N (mg/L)
Turag river	1	20	7
Buriganga river (Hazaribag to Keraniganj)	12	18	17
Buriganga river (Keraniganj to Fatulla)	9	16	5
Dhaleswari river	0	3	3
Shitalakhya river (Kalagachia to Mosinabanda)	0	4	5
Shitalakhya river (Siddirganj to Rupganj)	8	7	5
Balu river	11	12	4

3.1.5 Total Coliform

Presence of coliforms in surface water bodies in Dhaka is very common. River system of Dhaka contains coliforms of 100 to 9000 CFU/100 ml while the allowable limit is 10 CFU/100 ml. But Coliforms are removable by surface water treatment plants.

3.1.6 Other Water Quality Parameters

For the evaluation of water quality scenario other water quality parameters like pH, TDS, TSS, turbidity, total coliform are also significant. But their extents in the peripheral river system are acceptable for water supply purpose after proper treatment. Values of different water quality parameters are stated along with their Bangladesh Standard values in Table 5.

Table 5: Ranges and standard of water quality parameters

Quality parameter	Ranges in river system	Bangladesh standard
pH	6.1-7.8	6.5-8.5
TSS (mg/l)	11-176	250
TDS (mg/l)	50-440	1000
Quality parameter	Ranges in river system	Bangladesh standard
Turbidity (NTU)	2.6-8	5

3.2 River Hydrometrics and Storage Volume

The river system surrounding Dhaka city comprises a length of about 11100 km. The water level in the river system varies from 1 m PWD to 1.50 m PWD having an average level 1.35 m PWD during dry season. The river system gets virtually no fresh water inflows from upstream except some wastewaters in the dry season. Average channel widths at level 1.5 m PWD are 150m, 300m, 700m, 55m and 260m of Turag River, Buriganga River, Dhaleswari river, Balu river and Shitalakhya river respectively. Storage volume and open channel area of the river system at 1.25 m PWD level (an average water level in dry season) are 75 million m^3 and 32 million m^2 respectively.

Table 5: Interrelationship of elevation, surface area & storage volume in river system

Elevation (m PWD)	**Open surface area (Mm^2)**	**Storage Volume (Mm^3)**
0	18	75
0.5	20	90
1	22	95
1.5	25	110
2	30	120
2.5	35	130
3	45	150
3.5	55	160
4.0	62	180
4.5	68	215
5.0	72	235

3.3 Estimation of Pollution Load

Pollution loads contributed by the point sources, have been calculated in wet method. Wastewater discharge period has been assumed to be 12 hours out of 24 hours considering tidal influence of the receiving rivers as well as water consumption practice. The expression used in computation of pollution load in Wet method is as follows:

*Pollution load = Concentration (mg/l) * Flow (m^3/s) * Flow time*
= *Concentration (gm/m^3) / (1000 gm/ kg)* * Q (m^3/s) * 3600 s/h * 12.0 h/d
= *Load (kg/d)*

But wastewater flow and quality do not maintain uniformity throughout the day. The estimated pollution load is based on a single measurement on a day. Therefore it is very crude estimation and calculated value may not represent the daily pollution load discharged into the river system. However we can get a tentative idea about the volume of pollution load. The estimated pollution loads into the peripheral river system around Dhaka and Narayanganj is summarized in Table 6.

Table 6: Pollution load in river system

Type of Load	**Volume of Pollution load (ton/day)**
BOD_5	230
Ammonia (NH_3)	5
Phosphate (PO_4^-)	8
Chromium (Cr)	0.50
Lead (Pb)	0.10

4. CONCLUSIONS

The study has focused on water quality and storage volume in peripheral rivers around Dhaka city. Sufficient quantity of water remains in the river system in the five months of monsoon season but the flows are practically nil during dry period but the storage volume of Shitalakhya and Dhaleswari are significant for water supply. Water quality is evaluated with respect to dissolved oxygen (DO), biochemical oxygen demand (BOD), chemical oxygen demand (COD), ammonia (NH3), ammonium (NH4+), nitrate (NO3-), phosphate (PO4-), chromium (Cr), mercury (Hg), total coliform etc. Analysis of water quality data from December to April reveals that river water deteriorates from January, reaches its worst in February and March and then start to improve. Suspended solids and turbidity remain high in flood seasons. Required dissolved oxygen for water supply prevails only in the Dhaleswari river (downstream of Nabinagar and Kalagachia) and in the short downstream reach of Shitalakya River (from Mashinabanda to Kalagachia) throughout the year. The rest of the river reaches maintain DO level usually less than 1.0 mg/l in the dry period. Dissolved oxygen content in the dry season is gradually reducing in the river system which is very alarming. Very high BOD and COD value along with high concentrations of nitrogenous compounds are the indications of the severe extent of pollution in the entire river system. Concentrations of phosphate (PO4-), chromium (Cr), mercury (Hg) are within acceptable limits with negligible exceptions in few reaches. The sustainability of peripheral rivers as sources of Dhaka city water supply is evaluated on the basis of storage volume and water quality. Buriganga is so extremely polluted that it can't be used for water supply. Turag and Balu river do not have sufficient storage. Shitalakhya and Dhaleswari are two potential rivers for water supply. But water quality in some reaches of these rivers is not satisfactory. The Dhaleswari river (downstream of Nabinagar and Kalagachia) and the short downstream reach of Shitalakhya river (from Mashinabanda to Kalagachia) are the least polluted reaches. Thus these two reaches can be used for water supply providing required treatment for few contaminants. In general water quality is degrading. The study findings highlights that it is essential to make provisions for improving water quality in the peripheral river system to sustain city water supply system and the ecosystem in the rivers. For sustainable water supply with the help of peripheral rivers pollution must be controlled and we have to develop a common effluent treatment facility for the entire peripheral river system.

ACKNOWLEDGEMENTS

The authors wish to express their deepest gratitude to Dr. M.A. Matin, Professor, Department of Water Resources Engineering, BUET for his guidance and encouragement.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

COST EFFECTIVE BANK PROTECTION FOR HALDA RIVER

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ABSTRACT

Bank protection has initiated in various erosion prone parts of the Halda River in Chittagong, recently. However, major area of this riverbank remains vulnerable and immediate protection measures are vital. The aims of this work are to identify those locations and provide guidelines for cost effective riverbank protection system through a two phase field survey. In the Phase-I, intensive field study has been carried out to locate recently eroded banks of the Halda River. To aid the reported study, field survey Phase-II was carried out on applied bank protection techniques/materials of Jamuna River adjacent to Pabna and Sirajganj districts. A range of materials has been found to be used for this purpose including Cement Concrete (CC) blocks, geobags (sand filled geotextile bags), Sand filter, Khoa (Brick chips) filter, geo filter, and Chatka (Bamboo binding). Among these protection materials, geobags showed better hydraulic performances than the commonly used CC blocks. Thus, after critical investigations, latter part of this paper highlights the possible cost effective bank protection measures for Halda River based on the Phase-II observations. A follow up study (not reported here) would cover a series of laboratory hydraulic experimental study on different riverbank protection materials.

Keywords: *Cost effective, bank protection, Geobag, Halda River, Jamuna River*

1. INTRODUCTION

Riverbank erosion is a common and recurrent natural hazard that observed mostly during the largest floods. The causes of riverbank erosion are identified by Przedwojski *et al.* (1995) as: (i) Category I : soil particles erosion from the bank surface due to either a continuous or intermittent process over a period of time; (ii) Category II: sequential failures of small sections of bank material; and (iii) Category III: destabilization of a single large section of bank. The intensity of bank erosion varies widely from river to river as it depends on river flow hydraulics, topography, morphology and the properties of bank materials.

The adverse effect of flash flood in Halda River, Chittagong, turned to severe bank erosions in places for instance close to major bends (Daily Star, 2011a) (Figure 1) as well as adjacent villages for example Gunamordan (Daily Star, 2011b). Among the natural bends in Halda River, major bends namely Machuaghona, Barighona, Ankurighona, Sonaimuri and Gorduara have been cut down to strengthen the river during last century (Daily Star, 2011a). Traditionally this artificial river strengthening has been carried out by the local inhabitants to slow down the river current aimed to protect their homes and lands from erosion (Daily Star, 2011a). Due to lack of engineering knowledge, such local initiatives failed to achieve long term solutions. There are some institutional supports also practicing for the bank protection. However, almost in every flood season both protected and unprotected places in the riverbank experienced erosion and subsequent riverbank protection works have been practicing.

Composition of the braided stream and unstable bank materials accelerates massive river bank erosion in the major rivers in Bangladesh (RSP, 1996). Recently deposited bank materials are dominating the Jamuna riverbanks (RSP, 1996; Figure 1). Studies show that these loosely packed bank materials consist of silt and fine sand i.e. weakly cohesive silty- sand are highly susceptible to erosion (RSP, 1996). One of the recent innovation is using the riverbed deposited materials for preparing geotextile based sand bags (Geobags) and thus geobags are engaged for long term riverbank protection works (Akter *et al.*, 2011). So, the experience from erosion protection for a larger river like Jamuna is expected to offer better supports for Halda riverbank protections.

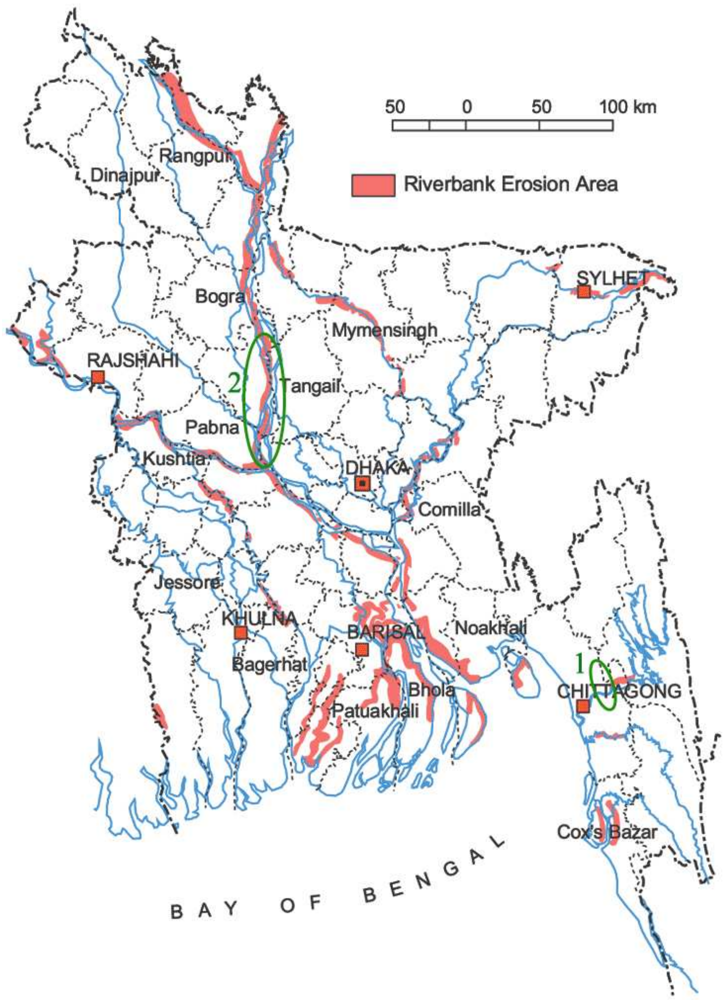

Figure1: Study area (**1**: Phase I and **2**: Phase II) (BWDB, 2011)

2. METHODOLOGY

The aims of this study were to identify riverbank failure locations (both protected and exposed) and provide guidelines for cost effective riverbank protection system through a two phase field survey.

- In the Phase-I, intensive field study has been carried out to locate recently eroded banks of the Halda River (Figure 1). Additionally, this phase considered the bank protection materials used and their performance; and
- Field survey Phase-II was carried out on applied bank protection techniques/materials of Jamuna River (Figure 1) adjacent to Pabna and Sirajganj districts.

2.1 Phase-I

For field investigation in downstream of Halda River, 1.5 km close to the landmark Madunaghat Bridge has been investigated. Both sides of the river were inspected visually. Based on the nature of failures in the exposed riverbank, the possible reasons of bank failures were noted down. Similarly, the protected banks prone to instability were considered for qualitative comparisons with Phase-II findings.

2.2 Phase-II

Jamuna river bank protection works were noted down while the field study was carried out in Pabna and Sirajganj district.

3. FINDINGS

Relevant data collected from field visit are discussed in the following sections:

3.1 Halda (Phase I)

Depths of Halda River are of 10 to 15 m (upstream), 12 to 16 m (downstream) in the landmark Maduna ghat and 20 to 24 m in Burishchar (around 2 km close to Maduna ghat) (Ahmed *et al*., 2007). Water level in Halda River fluctuates between 0.5 and 1.5 m during high and low tide and the water velocity ranges between 2.2 and 4.2 m/sec (Ahmed *et al*., 2007).

To note down the riverbank displacement, field study was carried out in dry season during December 2011.

Findings are:

- the exposed riverbank showed all the three categories of failures described by Przedwojski *et al.*, (1995) (stated in Section 1), i.e. due to bank surface soil particle erosion, bank section displacement either in smaller or larger segments (Przedwojski *et al.*, 1995; Figure 2); and
- the protected riverbank both locally initiated and institutionally offered were observed in the surveyed area (Figure 3). These types of protective works are well known and commonly practiced in Bangladesh. Bamboo bindings can offer a short term protection while the Cement Concrete (CC) blocks are considered for long design period. To achieve the design life of CC block revetment, it is urgent to get details understanding on the CC block revetment performances as well as possible alternatives.

3.2 Jamuna (Phase II)

Two types of structures are normally practiced to protect Jamuna River bank, i.e. revetments and spurs (Zaman, 2011). Field survey in this study can be summarised as:

- A range of materials could be used for riverbank stabilization purpose those including Cement Concrete (CC) blocks, geobags (sand filled geotextile bags), Sand filter, Khoa (Brick chips) filter, geo filter, and Chatka (Bamboo binding);
- Among these protection materials, Sand filter and brick chips normally used against small scale failures, geofilter prone to random slide whereas the bamboo binding can offer a short term protection normally 1 to 3 years (Karim, 2011);
- However, in recent years, geobag and CC block revetments are commonly used due to their stability (Figure 4); and
- Geobags showed better performances than the CC blocks not only in terms of hydraulics but also:
    a. being rigid in nature hydraulically, CC block generally settles with gap during dumping in field, whereas flexible geobags can easily overcome this shortcoming. The latter offers cost effectiveness (in terms of both manufacturing and implementing costs) and environment friendly nature by using river induced sand and local labour;
    b. Geobag can rather reduce manufacturing as well as carrying time (as the empty geotextile bags normally carry close to the site for sand filling), while the CC block needs considerable time in both phases; and
    c. Geobag also provides fairly long term protections (present design life is approximately 100 years).

In the Phase I survey no geobag protected revetment were observed. So, the recent experiences from the Jamuna river bank protection, geobag revetment could be suggested to reduce the Halda riverbank failures. Furthermore, to strengthen the protected riverbank of Halda, some recommendations are provided in light of the experiences from the Jamuna which are described in next section.

Location: left river bank, directed from Maduna ghat towards upstream of Halda, Burishchar

Observations:

1. small-scale exposed toe of the riverbank;
2. deployments of bank surface.
3. possibly removal of soil particles from the bank surface

(a) Category I

Location: left river bank, directed from Maduna ghat towards upstream of Halda, Burishchar

Observations:

1. displacements of relatively smaller bank sections; and
2. possibly river bank section destabilize due higher river flow action on this.

(b) Category II

Location: left river bank, directed from Maduna ghat towards upstream of Halda, Burishchar

Observations:

1. devastating bank failure spread just in one flood season; and
2. probably continuous displacements of large bank segments resulted the massive bank failure.

(c) Category III

Figure2: Field survey in Halda River (exposed riverbank)

Location: right riverbank, towards Maduna ghat

Observations:

1. locally initiated temporary protection offers protection against Category II failure;
2. Chatka (bamboo binding) acting as groynes for river training.

(a) Bamboo binding protection

Location: left river bank, directed from Maduna ghat towards upstream of Halda, Burishchar

Observations:

1. institutional initiated long term protection;
2. possibly the absence of interaction between Cement Concrete (CC) blocks and riverbank surface caused displacements of these protective materials.

(b) Cement Concrete (CC) block revetment

Figure 3: Field survey in Halda River (Protected riverbank)

(a) Geobag revetment

(b) CC block revetment

Figure 4: Revetments in Jamuna River bank

4. RECOMMENDATION FOR HALDA

Zaman (2011) has noted a series of design parameters based on the experiences of geobag revetment in the Jamuna River. Some of those are relevant to our Halda River study. To adopt geobag revetment for Halda riverbank, following issues should be taken into considerations:

- river induced wave, those are causing sudden bank segment failures;
- local velocity considerations for design aspect (where local velocity > 3 m/s, mean);
- selection of proper design slope, places of steep slope should be reviewed;
- maximum toe scouring should be taken into considerations; and
- geotechnical details, knowledge on river morphology and topography would strengthen the bank protection works in Halda River.

5. CONCLUSIONS

This study aimed to suggest cost-effective bank protection measure for the erosion prone parts of the Halda River in Chittagong through a two-phase field survey. The field observations can be concluded as:

- Phase-I (Halda River): cost-effective protections (both CC Block and Bamboo binding) failed to offer the long term protections and possibly there is a knowledge gap on different types of failures occur in exposed riverbank; and
- Phase-II (Jamuna River): experiences with few protection materials suggest geobags for long term riverbank protection.

Thus, after critical investigations, last section of this paper highlighted the required consideration to achieve the cost effective bank protection measures for Halda River. Further study (Khan *et al*., 2012) has been considered a series of laboratory hydraulic experiments on various riverbank protection materials. Detail outcomes of that study expected to provide a details design guideline for Halda riverbank protection.

ACKNOWLEDGEMENTS

The authors are grateful for the financial, laboratory and logistic supports of the department of Civil Engineering, Chittagong University of Engineering and Technology (CUET). Appreciation extended to staffs of the Jamuna-Meghna River Erosion Mitigation (JMREM) project, Pabna, and Sirajganj Town Protection Embankment project, Sirajganj, Bangladesh for their kind support during field survey and providing relevant information.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

THE IMPACT OF SEA-LEVEL RISE ON MANGROVE ECOSYSTEMS IN BANGLADESH – A REVIEW

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ABSTRACT

Mangrove forests are abundant along the world's tropical coastlines. They compose ecosystems that are highly adapted to saline wet environments and that, owing to their great productivity, have become valuable also to humans. The largest coherent mangrove forest is the Sundarbans situated in Bangladesh, a country built on the vast delta of the Meghna, Ganges and Brahmaputra rivers. Topography is remarkably flat and the landscape is very close to sea-level. Because of this, and because of the evident spatial proximity to the oceans, the Sundarbans are naturally exposed to sea-level rise. The rate of any change is important for an ecosystem's ability to adapt. Unfortunately, owing to described spatial conditions and as read from observations and future projections, rates of sea-level rise are high and reach far inland. The following inundation and salt intrusion initiate alteration in species distribution and a massive loss of habitats and vulnerable species. Because the ecosystem is already at the very boundary of high water levels and salinity tolerance, small changes bring large impacts. Relevant studies and monitoring, together with mitigation actions, are therefore crucial.

Keywords: *Sea-level rise, Ecosystem impacts, Mangroves, The Sundarbans, Ecosystem Mitigation*

1. INTRODUCTION

Mangrove ecosystems cover around one-fourth of the Earth's coastline in tropical areas. The total area is estimated to be somewhere around 167000km to 2181000km2 and the largest continuous area is situated in the Sundarbans, Bangladesh. This forest exceeds 600000ha in size (Miththapala 2008) and is located along the coast of Western Bangladesh and in neighboring India (Agrawala et al. 2003). The Mangroves have a great social as well as ecological value; they are one of the world´s most productive ecosystems. The products and services that the ecosystem provides globally have been estimated to range between $200000-$900000 per hectare every year (McLeod & Salm 2006). Bangladesh is a heavily populated country situated in South East Asia. Most of the country's land area is underlain by the floodplains that constitute the vast delta of the Meghna, Ganges and Brahmaputra rivers (Agrawala et al. 2003). Continuous sedimentation in the river mouths of these rivers has built the delta on a level with sea surface and successively lowered the river's gradients. Hence, the region is exceptionally flat and it is close to the sea-level and coast line respectively. Because of this, the region is vulnerable in terms of sea-level rise; a small rise results in a vast land area inundation. Sea-level in the Indian Ocean outside Bangladesh has been observed rising at a higher rate than the global average levels and from IPCC AR4 SRES scenarios; levels are projected to increase –at accelerating rates (Bindoff et al. 2007). Further, the coastal delta region is still highly tectonically and morphologically active (Agrawala et al. 2003). Impact studies on sea-level rise in this area are thus of great importance. Mangroves are some of the world's toughest ecosystems; they possess an unusual ability to tolerate extraordinary amounts of stress. At the same time however, mangroves are one of the most threatened ecosystems when it comes to global sea-level rise (McLeod & Salm 2006). The Sundarbans in Bangladesh is no exception and climate change is not their only threat. Mitigation management in order to aid the ecosystem´s resilience is therefore necessary. Because of a tight interaction, studying ecosystem development is also a way of climate change detection. Much is already accomplished, e.g. actions supporting siltation and stabilization, and much is yet to be done. This review aims to illuminate the issue of sea-level rise and its impacts on the Sundarban mangrove ecosystem in Bangladesh.

2. MANGROVE ECOSYSTEMS

Mangroves are tropical/sub-tropical forests consisting of salt tolerant, flowering species of trees, shrubs, palms and ground ferns that grow at the boundary between land and ocean. Mangrove ecosystems can be found in areas which are flooded from time to time, primarily in tropical or sub-tropical regions, with a maximum development between 250N and 250S (McLeod & Salm 2006).

There is a great regional variance in mangrove ecosystems, from highly productive tall forests to lower more shrub-like trees with lower productivity, all depending on the amount of salt stress as well as temperature. Temperature seems to be the main controlling factor and the most diverse mangrove ecosystems are found in areas where water temperatures exceed 240C during the hottest month (McLeod & Salm 2006).

The plant species constituting a mangrove forest are of many different genera and families, not necessarily closely related. They are therefore hard to define since they are a n ecological assemblage of species rather than a taxonomically or morphologically grouping. Some mangrove species may even be able to survive in other habitats, while other species can only exist when the special conditions of mangroves are met (Saenger 2002).

Because of the plants distribution in areas where the tide rises and fall daily, adaptations to tolerate high changes in salinity and to low amounts of oxygen in the soil are needed. The shifts in tides result both in an unstable substrate and changing salinity. To counteract the unstable substrate, mangrove plants have developed stilt roots that grow sideways instead of penetrating the soil too deep. To meet the salinity-problem the plants are adapted in a way so that they are able to conserve water exceptionally well, e. g. by having leathery, waxy leaves that reduce water loss. Some species are also able to exclude the salts during water uptake, while others are able to excrete salt through specialized glands. Adaptations to low amounts of oxygen in the soils have given mangrove plants their characteristic look with the roots that are above ground, these “breathing roots” (figure 3) can transfer oxygen to the root system below ground (Miththapala 2008).

Figure 1: A picture of a mangrove forest, with visible “breathing roots”. (Wickimedia Commons 2009)

Mangroves provide a number of different ecosystem services that are of great socio-economic importance, for example as provider of food such as seafood and fish. Mangroves also provide lumber for building materials and fuel wood and charcoal for energy. Ecologically mangroves offer habitats for a number of different organisms such as molluscs, crustaceans, insects, birds, reptiles and monkeys. Other services include protection against destructions caused by storms, trapping sediments and thereby stabilizing the coastal land as well as filtering and trapping pollutants (McLeod & Salm 2006).

3. THE SUNDARBANS

As mentioned earlier the Sundarbans mangrove is the largest continuous mangrove ecosystem in the world (Miththapala, 2008). It is located in the south western region of Bangladesh as well as in India, along the Bay of Bengal. The ecosystem has been created by the deposition of materials from the Ganges, Brahmaputra and Meghna rivers. The Sundarbans contains a great biodiversity with at least 69 different plant species and as many as 425 different animal species (Agrawala et al. 2003), including 260 species of birds (UNESCO 2011). One of the Sundarbans mangroves key species is the Royal Bengal tiger (Panthera tigris tigris) which is listed as Endangered by the IUCN Red List of Threatened Species (IUCN 2011). It is the only place in the world where tigers are adapted to live in mangroves (Louks et al. 2010). Other important species in the Sundarbans include the estuarine crocodile (Crocodylus porosus) and the Indian python (Python molurus). Both the Indian and the Bangladesh Sundarbans has been declared a world heritage site by UNESCO (UNESCO 2011).

4. SEA LEVEL CHANGE IN BANGLADESH

During the 20th century, tide gauges and satellite altimetry have measured a global average sea-level rise of approximately 17 cm. Further, observations from satellite altimetry, which has improved data availability and reliability considerably, indicate an accelerated rate of 3 mm/year since 1993. As for future projections, IPCC expects rates to increase to 4 mm/year during the 21th century and levels in 2100 will range between 0.18m and 0.59m higher than the 1990 levels in different SRES scenarios. The rise has been attributed to recent global warming and the resulting increase in ocean water volume from (i) thermal expansion and (ii) additive input from land- and ocean-based ice bodies. Large uncertainties lies in the contribution of melting ice sheets and the rise might therefore be higher than projected (Bindoff et al. 2007). Other estimates are likewise much larger; Broadus (1993) for example, suggests an increase of 1 m by 2050 and almost 2 m by 2100. The difference is evident, yet sea-level rise by 2100 could be substantial. Beyond 2100, whatever happens, sea-levels will keep rising because of the slow response in the ocean system (Houghton 2009). Naturally, owing to various controlling factors, sea-level rise varies geographically. Sea-levels in the Indian Ocean outside Bangladesh have been observed rising at noticeably higher rates, about 3.5 mm/year, compared to global average (figure 2) (Bindoff et al. 2007). The water volume of the ocean is not the only thing that is responsible for this regional sea-level change. Along the coast of Bangladesh particularly, sea-level is in interplay with active tectonic subsidence as well as substantial sedimentation in the massive delta of Ganges and Brahmaputra. It is a coastal deltaic region that is highly active and dynamic in its morphology (Agrawala et al. 2003). In addition, much of the interannual sea-level variation in this region has been attributed to the El Niño Southern Oscillation (Bindoff et al. 2007).

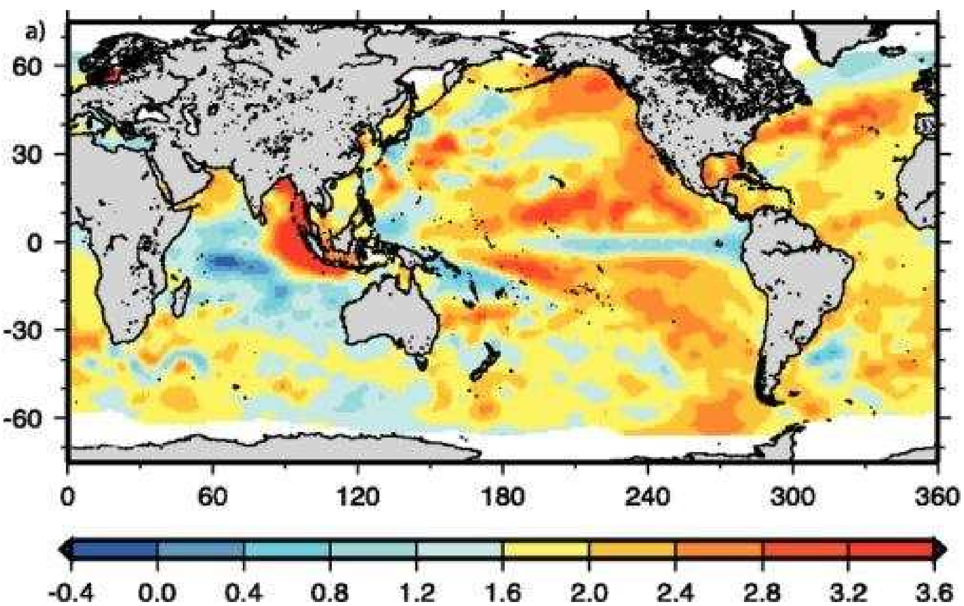

Figure 2: "Geographic distribution of long-term linear trends in mean sea-level (mm yr^{-1}) for 1955 to 2003 based on the past sea-level reconstruction with tide gauges and altimetry data (updated from Church et al., 2004)" (Bindoff et al. 2007).

Apart from morphology, more climate elements influence regional sea-level rise. These are often affected by climate change also, directly or indirectly, and are important for sea-level change impacts. *Precipitation,* as an example, is key when it comes to land hydrological regimes. It is controlled by the Asian monsoon that brings more than 80 % of annual rainfall during one wet season in June to September. With global warming and projected strengthening of pressure gradients in Asia, the monsoon is expected to intensify and bring more heavy rainfall during the wet season and less during the dry. This, together with increased *melt water input* from the Himalayan glaciers, will lead to higher monsoon river flows in the Ganges and Brahmaputra rivers and, as consequence, enhanced flooding both inland and in the coastal areas (Agrawala et al. 2003 and Begum & Flemming 1997).

Nonetheless, the largest impact associated with climate change and sea-level rise is that from *extreme high levels* such as during storm events. Higher sea surface temperatures and sea- levels are projected to result in intensified cyclone activity, heavier precipitation and in storm surges reaching further inland (Agrawala et al. 2003 and Bindoff et al. 2007). Studies have shown that there is indeed an evident global increase in extreme high sea-levels and that it is spatially associated to regional climate change (Woodworth & Blackman, 2004). Hence, damage from storm events and high water levels are projected to increase in response to sea- level rise.

Except from extended and deepened inundation and flooding (Begum & Flemming, 1997) (figure 3), *salinization* is a natural consequence of sea-level rise. This comes both for low lying coastal delta regions and for rivers further inland, i.e. through the backwater effect. The latter together with extreme high river flows, give more extensive and "saltier" inland and riverine flooding; areas and ecosystems not normally inundated would now experience saline water intrusion. The saline front along the coast, and its exact location during the monsoon, will be determined by the opposing effects of (i) an increased freshwater discharge from the Ganges and Brahmaputra river systems and (ii) the boosted backwater effect from sea-level rise. During the dry season, with a continuous decreasing lower precipitation, the balance will shift and salinity levels could rise to higher seasonal levels than before (Agrawala et.al 2003).

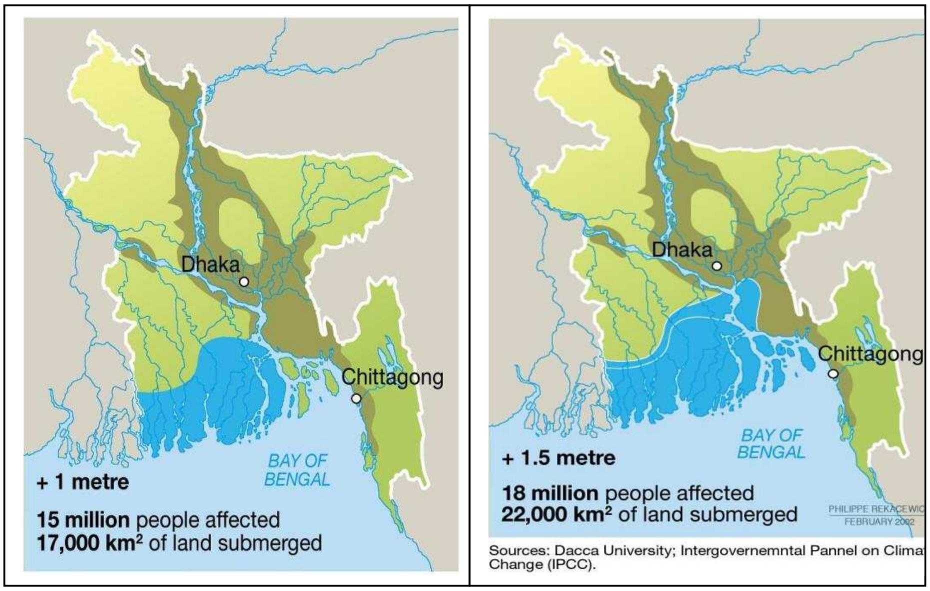

Figure 3: Extent of inundation and impacts in Bangladesh following sea level rise (UNEP 2011)

In conclusion, a rise in sea-level, in combination with amplification in freshwater discharge, affects the entire hydraulic regime of Bangladesh. Alteration of the river hydrodynamics (Begum & Flemming, 1997), increased flooding, and salinization are the most prominent consequences significantly affecting the local ecosystems.

5. IMPACTS OF SEA-LEVEL RISE ON THE SUNDARBANS MANGROVE ECOSYSTEM

Sea-level rise is the largest threat related to climate change that mangroves have to face (Louks et al. 2010). The possibility for mangroves to adapt to sea-level rise requires that the rise takes place slow enough, that there is space for a landward shift and that other environmental conditions are met (McLeod & Salm 2006). The Sundarbans face both direct and indirect threats related of sea-level rise, the direct effects resulting from inundation and the indirect through salinity intrusion (Agrawala et al. 2003).

A sea-level rise of 0,45m would inundate 75% of the Sundarbans, while a rise of 0,67m would submerge the entire system. If the Sundarbans where to be inundated many valuable species would be lost. Because the Sundarbans is the largest mangrove ecosystem in the world its survival is of great value and a potential rise in sea-level could cleverly alter or completely destroy the entire ecosystem (Agrawala et al. 2003). Louks, et al. (2010) suggest that as little as a 28cm sea-level rise above 2000 values will be significantly enough to wipe out all the suitable tiger habitats of the Sundarbans mangroves.

The largest effect of climate change and sea-level rise on the Sundarbans will be during the dry season from November to April. As mentioned, the decrease in freshwater flow due to predicted decrease in precipitation together with higher levels of evaporation will, when coupled with sea-level rise result in an enhancement of the salinit y levels during the dry seasons. A consequence of the salt water intrusion to the Sundarbans is that most of the mesohaline areas (moderately brackish water with salt concentrations of 5-8ppt) will turn into polyhaline areas (brackish waters with salt concentrations of 18-30ppt) and the oligohaline areas (low salinity waters, 0,5-5ppt) will be highly reduced (Agrawala et al. 2003).

A number of different studies have shown that both the natural regeneration of the mangrove forests and the forest succession depend on the salinity regime. It has therefore been argued that the effects of climate change and sea-level rise will have an adverse effect on the regeneration and succession of the mangrove. It is for example thought that some of the freshwater loving plant species, in particular the Sundari (Heritiera fomes) trees, will disappear entirely and areas that are now dominated by high quality timber will be replaced by trees with lower quality and shrubs. This will contribute to the canopy becoming less dense and plant height would decrease considerably. This will have a great negative influence on the mangroves productivity. Because the composition of the vegetation is of great importance to the mangrove fauna, the expected change in regeneration and succession will have a substantial long-term effect on the sustainability of the entire ecosystem (Agrawala et al. 2003).

6. HUMAN INFLUENCE AND MITIGATION MANAGEMENT

A variety of causes might put a mangrove forest in a stressed situation. One of the most important of these, as discussed, is climate change and its impacts, but there are other factors that might also affect mangroves forest, directly or indirectly. These factors can be categories into two groups: natural and anthropogenic. Natural factors can be e.g. biological pests and parasites, animal grazing and intruding exotic species. Human activities that can significantly destruct the mangrove forest are e.g. exploitation, agriculture, and pollution (Kathiresan, 2005).

When a mangrove forest is destroyed, it is extremely difficult or even impossible to replace the habitat, mitigation management plans are therefore urgent. The strategies for conservation should start as soon as possible to decrease the negative effect in the future. Actions can be made on different scales: (i) globally, e.g. trough general climate change mitigation, (ii) regionally through international cooperation and (iii) locally by conservation strategies and limitation of ecosystem threats (Loucks et al. 2010). In Bangladesh, as well as in other countries with mangrove forests, some efforts have already been made. For instance there are strategies that help accelerate the process of siltation and stabilization of soil in the forest that create better protection and environment for wildlife, fishes and other estuarine and marine fauna (Islam & Wahab 2005).

There are many ways of managing sea-level rise mitigation in the mangrove forests. First of all, continuous monitoring of the mangrove ecosystems, the impacts and resulting vulnerability is of great importance (Kathiresan 2005). Mapping and identification of the different habitats and species within the ecosystem, as part of the monitoring, is also central; those that are more vulnerable would need more intense preservation. An example of a measure to put into practice is the establishment of greenbelts as buffer zones to prevent high level damages (McLeod & Salm 2006). Restoration of critical, already degraded, areas is of course also essential for the mangrove´s resilience. All mitigation actions are crucial, despite the associated high costs (Lewis & Streever 2000).

7. DISCUSSIONS

The ongoing and projected sea-level rise in Bangladesh, resulting from global climate change and magnified by regional dynamics, is of considerable magnitude. Adding to this, the regional coastal topography is exceptionally flat and close to sea-level; small changes in sea-level would bring extensive inundation and salinity problems. Ecosystems established in these environments are therefore in a very vulnerable position. Moreover, because the baseline changes, impacts are not restricted to coastline only but extend inland through the backwater effect; the entire hydrological dynamics would also be affected and ecosystem conditions with it. The fact that other climatic elements are shifting simultaneously, especially precipitation and the resulting freshwater regime, makes the outcome far from straightforward. Increased river discharge for example, counteracts the landward migration of the saline front.

These changes will have large consequences for local ecosystems such as the Mangrove forests because of their distribution in the areas between land and ocean. However, its distribution might also be its advantage since it makes it possible for the ecosystem to climb inland as the ocean rises. This requires both space for a landward movement and that environmental demands are met, sea-level rise also has to take place slow enough. This does not seem to be the case for the Sundarbans. Because the Sundarban mangroves are situated in a flat, low lying area, only a slight increase in sea-level will inundate a large part of the ecosystem resulting in devastation for the organisms living there. Adding to this situation is the observed and projected relatively high rate of sea-level rise. Mangrove ecosystem resilience therefore appears cumbersome and mitigation activities crucial.

Loss of plant species due to salinization and the resulting severe alteration of the biome will result in an even further loss of animal species. Because the Sundarbans houses a great amount of biodiversity and is the home to a great number of important species such as the Royal Bengal tiger, the estuarine crocodile and the Indian python, the survival of this unique ecosystem is of great importance.

The management of mangrove forests in the Bangladesh needs strategies and actions for both short- and long-term periods. Particularly some actions need a long time to reduce the negative impacts which have to be considered in management strategies. As mentioned, some of these actions have to be done between several countries, hence international coordination is essential. This is important for the study area also since the Sundarbans are shared between India and Bangladesh. Although, in this report, focus is on sea-level rise and its impacts, it should be noticed that other factors, such as social, economic and biological, also have big influence on mangrove forests. Planning strategies therefore, need to include all these factors to improve the efficiency of mitigation actions.

Ecosystem alterations are often indirect effects of, and strongly coupled to, climate change. In this study e.g., coupling is through sea-level rise but it might also be from simple temperature and precipitation change. Because of the tight interaction, studying ecosystem development is therefore also a way of climate change detection.

8. CONCLUSIONS

Climate change will have significant effect on sea-level rise round the Indian Ocean and in the area of study. The Sundarban mangroves are situated in the low lying areas of the Bay of Bengal, and therefore a large part of the ecosystem, if not the entire, will be inundated and destroyed by just a small rise in sea-level. The rate of sea-level change is key to the adaption ability of the mangroves. Unless it is slowed down so that the mangroves have a chance to move inland, loss of a great amount of both plant and animal species will be inevitable. Even if the ecosystem is not entirely inundated though, the organisms living there have to face the stress of salinization that can result in the disappearance of some important plant species, entirely rearranging the biome and resulting in a further loss of species. The end result will be a loss of a both socially- and ecologically important ecosystem.

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APPLICATION OF A SIMPLE STOCHASTIC MODEL FOR PREDICTING GROUNDWATER TABLE FLUCTUATIONS IN A SHALLOW AQUIFER OF BANGLADESH

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ABSTRACT

This paper demonstrates a methodology for modeling the groundwater table fluctuations observed in the shallow unconfined aquifer of Bangladesh. For this purpose, Kushtia district has been selected as a case study area. A groundwater table monitoring well is selected in each upazilla (sub-district) under the study area. The time series water table observations collected on a weekly basis during the period from 1999 to 2006 from each well is used for experiments. Empirical result demonstrates that observed data of groundwater level exhibits cyclic patterns and shows an annual periodicity. Box and Jenkins univariate stochastic models widely known as ARIMA, are applied to simulate groundwater table fluctuations in all monitoring wells under consideration in the study area. The predicted data using the best models are compared to the observed time series and the resulting accuracy is checked using different error parameters. The simulation revealed that ARIMA models generate reasonably accurate forecasts in terms of numerical accuracy. The results also showed that the predicted data represent the actual data very well for each monitoring well.

Keywords: *ACF, ARIMA Model, Periodogram, PACF, Seasonality, Stochastic*

1. INTRODUCTION

Various quantitative analyses are required for complex and dynamic nature of water resources systems to manage it properly. Groundwater level fluctuations over time in shallow aquifer systems need to be evaluated for formulating or designing an appropriate groundwater development scheme. Models can be simple images of things or can be complex, carrying all the characteristics of the object or process they represent. A complex model will simulate the actions and reactions of the real thing. There has been considerable research on modeling for various aspects of groundwater management strategy. The conceptual and physically based models are the main tools for depicting hydrological variables and understanding the physical processes that are taking place within a system (Anderson and Woessner, 1992). There are varieties of techniques and methods available for analyzing the groundwater level fluctuation through probability characteristics, time series methods, synthetic data generation, theory of runs, multiple regression, group theory, pattern recognition and neural network methods. And yet, time series modelling can be the better option for the area where nothing but the hydrological time series data is in hand. A time series model is an empirical model for stochastically simulating and forecasting the behaviour of uncertain hydrologic systems (Kim et al., 2005). However, it is very common that sufficient hydrogeological parameters and domain boundary or initial conditions are often unavailable for physical modelling. Most often these parameters are very difficult to obtain as well because of the several natural and anthropogenic factors (Kim et al., 2005). The stochastic time series models are the popular and useful tools for medium-range forecasting and generating the synthetic data. A number of stochastic time series models such as the Markov, Box–Jenkins (BJ) Seasonal Autoregressive Integrated Moving Average (ARIMA), deseasonalized Autoregressive Moving Average (ARMA), Periodic Autoregressive (PAR), Transfer Function Noise (TFN) and Periodic Transfer Function Noise (PTFN), are in use for these purposes (Box et al. 1994; Brockwell and Davis, 2002; Hipel and McLeod, 1994). The first three of these are univariate models and the last two are multivariate models. In addition, the PAR and PTFN models are periodic in nature (Mondal and Wasimi, 2007). The selection of an appropriate method for modelling a particular problem depends on many factors, such as the number of series to be modelled, required accuracy, modelling costs, ease of use of the models, ease of interpretation of the results, etc. (Mondal and Wasimi, 2006). Several applications of all these models have been proved to be useful for analyzing groundwater level fluctuations over time in several groundwater hydrology applications (Houston, 1983; Knotters and De Gooijer, 1999; Lee and Lee, 2000; Salas and Obeysekera, 1982; Tankersley et al., 1993) and also for other relevant engineering applications (Kumar and Jain, 1999). However, it is also reported in the literatures that when the number of series to be modelled is relatively small and a large expenditure of time and effort can be justified, BJ method (seasonal ARIMA) is generally preferable. The choice is due to its inclusion of a family of models, which can be fitted to a wide variety of time series processes. An inherent advantage of the seasonal ARIMA family of models is that few model parameters are required for describing time series, which exhibit nonstationarity both within and across seasons (Hipel and McLeod, 1994; Mondal and Wasimi, 2006). This paper attempts to develop seasonal ARIMA models as well as generalized equations using the parameters of Seasonal ARIMA models to show on of the modelling schemes regarding the groundwater level fluctuations. This paper might be a supplementary material for further research which surely provides useful information for developing the appropriate groundwater resources management strategy.

2. METHODOLOGY

2.1 Description of Study Area and Data Acquisition

Kushtia district of Bangladesh has been selected as a case study area for this study (Figure 1) based on the data availability of groundwater level (GWL) data. It has an area of 1621 square kilometres and is bounded by Rajshahi, Natore and Pabna districts to the North, by Chuadanga and Jhenaidah districts to the South, by Rajbari district to the east and by West Bengal and Meherpur district to the west. The upazillas (sub-district) are Kushtia Sadar, Kumarkhali, Daulatpur, Mirpur, Bheramara and Khoksa. Ganges, Gorai, Mathabhanga, Kaligonga and Kumar are the main rivers flowing through the district. The average maximum and minimum temperature is 37.8 ^{0}C and 11.2 ^{0}C, respectively with an annual average rainfall of 1,467 mm.

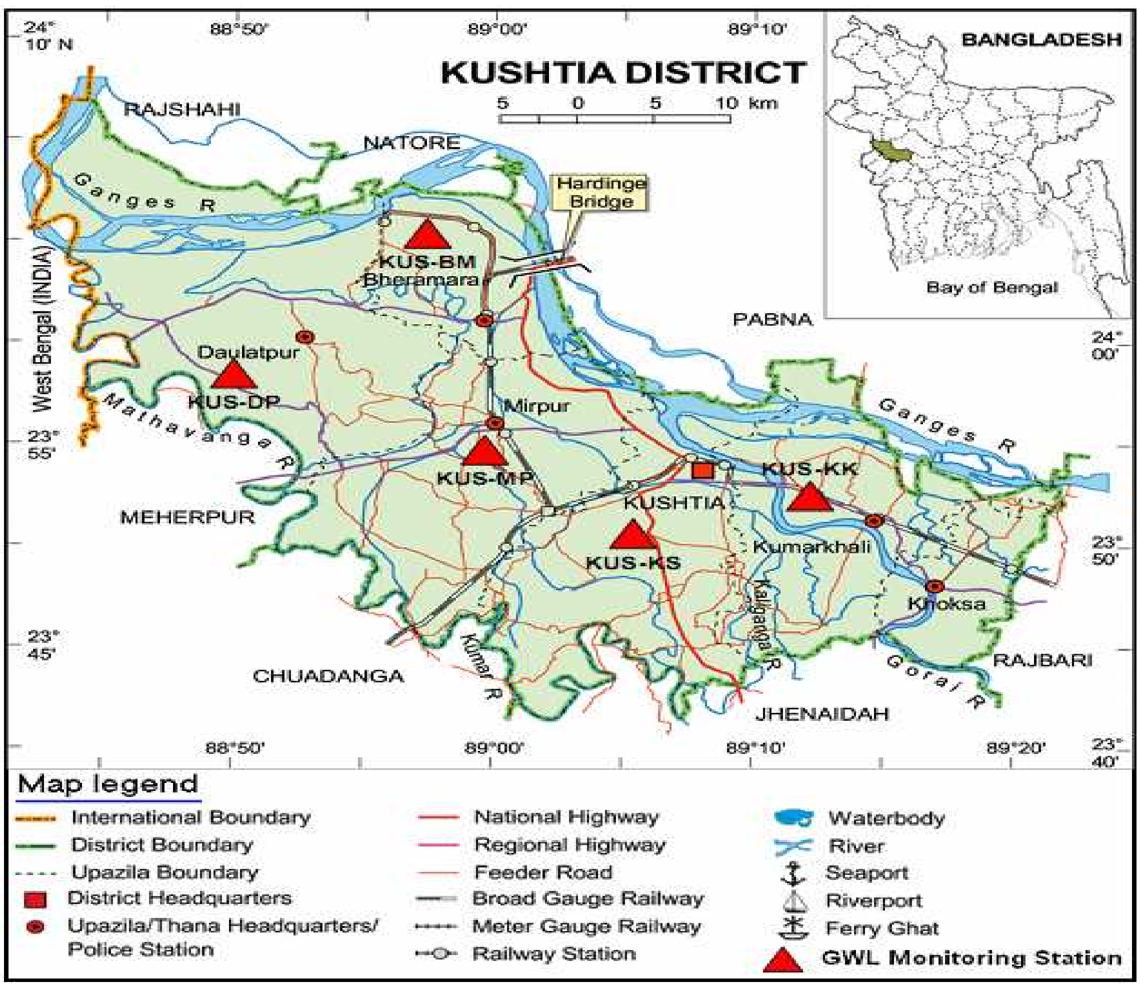

Figure 1**:** Study area showing the locations of BWDB GWL monitoring stations

A larger portion of the Ganges-Kobadak irrigation project (also known as G-K project) of the country is located in this district. During dry season, insufficient water supply from the Ganges River for irrigation activities in the command area set a huge burden on the underground water reserves to satisfy the ever increasing demand. Therefore, the groundwater withdrawal rate is comparatively higher in the area because of the growing demand for water due to intensive irrigation activities along with municipal and commercial uses of water, and other purposes. The time series data of the observed GWL is also available from the Groundwater Circle (GWC) of Bangladesh Water Development Board (BWDB). The fluctuations of the groundwater flow can be understood and visualized from the observed groundwater levels. Observed GWL of five monitoring wells for a period spanning over 1999 to 2006 is collected to conduct this study. It can be mentioned that at least one representative monitoring well has been selected from each upazilla (sub-district) based on data availability. The BWDB data is available on weekly basis with a unit of m.PWD. The m.PWD is the public works datum (PWD) used by BWDB which is located at 0.46 m below the mean sea level (MSL) near Bangladesh coast. For ease in identification, each monitoring well is given a unique name (e.g. KUS-DP) using the first three characters from its located district (Kushtia) with additional two characters from its located upazilla (e.g. Daulatpur) name. All the monitoring stations are positioned on the study area map in the ArcGIS® framework using their location (latitude and longitude) coordinates, 23°54′N 89°00′E / 23.90°N 89.00°E (Figure 1). Locations and their ground elevations are shown in Table 1.

Table 1: Locations and Elevations of the Monitoring Wells.

Location (Dist:Thana:Union:Mouza)	Well ID	Latitude	Longitude	Ground Elev. (m.PWD)
Kushtia, Bheramara, Bahadurpur, Golapnagar	KUS-BM	24.09	88.96	18.143
Kushtia, Daulatpur, Boalia, Taragunia	KUS-DP	23.98	88.83	16.619
Kushtia, Mirpur, Barui Para, Talbaria	KUS-MP	23.93	89.02	17.228
Kushtia, Kushtia Sadar, Alampur, Bittipara	KUS-KS	23.83	89.10	13.570
Kushtia, Kumarkhali, Chapra, Jodu Boyra	KUS-KK	23.84	89.20	12.960

2.2 Description of Seasonal ARIMA Model

Box et al. (1994) have generalized the ARIMA model to deal with Seasonality and have defined a general multiplicative seasonal ARIMA model, commonly known as a seasonal ARIMA model. The seasonal ARIMA model described as ARIMA *(p, d, q) (P, D, Q)*$_S$, where (*p, d, q*) non-Seasonal part of the model and (*P, D, Q*) Seasonal part of the model with a seasonality *S*, can be written as

$$\phi_p(B)\Phi_P\left(B^S\right)\nabla^d\nabla_S^D z_t = \theta_q(B)\Theta_Q\left(B^S\right)a_t \tag{1}$$

Where, $\phi(B)$ and $\theta(B)$ are polynomials of order p and q, respectively; $\Phi(B^S)$ and $\Theta(B^S)$ are polynomials in B^S of degrees *P* and *Q* respectively; *p* = order of non-seasonal autoregressive operator; *d* = number of regular differencing; *q* = order of the non-Seasonal moving average; P = order of seasonal auto-regression; *Q* = order of seasonal moving average. The ordinary and seasonal difference components are designated by $\nabla^d = (1-B)^d$ and $\nabla_S^D = \left(1-B^S\right)^D$ of orders *d* and *D*; *B* is the backward shift operator; *d* is the number of regular differences; *D* is the number of seasonal differences; S = seasonality. z_t denotes observed value at time *t*, where *t* = 1, 2,………*k*; and a_t is the Gaussian white noise or estimated residual at time *t*.

2.3 Modelling Technique for Seasonal ARIMA Model

Following the stochastic modelling principle, there are mainly four sequential but iterative steps in Seasonal ARIMA modelling approach such as (a) identification of the Seasonal ARIMA (*p, d, q*) × (*P, D, Q*)$_S$ structure, (b) parameter estimation, (c) diagnostic checking of model residuals, and (d) generation of forecasted values based on the known data (Box et al., 1994; Brockwell and Davis, 2002; Hipel and McLeod, 1994). However, the first to third steps are basically model calibration functions, and the last step is the checking for adequacy of the fitted model. At first, observed time series must be made stationary by applying the appropriate differencing of observed data. This stationary time series is frequently referred to as being prewhitened. In the identification stage, graphical methods such as periodogram, autocorrelation function (ACF), and partial autocorrelation functions (PACF) are useful to detect the Seasonality and the order as preliminary identification. The identification stage of Seasonal ARIMA modelling approach applies these functions to tentatively identify the model form for pre-whitened time series of the observed data. After identification of the seasonal ARIMA model structure, the model parameters are estimated by the method of maximum likelihood method (Box et al., 1994). All the developed models are then compared with respect to their error parameters. Appropriate model is then selected and forecasting is generated for the groundwater level for each monitoring well within the study area. The well-known statistical software packages MINITAB® and SPSS® for Windows were employed in this study to develop the models and all relevant statistical analysis within their software framework using Box-Jenkins ARIMA analysis. The present study investigates the pattern of GWL fluctuations for five monitoring wells in the Kushtia district of Bangladesh (Figure 1). For simplicity, the analysis and modelling of GWL data of only one monitoring well (KUS-DP) is explored in details, while that of the remaining monitoring wells data are used to support the results.

3. RESULTS AND DISCUSSIONS

3.1 Identification of Model Structure

The first step in developing a Box-Jenkins time series model is to checking of the stationarity and presence or absence of Seasonality in the observed time series data that needs to be modelled (Box et al., 1994). For model identification, regression and periodic analyses were carried out to find the stationary time series and periodicity or Seasonal components of the observed data (Table 2).

3.1.1 Regression Analysis

A simple linear regression model is developed for the observed series to characterize the trend component (Figure 2). The results of regression analysis are shown in Table 2. The trend of the overall GWL fluctuations develops through time which depicts a downward trend. The reason is that the water suction rate gets high during the dry season due to irrigation purpose and other water evolved activities. The first insight into the statistical properties of the time series is shown in Table 3. Performing the first differencing on the groundwater level series reduces the series mean from 12.414 to -0.001. The first differencing often results in a stationary mean value of approximately zero.

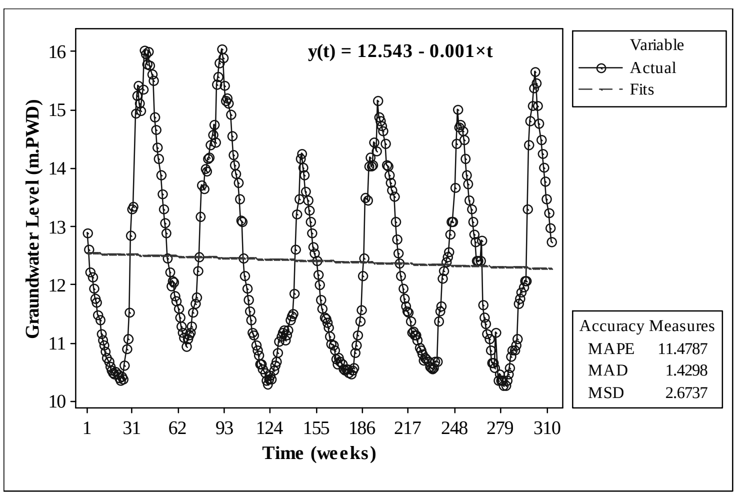

Figure 2: Linear Regression Model for KUS-DP Monitoring Well.

Table 2: Statistical results for KUS-DP monitoring well

Intercept (m.PWD)	β-coefficient	Mean Absolute Deviation (m.PWD)	Mean Squared Deviation (m.PWD)
12.543	- 0.001	1.430	2.674

Table 3: Statistics of observed data and data from the first differencing

Data	Maximum (m.PWD)	Minimum (m.PWD)	Mean (m.PWD)	Variance
Raw	16.051	5.770	12.414	3.578
1st Difference	1.603	-1.120	-0.001	0.092

3.1.2 Detecting Periodicity

Periodogram is been used to characterize the periodic behaviour of the time series. Time series representing GWL typically have an underlying annual periodicity. Building a model of time series, detecting periodicity is a must. A plot of the time series may not always uncover the annual periodicity because time series contain random fluctuations that often mask the underlying structure. The plot of the periodogram shows a sequence of peaks with the lowest frequency peak at a frequency less than 0.02. Each of the data points in the time series represents a week, so an annual periodicity corresponds to a period of 52 in the current data set. As period and frequency are reciprocals of each other, a period of 52 corresponds to a frequency of 1/52 (or 0.0192). Figure 2 shows the periodogram for the given data set. Periodicity 52 has been determined from the periodogram as it is showing the frequency about 0.0189.

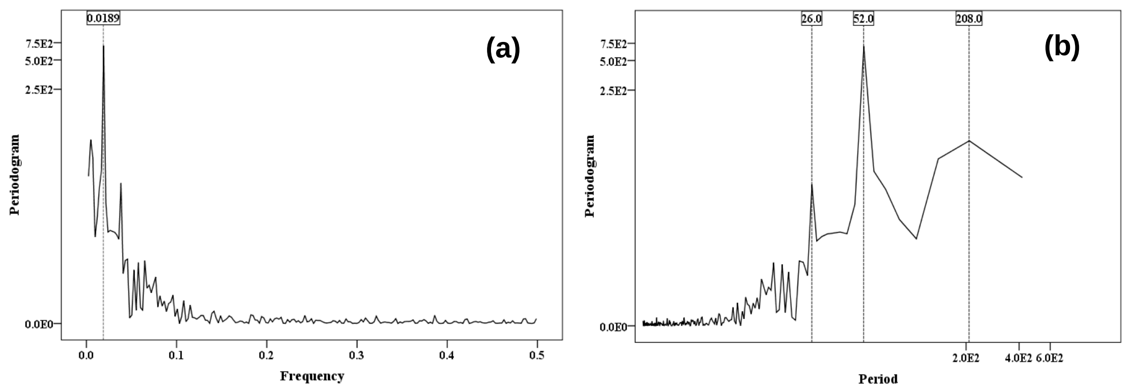

Figure 3: Periodogram of observed GWL for KUS-DP monitoring well (a) by frequency (b) by period

3.1.3 Autocorrelation Function and Partial Autocorrelation Function

Further illustration of the time series is obtained from the estimated autocorrelation function (ACF) and partial autocorrelation function (PACF). As shown in Figure 4, lags up to 160 weeks long are taken. The ACF in Figure 3 dies down slowly in a damped sine-wave pattern, indicating that the raw data is non-stationary. Significant correlations (|t-value|>1.6) exist at the lag 1 and lag 2 phases in Figure 4(a). The ACF exhibits a strong 52 weeks Seasonal component. To remove Seasonal nonstationarity of the series, the first Seasonal differencing of lag 52 is applied (Figure 5), although some significant correlation exists and its magnitude appears to be small as compared the Seasonal differencing. As the trend shows long-term effect, another difference of lag 1 has induced to obtain stationarity and ensured not to be over differenced. A spike at the first Seasonal lag 52 (|t-value|>1.6) appear on both ACF and PACF (Figure 6) indicating that the period of differencing is 52 weeks. The ACF and the PACF both exhibiting large spikes that gradually die out indicates that both autoregressive and moving averages processes are present.

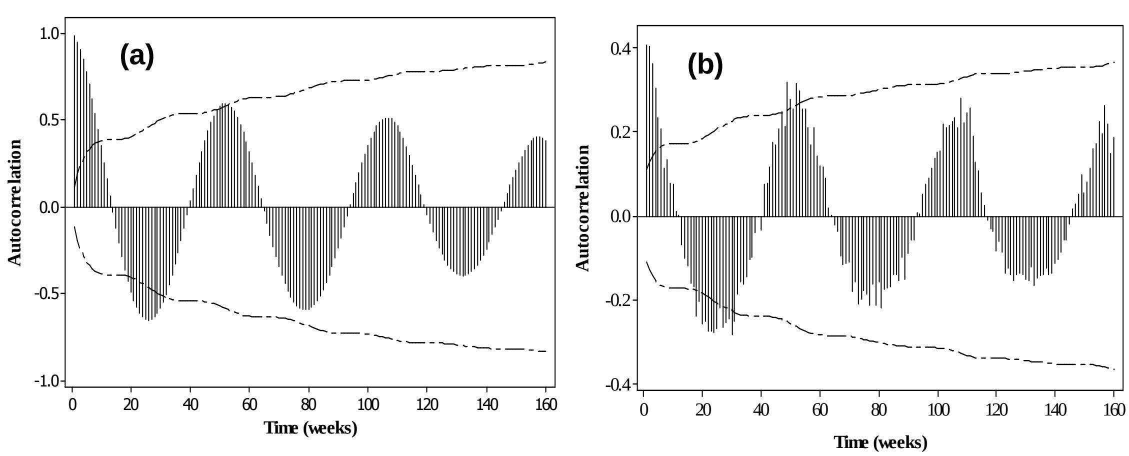

Figure 4**:** The ACF of KUS-DP monitoring well for (a) observed time series and (b) time series from the first differencing of observed time series

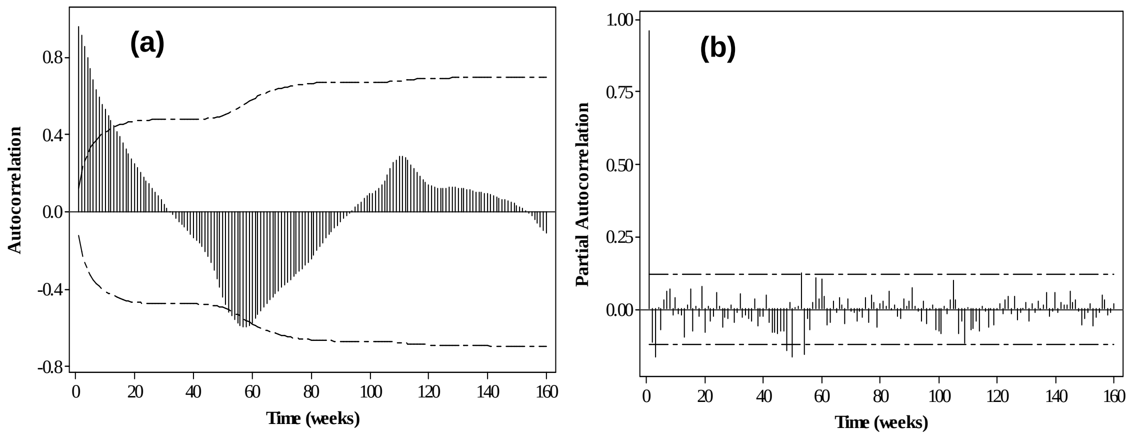

Figure 5: Plot of (a) ACF and (b) PACF for the time series of KUS-DP monitoring well obtained from the first seasonal differencing.

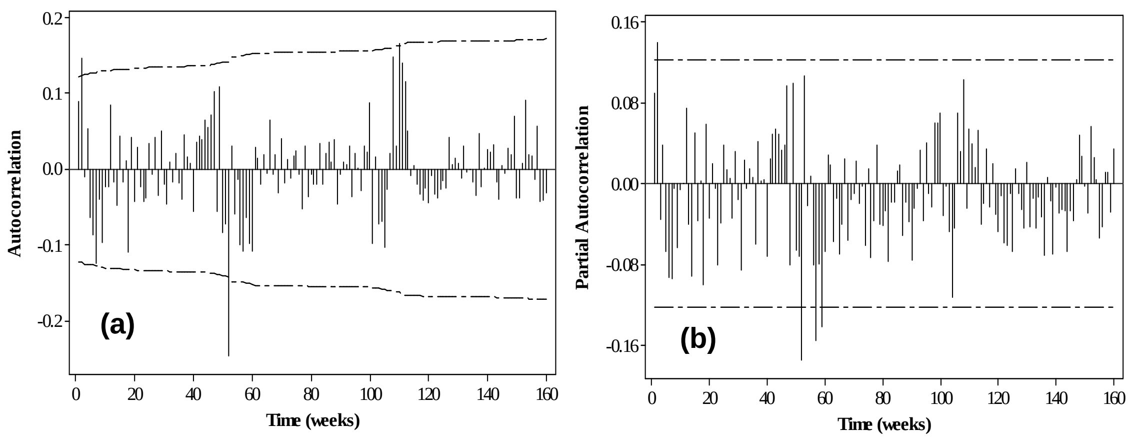

Figure 6: Plot of (a) ACF and (b) PACF for the data of KUS-DP monitoring well obtained from additional first seasonal differencing.

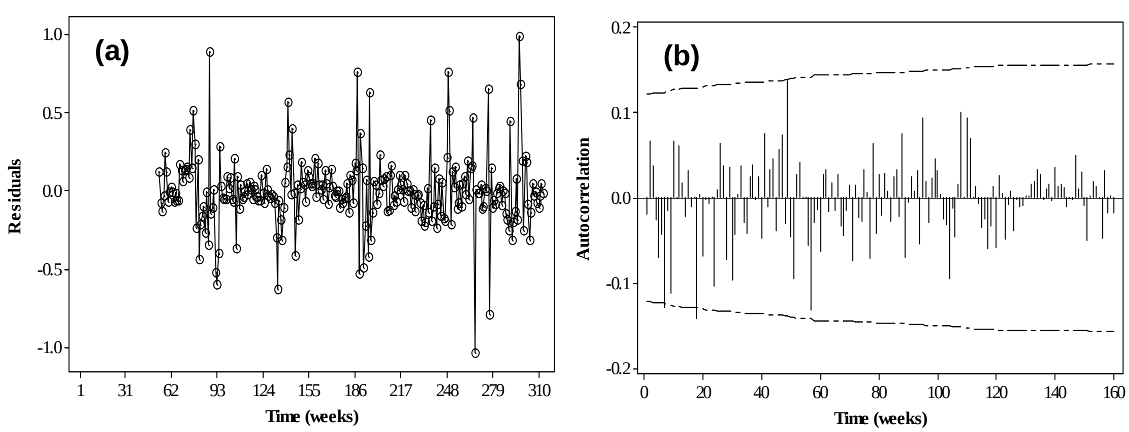

Figure 7: Plot of (a) the residuals and (b) ACF of the residuals of seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$ model for KUS-DP monitoring well.

3.2 Estimation of Model Parameters

The parameters of each model are estimated with the ARIMA module of MINITABv14 statistical software and the results are presented in Table 4. The constant terms of all cases are negligibly small since the modelled differencing series has a nearly zero mean. The good quality of the coefficients are significantly greater than zero (|t-value|>2.0) and satisfy the stationarity conditions. Absolute values for all coefficients are also significantly different from 1.0.

Table 4**:** Estimated Parameters of Seasonal ARIMA Model for KUS-DP Monitoring Well.

Model	Model Parameters	Model Output		
		Value of Parameters	**Standard Error Coefficient**	**t-value**
Seasonal ARIMA $(0,1,1)\times(1,1,0)_{52}$	Φ_1	-0.4399	0.0626	-7.03
	θ_1	-0.0748	0.0623	-1.20
	Constant	-0.0011	0.0186	-0.06
Seasonal ARIMA $(1,1,0)\times(1,1,0)_{52}$	ϕ_1	0.0913	0.0622	1.47
	Φ_1	-0.4397	0.0627	-7.02
	Constant	-0.0010	0.0173	-0.06
Seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$	ϕ_1	0.6631	0.5003	1.33
	Φ_1	-0.0828	0.0816	-1.01
	θ_1	0.6077	0.5286	1.15
	Θ_1	0.8022	0.0734	**10.93**
	Constant	**0.0003**	**0.0020**	0.15
Seasonal ARIMA $(1,1,1)\times(1,1,0)_{52}$	ϕ_1	0.5218	0.4005	1.30
	Φ_1	-0.4394	0.0634	-6.94
	θ_1	0.4164	0.4269	0.98
	Constant	-0.0006	0.0101	-0.05
Seasonal ARIMA $(1,1,0)\times(0,1,1)_{52}$	ϕ_1	0.0407	0.0627	0.65
	Θ_1	0.8185	0.0599	13.67
	Constant	0.0007	0.0049	0.13
Seasonal ARIMA $(0,1,1)\times(0,1,1)_{52}$	θ_1	-0.0356	0.0628	-0.57
	Θ_1	0.8182	0.0598	13.68
	Constant	0.0016	0.0051	0.30
Seasonal ARIMA $(1,1,1)\times(0,1,1)_{52}$	ϕ_1	0.6281	0.5365	1.17
	θ_1	0.5693	0.5657	1.01
	Θ_1	0.8146	0.0608	13.41
	Constant	0.0004	0.0022	0.18
Seasonal ARIMA $(0,1,1)\times(1,1,1)_{52}$	Φ_1	-0.0701	0.0829	-0.85
	θ_1	-0.0303	0.0631	-0.48
	Θ_1	0.7977	0.0772	10.34
	Constant	0.0010	0.0052	0.19
Seasonal ARIMA $(1,1,0)\times(1,1,1)_{52}$	ϕ_1	0.0364	0.0631	0.58
	Φ_1	-0.0412	0.0848	-0.49
	Θ_1	0.8079	0.0781	10.34
	Constant	0.0010	0.0050	0.20

3.3 Diagnostic Checking

The statistical adequacy of the estimated models is then verified. The ACF function for the residuals resulting from a good ARIMA model will have statistically zero autocorrelation coefficients. Figure 6 shows a plot of the residuals for Seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$ model. The residual plot shows small variations around the zero mean. The plot of the estimated residuals and its ACF presented in Figure 7 indicates that there is no significant autocorrelation, and the model adopted will be acceptable.

3.4 Generation of Forecasts

Seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$ model was applied to forecast the water level values from January 2005 to December 2006. The forecasts are then compared with the measured data. The forecasted time series and its 95% confidence level error bound are plotted in Figure 8 for both models. It is observed that all measured values fall within the error bound, and the forecasts track the Seasonal pattern reasonably well.

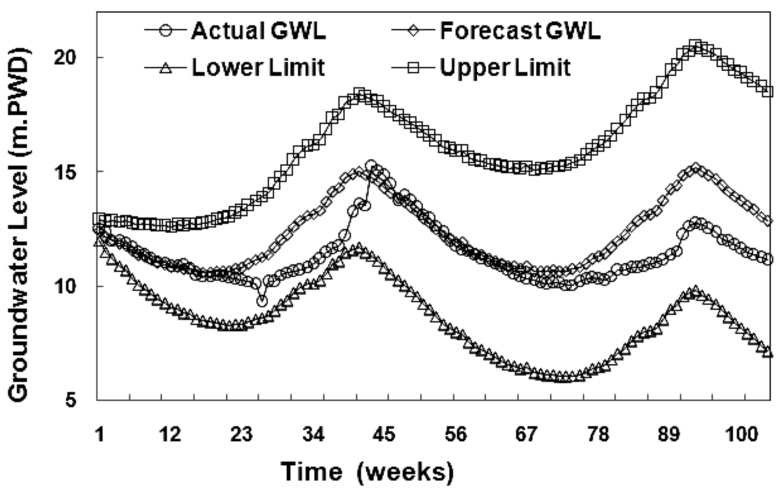

Figure 8: Observed and forecasted values by seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$ model for KUS-DP monitoring well.

4. DECOMPOSITION MODEL

Decomposition models are among the oldest approaches to analyze a time series although a number of theoretical weaknesses exist from the statistical point of view. Decomposition model is then induced to forecast the GWL data using multiplicative type model with a seasonality of 52. A total 104 number of forecasts have been generated in this case using the data from 1999 to 2004 and validated the forecast values with the actual data. Figure 9 shows the sequential plot of groundwater level and forecast values using decomposition model.

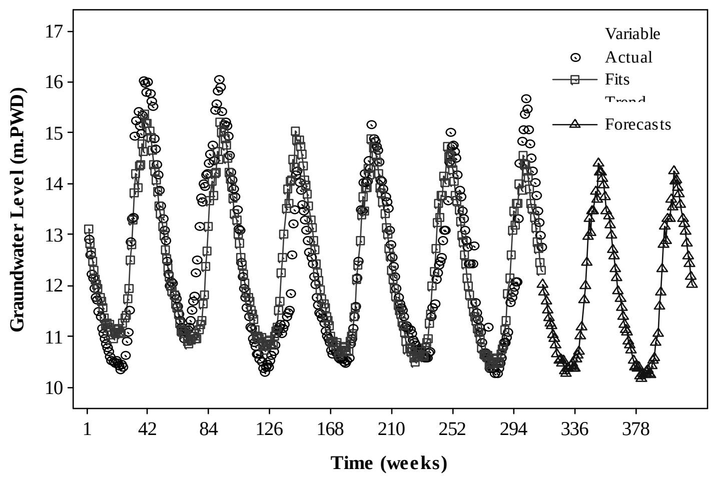

Figure 9: Plot of actual and forecasted values for KUS-DP monitoring well by Decomposition model.

5. MODEL PERFORMANCE

Three accuracy indices such as mean absolute error (MAE), mean square error (MSE), and maximum absolute error (MaxAE) for the nine possible combinations (Table 4) of seasonal ARIMA models and a decomposition model for each observation well are computed and the best combination of seasonal ARIMA model and the corresponding decomposition model results are presented in Table 5.

Table 5**:** Forecasting performance of Seasonal ARIMA and Decomposition models

Well ID	Model Structure	MAE (m)	MSE (m^2)	MaxAE (m)
KUS-DP	Seasonal ARIMA $(1,1,1)\times(1,1,1)_{52}$	0.98	1.88	2.87
	Decomposition Model	1.10	1.92	2.94
KUS-BM	Seasonal ARIMA $(1,1,1)\times(1,1,0)_{52}$	0.54	0.48	1.71
	Decomposition Model	0.46	0.43	1.91
KUS-MP	Seasonal ARIMA $(1,1,1)\times(0,1,1)_{52}$	0.46	0.37	1.79
	Decomposition Model	0.48	0.39	1.92
KUS-KS	Seasonal ARIMA $(1,1,1)\times(0,1,1)_{52}$	0.76	0.92	1.99
	Decomposition Model	0.85	0.99	2.05
KUS-KK	Seasonal ARIMA $(1,1,0)\times(0,1,1)_{52}$	0.67	0.59	1.53
	Decomposition Model	2.57	7.17	4.56

It is observed that Seasonal ARIMA model of almost all monitoring wells shows reasonable forecasting performance than their corresponding decomposition model. The finally selected Seasonal ARIMA stochastic models for five monitoring wells in the study area are developed as equation 2 to equation 6.

(i) KUS-DP monitoring well

$$(1-0.6631B)(1+0.0828B^{52})[(1-B)(1-B^{52})x_i] = 0.0003 + (1-0.6077B)(1-0.8022B^{52})a_i \quad \textbf{(2)}$$

(ii) KUS-BM monitoring well

$$(1-0.8180B)(1+0.4241B^{52})[(1-B)(1-B^{52})x_i] = 0.0002 + (1-0.9470B)a_i \quad \textbf{(3)}$$

(iii) KUS-MP monitoring well

$$(1-0.7476B)[(1-B)(1-B^{52})x_i] = 0.0003 + (1-0.8555B)(1-0.7759B^{52})a_i \quad \textbf{(4)}$$

(iv) KUS-KS monitoring well

$$(1-0.6282B)[(1-B)(1-B^{52})x_i] = 0.0005 + (1-0.9272B)(1-0.8167B^{52})a_i \quad \textbf{(5)}$$

(v) KUS-KK monitoring well

$$(1-0.6631B)[(1-B)(1-B^{52})x_i] = 0.0015 + (1-0.7000B^{52})a_i \quad \textbf{(6)}$$

6. CONCLUSIONS

This paper presents a data conservative approach of modelling the GWL time series to evaluate the groundwater fluctuations in the Kushtia district of Bangladesh. For KUS-DP monitoring well (Table 2 and table 3), regression analysis shows a slope of -0.001, which indicates that it will be lowered up to 1 m in 20 years. In addition, it demonstrates that the area, which contains the KUS-DP monitoring well, should be artificially recharged to maintain the sustainability. Otherwise, the location may face a deficiency of the groundwater availablity. The maximum and minimum value of GWL is 16.055 m.PWD and 5.771 m.PWD, respectively and the difference between the ground surface elevation and the minimum value is about 12 m, which indicates that at least 13 to 14 m deep tube well should be installed to extract groundwater. But the decreasing trend reveals that after 20 years, 13 m deep tube well would be abandoned, if this well is not recharged naturally or artificially. Similar conclution can also be drawn for all the monitoring wells analyzed in the study. The multiplicative combinations of non-seasonal and seasonal ARIMA models have been developed and applied to forecast groundwater fluctuations in all cases. The forecasting performance of the seasonal ARIMA model shows a seasonal trend. The various seasonal ARIMA models forecast weekly data for the evaluation with a

MAE of about 0.46 m to 0.98 m, while the decomposition model gives a MAE of about 0.46 m to 2.57 m. According to the numerical accuracy measures, the study concludes that the seasonal ARIMA model generates better forecasts than that of the decomposition model. However, it can be emphasized that the developed stochastic models for GWL fluctuations can be applied for proper management of groundwater extraction policy in the study area or similar area in Bangladesh.

ACKNOWLEDGEMENTS

The authors wish to express thanks to Bangladesh Water Development Board (BWDB) and Institute of Water Modelling (IWM), Bangladesh for their data supports. Special thanks to Prof. Dr. Quazi Sazzad Hossain, Department of Civil Engineering, Khulna University of Engineering & Technology, for providing the expensive MINITABv14 statistical software for this study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SPATIAL AND TEMPORAL VARIATION OF SURFACE WATER QUALITY OF SUNDARBAN RESERVED FOREST

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ABSTRACT

The Sundarbans, the world largest contiguous mangrove ecosystem, is now in environmental risk due to the discharge of untreated industrial effluents, municipal waste and agricultural waste from the surrounding areas of Sundarban. These wastes ultimately go to the Bay of Bengal through the rivers passing through Sundarban area. However these wastes containing chemical and organic pollutants that make the changes of the water quality of the river in Sundarban. Therefore this study analysed the water quality data of selected areas in Sundarbans and identified changes in terms of time and space. To exhibit the spatial variation of different water quality parameters different maps were generated using ARC VIEW GIS 3.2 software. To express the temporal variation of the water quality parameters different types of line graphs were drawn. This study assess the spatial and temporal variations of the water quality of Sundarban Rreserved Forest (SRF) area by extensive comparative study between the collected data and the existing standard data from Environment Conservation Rules (ECR), USEPA and drinking water quality data from World Health Organization (WHO). In this study, organic pollution was found in rich amount in coastal region (Katka) in dry period and some nutrients followed similar patterns like organic pollution but heavy metals were found high in both north-east and south-east part of Sundarban. Moreover this study investigated the effects of water quality in the SRF ecosystem and provided suggestion to maintain the ecology of SRF area.

Keywords: *SRF, Water quality, Arc View GIS, Spatial and Temporal Variation, Industrial waste, Organic pollution*

1. INTRODUCTION

Sundarban is the world largest contiguous mangrove ecosystem which is known for its spectacular beauty and bio-diversity. It is also a repository of natural resources. Sundarban is located in Bangladesh between latitude 21°30N to 22°30N and longitude 89°0E to 90°0E as shown in Figure 1. The Sundarban Reserved Forest (SRF) represents a significant storehouse of biodiversity which includes 245 genera and 334 species of plants. It is the major habitat for wild shrimp breeding and home to many species of fish, birds, dolphins and other wildlife. It is the most important contiguous habitat in the world for the highly endangered The Royal Bengal Tiger. The Sundarban is intersected by numerous rivers, channels and creeks. These waterways occupy an area of 175000 hectars and are of varying width and length. The rivers of the Sundarbans mainly come from the Ganges- Padma through Garai-Madhumati and from the lower Meghna through the Swarupkati-Kocha river (Akteruzzaman, 2003).

The Sundarban Reserved Forest (SRF) covers 6017 square km in the south-west region of Bangladesh, including three wildlife sanctuary areas on its southern coast which comprise 23% of the SRF. There is a Marine Zone extending 20 km into the Bay of Bengal. The area includes an Impact Zone, comprising 17 adjacent upazillas (in Khulna, Sathkhira, Bagerhat, Pirojpur and Barguna districts) having 4 million people, of whom about 1.2 million are dependent on products extracted from the SRF. The total area of the ecosystem is 6017 square km out of which 4143 square km is land area & 1874 square km is water area comprising river and tidal waterways (Akteruzzaman, 2003).

The SRF is a complex ecosystem but the growth and survival of the ecosystem largely depends upon the circulation of fresh water from the upland. Now a day the ecosystem of Sundarban falls in danger. The ecosystem changes with the changing in water quality. The upstream surface water of the Sundarban is unprotected from discharge of untreated industrial effluents, municipal waste and agricultural waste. The rivers flowing through the Sundarban carry a significant volume of industrial products that contain chemical ingredients. Therefore this may affect the water quality of Sundarbans and as a consequence change the ecosystem.

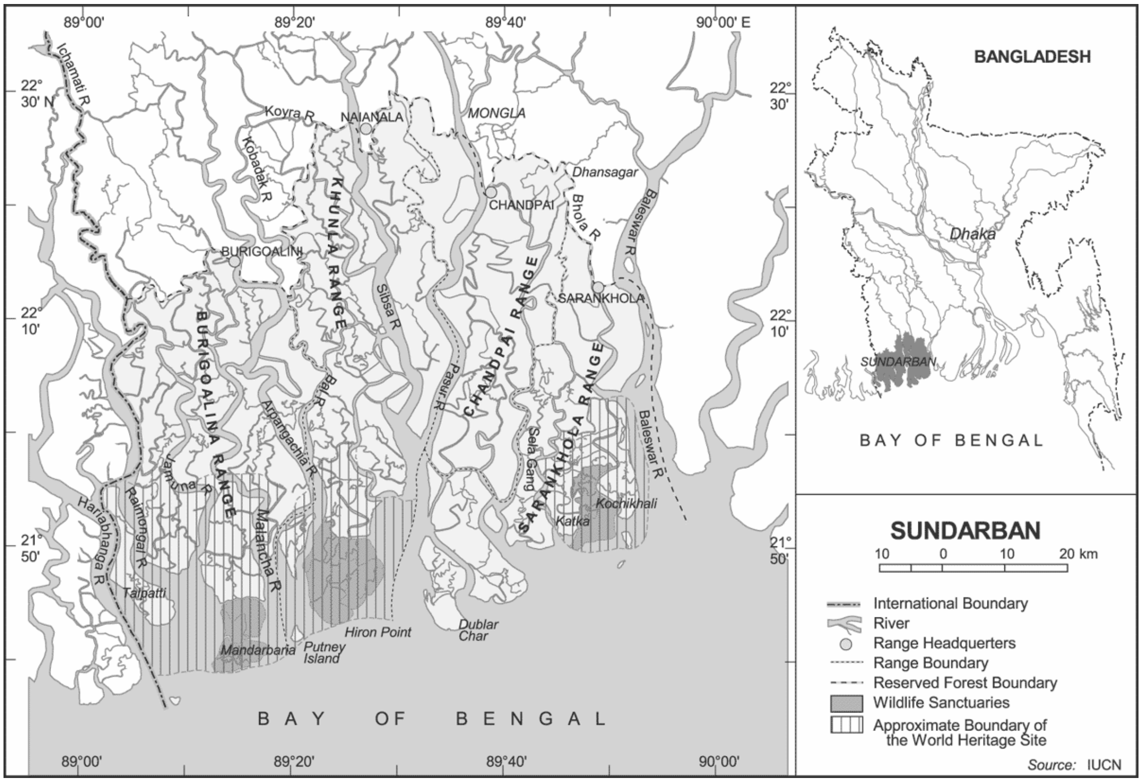

Figure 1: Map of Sundarbans, Bangladesh, Developed by IUCN, Available [Online] at: http://www.ngof.org/wdb/mapsdetail.php?id=8

The main objective of this study is to illustrate the change of water quality with respect to space and time in the Sundarban area. Different water quality parameters were considered and extensive comparative study was conducted considering existing standard values as reference. Moreover this paper investigates the effects of water quality in the SRF ecosystem and provides suggestion to maintain the ecology of SRF area. This paper also establishes a baseline condition of water quality of Sundarban with respect to important water quality parameters like Biochemical Oxygen Demand (BOD_5), Chemical Oxygen Demand (COD), Ammonia (NH_3-N), Nitrate (NO_3-N), Chromium (Cr) and Lead (Pb).

2. METHODOLOGY

Various rivers, channels and creeks are passing through the SRF area which mainly come from the Ganges-Padma through Garai-Madhumati and from the lower Meghna through the Swarupkati-Kocha river. Other rivers Bhairab, Bhadra, Chitra, Khulpetua etc. receive runoff from their own catchment areas. Therefore the SRF is maintaining a complex ecosystem. However, presently the ecosystem of Sundarban falls in danger due to the disposal of Industrial waste, organic pollutants, agricultural waste. These Industrial wastes, organic pollutants change the water quality of SRF area. Therefore this study described the changes of water quality in terms of space and time.

In order to conduct this study, seven locations in the Sundarban area were selected named Mrigamari, Harintana, Dudmukhi, Shawran khola, Katka, Jafa gang and Harbaria. Some water quality parameters for this study were selected that was Biochemical Oxygen Demand (BOD_5), Chemical Oxygen Demand (COD), Ammonia (NH_3-N), Nitrate (NO_3-N), Chromium (Cr) and Lead (Pb). A series of secondary data of water quality parameters from Bangladesh University of Engineering and Technology (BUET) were used. Collected water quality data were divided into three groups such as;

- Organic pollution (BOD_5 and COD),
- Nutrients (Ammonia (NH_3-N) and Nitrate (NO_3-N)) and
- Heavy metals (Lead (Pb) and Chromium (Cr)).

The allowable limit of water quality for marine environment was not available for the study area therefore collected water quality data was compared with the standard values of Environment Conservation Rules (ECR) 1997 and drinking water parameters data from World Health Organization (WHO).

In this study the temporal variation was presented to describe the changes in concentration of water quality parameters with time for each station. To present the temporal variation of different water quality parameter, line graph was used. The highest and lowest concentration of the water quality was also depicted easily from the temporal variation.

The changes of the concentration of water quality with respect to space are called Spatial Variation. To present the spatial variations of different water quality parameters, maps were produced using ARC VIEW GIS 3.2 software. Spatial Variation was divided into two seasons (Dry and Wet season) for each station to compare the effect of the water quality between two seasons. The dry season includes the time period of November to April and wet season include May to October according to the monthly distribution of rainfall in a normal year in Bangladesh (Ahmed and Rahman, 2000).

3. RESULTS AND FINDINGS

The surface water quality of Sundarban area always changes with time and space. Firstly, this study identified the temporal variation where space is fixed but the concentration of the water quality parameter varied with time. Secondly to show the spatial variation, season was fixed for each map but the concentration of water quality parameter was varied with space. The following sub-sections elaborate the result for all selected parameters.

3.1 Temporal Variation

Temporal variation of different water quality parameters in Sundarban area are expressed through line graph. It described into three different sections such as Organic Pollution, Nutrients and Heavy metal.

3.1.1 Organic Pollution

In the Sundarban area, the changes of the organic pollution are expressed through the change in the concentration of Biochemical Oxygen Demand (BOD_5) and Chemical Oxygen Demand (COD). The change in the concentration of BOD_5 and COD of different stations are shown in Figure 2 and Figure 3.

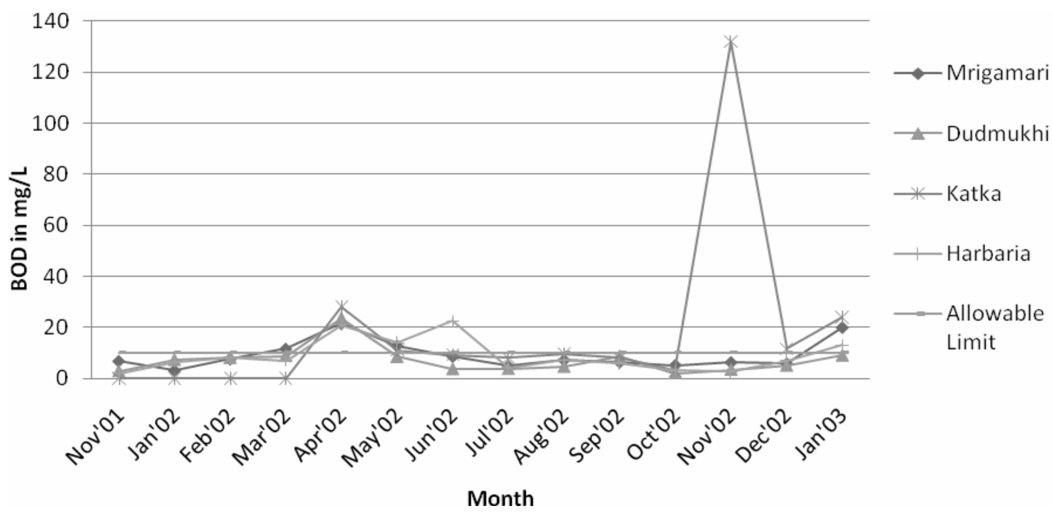

Figure 2: Temporal Variation of BOD_5 in Mrigamari, Dudmukhi, Katka and Harbaria station

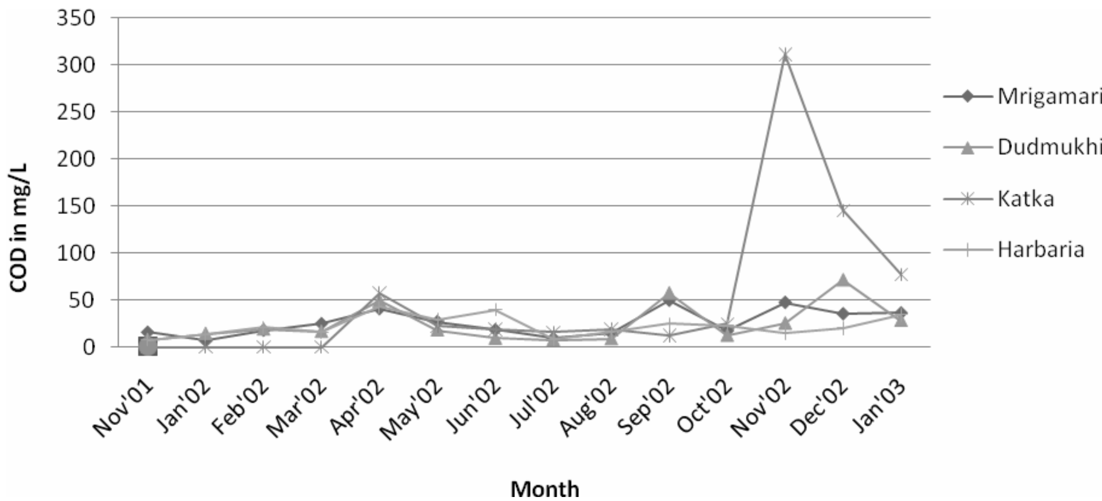

Figure 3: Temporal Variation of COD in Mrigamari, Dudmukhi, Katka and Harbaria station

From the Figures 2 and 3 it was found that in the Sundarban area the changes of concentration of BOD_5 and COD with time period was not constant and did not follow any specific pattern. In the Figure 2, the allowable limit of BOD_5 was shown that was 10 mg/L according to ECR,1997. Although the standard allowable limit of COD was not available, therefore it could not be compared. The range of collected BOD_5 was 2-132 mg/L. So it was found that the value of BOD_5 and COD was remained within the allowable limit in the wet period due to dilution factor. Therefore it was concluded that in wet period, the organic pollution in Sundarban was within the tolerable limit. It can be said that in the dry period the value of BOD_5 exceeded its allowable limit. However the changes of BOD_5 and COD values both followed the same pattern in the selected area and the ratio between BOD_5 and COD was within the allowable range from 1.5 to 2. The maximum BOD_5 and COD value was found at Katka in the month of November 2002.

3.1.2 Nutrients

Ammonia and Nitrate were considered as the Nutrients concentration in the Sundarban area. Figure 4 and Figure 5 show the Nutrients concentration of different stations. Among the Nutrients, the concentration change of ammonia did not follow any specific pattern in the selected stations of Sundarban area.

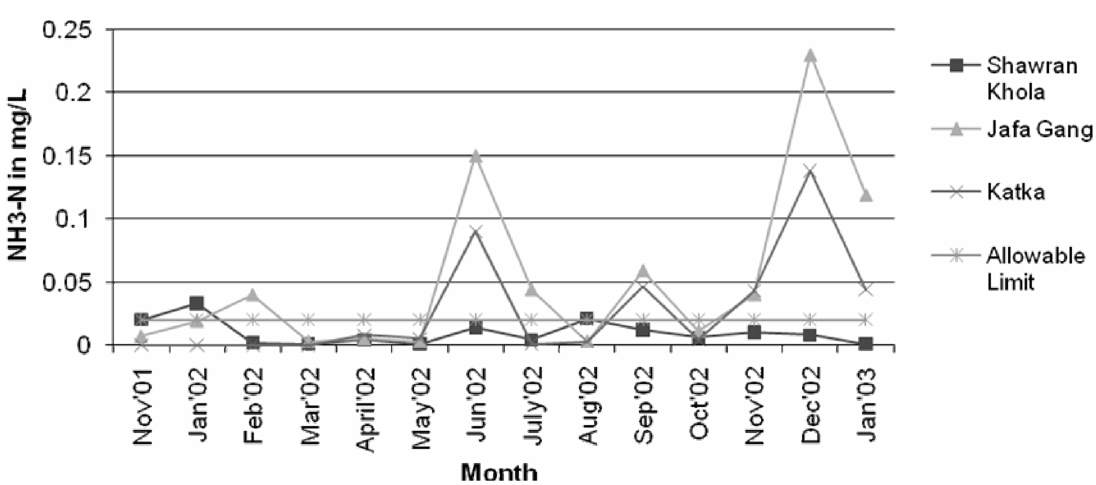

Figure 4: Temporal Variation of Ammonia in Shawran Khola, Jafa Gang and Katka station

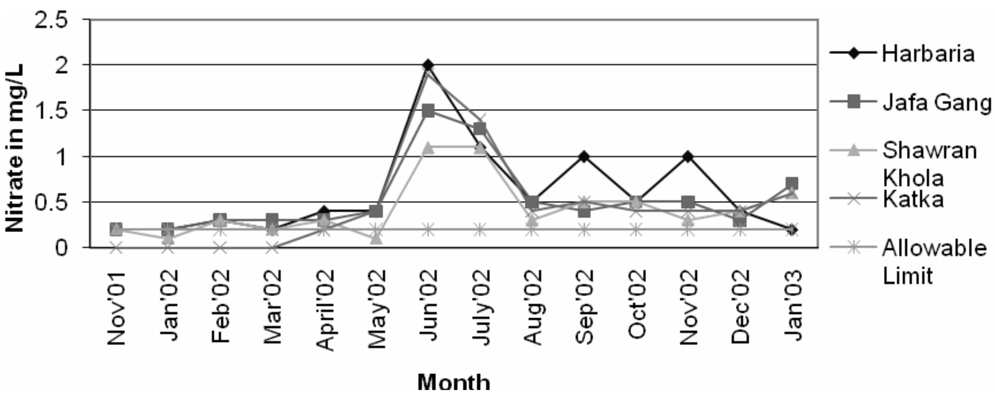

Figure 5: Temporal Variation of Nitrate in Harbaria, Jafa Gang, Katka and Shawran Khola station

Most of the time, it had low value than allowable limit (0.02 ppm (USEPA)) for every location. From the data analysis it was found that ammonia was high at the coastal region Katka, Jafa Gang where BOD_5 and COD was also high. If the over all data change at different locations were considered then it was found that the value of Ammonia increases from upstream to downstream as the value of Ammonia found in Mrigamari 0.05 mg/L, Harbaria 0.09 mg/L and in Jafa Gang 0.15 mg/L in June'02. These type of change of the concentration was also followed by Shawran Khola, Dudmukhi and Harintana.

The concentration of Nitrate of all locations followed a pattern that was first it slightly increased and decreased for a long time period but from June 2002 to July 2002 it rapidly increased for a certain period. Then it decreased again as shown in Figure 5. According to USEPA, the allowable limit of Nitrate is 0.1-0.2 mg/L. The collected data range of Nitrate was 0.1 to 2 mg/L. So most of the time it exceeded its allowable limit. The maximum Nitrate's value was found 2 mg/L at Harbaria in the month of June 2002 and the lowest value was found 0.1 mg/L at many stations at different time period.

3.1.3 Heavy Metals

Chromium and Lead concentration were considered as heavy metals in the selected area of Sundarban. The change in concentration of Lead and Chromium of different locations are shown in Figure 6 and Figure 7.

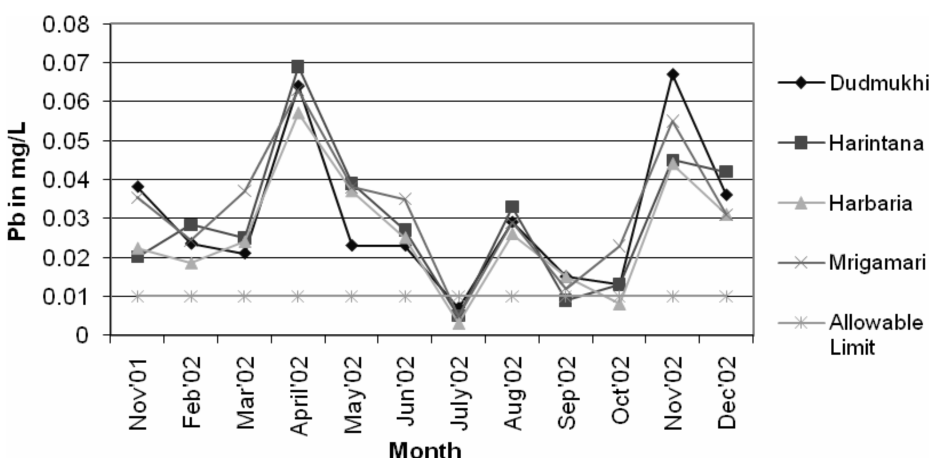

Figure 6: Temporal Variation of Lead in Dudmukhi, Harintana, Harbaria and Mrigamari station

From the Figure 6, it was found that the concentration of Lead had no significant pattern. However maximum value of lead concentration was observed in April for all the stations . The allowable limit of lead is 0.01mg/L according to WHO. So most of the time it exceeded its limit.

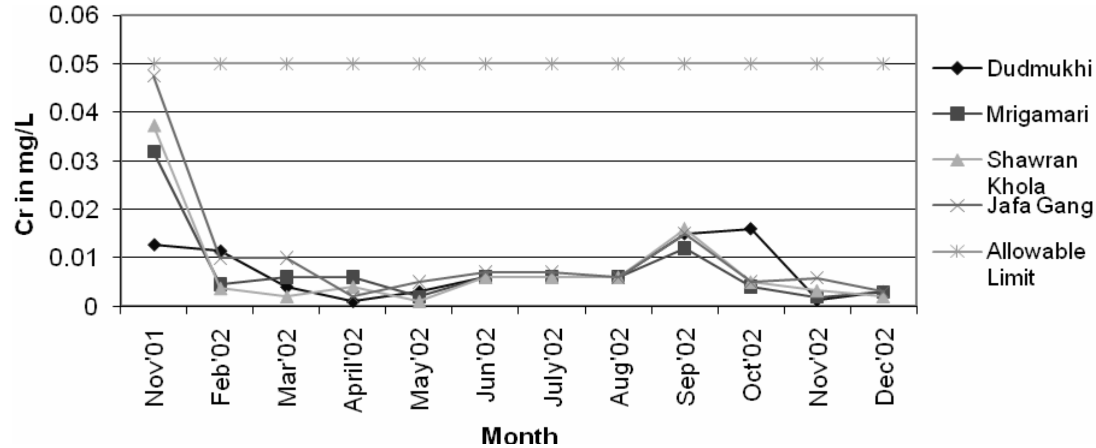

Figure 7: Temporal Variation of Chromium in Dudmukhi, Mrigamari, Shawran Khola and Jafa Gang station

The change in concentration of Chromium also did not follow any significant pattern in the Figure 7. But the concentration of Chromium was always remained within its allowable limit of 0.05 mg/L according to WHO and (Sabhapandit, P. 2010).

3.2 Spatial Variation Of Water Quality Parameter

The changes of water quality with respect to space in Sundarban is the Spatial variation which is shown as maps plotted using GIS software. Here water quality parameters were divided into three groups such as organic pollution, heavy metals and nutrient for seven locations to generate the maps.

3.2.1 Organic Pollution

BOD_5 and COD were considered to describe the changes of water quality due to organic pollution. Table 1 shows the average value of BOD_5 and COD for dry period and wet period of the selected locations. Using these values, the dry period and wet period maps were produced as shown in Figure 8 and Figure 9. It was observed from the dry period map as shown in Figure 8 that the BOD_5 value increased from the north-east to the south-east. The most concentration of BOD_5 was in Katka that was 48.9mg/L and lowest was in Shawran Khola, 5.36 mg/L. During the wet period, it was observed that the BOD_5 values did not follow any pattern as shown in Figure 9.

Table 1: The average value of BOD_5 and COD of different locations of Sundarban

Station Name	River	The average value of BOD_5(mg/L)		The average value of COD (mg/L)	
		Dry Period	Wet Period	Dry Period	Wet Period
Mrigamari	Mrigamari gang	10.34	7.5	27.85	22.47
Harintana	Sela gang	7.94	6.0	27.25	23.995
Dudmukhi	Betmar	8.46	5.13	28.72	18.72
Shawran Khola	Bhola	5.36	4.85	15.75	11.96
Katka	Coastal	48.9	8.15	147.58	18.45
Jafa Gang	Jafa	12.03	9.1	58.65	32.88
Harbaria	Passur	8.39	9.5	21.09	23.35

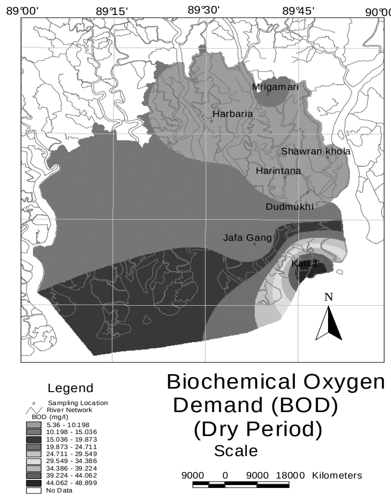

Figure 8: Dry Period Map of BOD_5

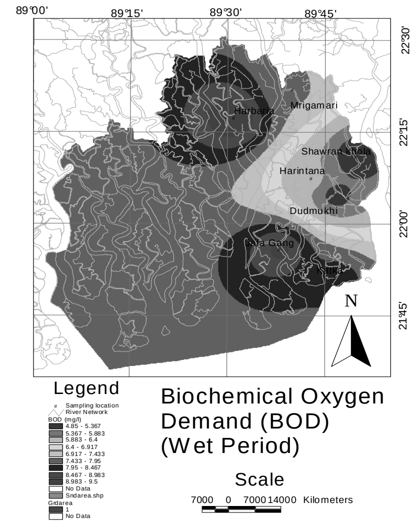

Figure 9: Wet Period Map of BOD_5

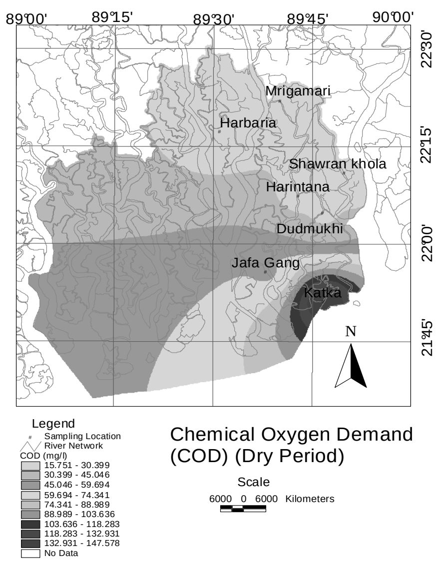

Figure 10: Dry Period Map of COD

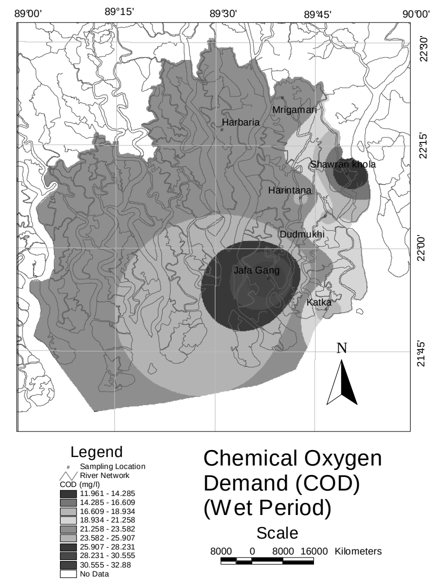

Figure 11: Wet Period Map of COD

It was observed that the COD value increased from the north-east to the south-east area during dry period as shown in Figure 10. The maximum concentration of COD was in Katka that was 147.58 mg/L and lowest was in Shawran Khola that was 15.75 mg/L. During wet period, it was observed that the change of COD did not follow any pattern as shown in Figure 11. The highest concentration of COD was in Jafagang that was 32.88 mg/L and lowest was in Shawran Khola that was 11.96 mg/L. Moreover it was observed from Table 1 that the value of COD or BOD_5 at wet period found less than the dry period due to dilution factor.

3.2.2 Nutrients

Spatial variation of Ammonia and Nitrate were considered to represent the change in concentration of nutrients. The maps for Nutrients were plotted in the similar way as of Organic pollution. From the map it was observed that the concentration of Ammonia during dry period increased from the north-east to the south-east area as shown in Figure 12. The maximum concentration of ammonia was in the south-eastern part of the Sundarban. It was observed that Ammonia changes did not follow any pattern during wet period as shown in Figure 13.

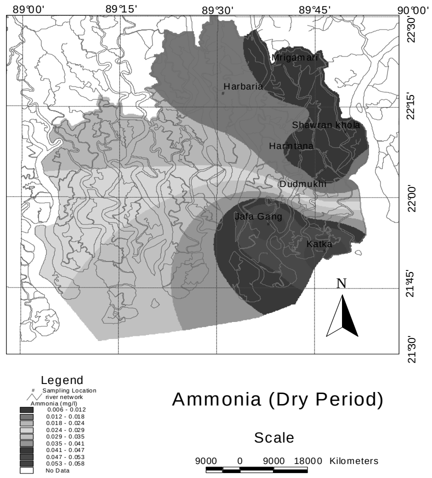

Figure 12: Dry Period Map of Ammonia

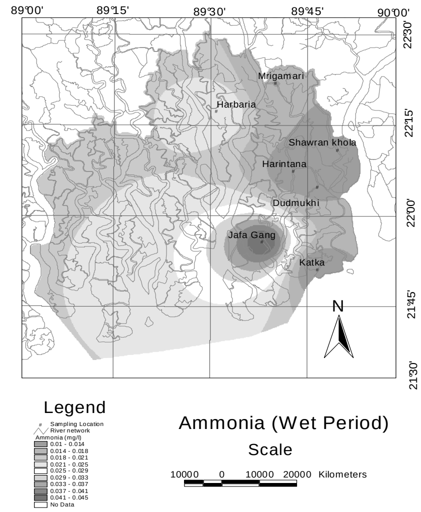

Figure 13: Wet Period Map of Ammonia

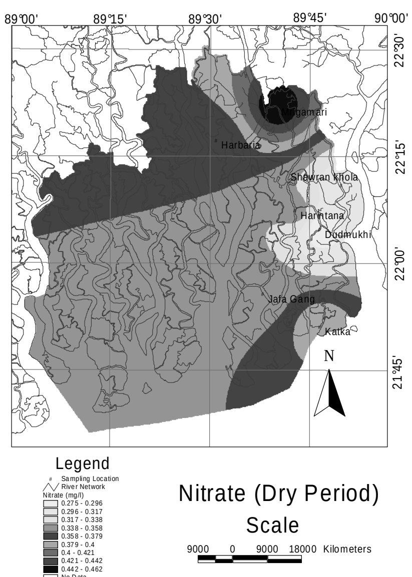

Figure 14: Dry Period Map of Nitrate

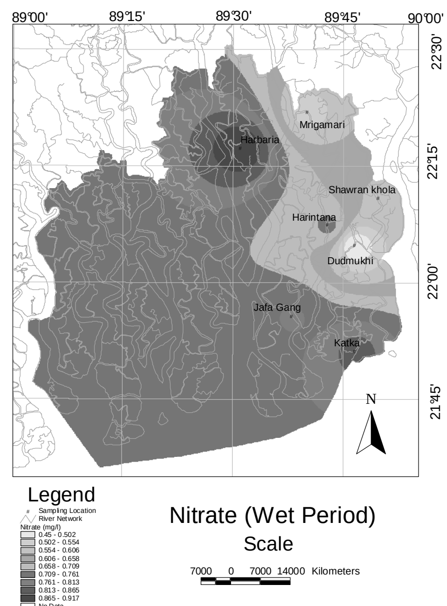

Figure 15: Wet Period Map of Nitrate

The maximum concentration of Ammonia was in Jafagang that was 0.045 mg/L and lowest was in Shawran Khola that was 0.0097 mg/L. The concentration of nitrate during dry period and wet period did not follow any pattern as shown in Figure 14 and Figure 15. It had higher concentration in northern and southern part of Sundarban and lower concentration in eastern part.

3.2.3 Heavy Metals

Chromium and Lead were considered for the representation of Spatial variation of Heavy metals. In the wet period, the concentration of Chromium was less in north-east part but it is constant for all other areas as shown in Figure 17. During dry period, the concentration of Chromium was less in the central part of Sundarban area but higher in both north-east and south-east part of the area as shown in Figure 16. The allowable limit of Chromium is 0.05 mg/L. Therefore it was found that the concentration of Chromium was less than the allowable limit of both dry and wet periods. During dry period, the concentration of lead increased from the north-west to

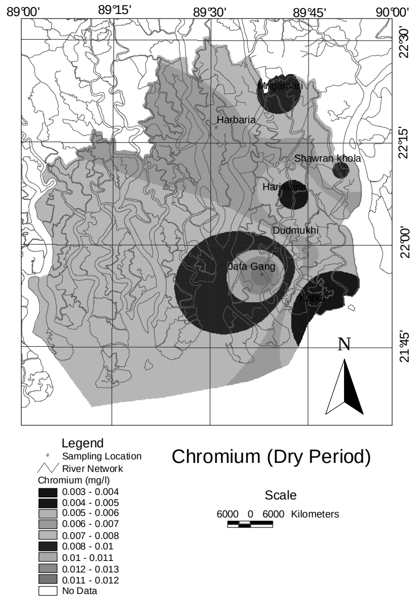

Figure 16: Dry Period Map of Chromium

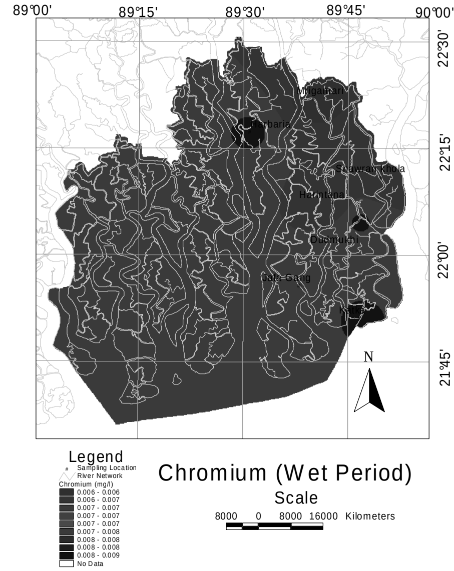

Figure 17: Wet Period Map of Chromium

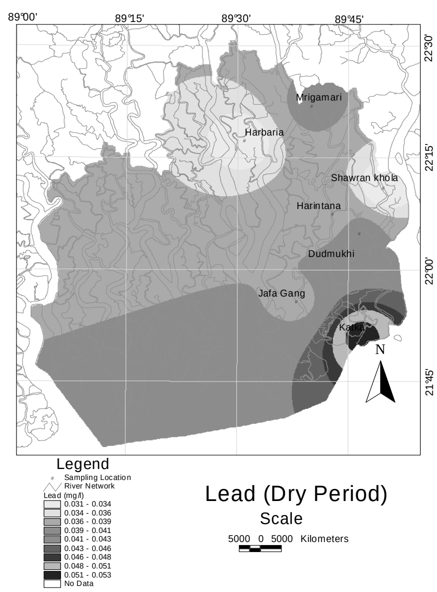

Figure 18: Dry Period Map of Lead

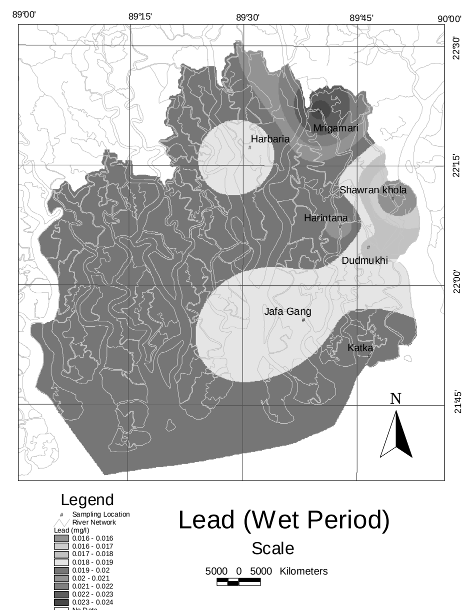

Figure 19: Wet Period Map of Lead

south-east part as shown in Figure 18. However in the wet period, the concentration increased from the east to west part of the area as shown in Figure 19. The concentration of Lead was much in Mrigamari in the wet period.

4. CONCLUSIONS

Sundarban is the most important mangrove forest in the world. Its ecosystem is now in danger due to natural and anthropogenic causes. Seasonal variation of water quality depends on the flow of freshwater, tidal changes and man made reasons. In this study water quality parameters were divided into three groups named organic pollution, nutrients and heavy metals and from the analysis it was found that organic pollution is very high in coastal zone like Katka. There are many industries in the surrounding area of Sundarban especially in Khulna. These industries produce huge industrial wastes that falls into the river system and eventually falls into the Bay of Bengal through the Sundarbans. With the flow of waste into the river system, organic pollutants accumulated with river water and may be one of the reason to the high BOD_5 and COD value at Katka. It was found from this study that the value of BOD_5 exceeded its allowable limit in the dry period. It was also found that during wet period, the organic pollution in Sundarban is still tolerable due to dilution factor.

The Nutrient is an important factor for aquatic life. The analysis result shows that Ammonia and Nitrate were high at the river mouth in Katka. The primary sources of organic nitrates includes human sewage and livestock manure, especially from feedlots and primary inorganic nitrate's like potassium nitrate and ammonium nitrate from agricultural fertilizers. Nitrate does not evaporate therefore it remains until consumed by plants or other organisms. Moreover nitrate is very soluble and do not bind to soils therefore nitrates have high potential to migrate to ground water. Therefore nitrate was found in Sundarban water.

Among heavy metals, Lead and Chromium data was found but lead exceeded its allowable limit of drinking water but Chromium remained within its allowable limit.

From the analysis of Temporal variation it was found that most of the water quality parameters did not follow any specific trend in changing values. However all parameters were in allowable limit in the wet season and in the dry season some water quality parameters exceeded its limit. As the secondary data of very short time period (14 months) was used therefore, it was very difficult to identify the trend with this limited data.

This study was limited by only seven location's available data of the eastern portion of the Sundarban and the simulation was done for the whole Sundarban area therefore it was very difficult to get accurate value through this simulation. However within this generated map it was easily understood that due to the Industrial pollution the water quality of Sundarban were fallen in danger. Therefore it is the right time to be aware to sustain the environment of the Sundarban.

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DELINEATING AQUIFER SYSTEM AND GROUNDWATER RESOURCES ASSESSMENT OF LOHAGARA MUNICIPALITY IN NARAIL DISTRICT, BANGLADESH

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ABSTRACT

The present study is an attempt to investigate existing hydrogeological framework for delineating the underlying aquifer system of Lohagara Municipality under Narail district of Bangladesht for existing groundwater resource assessment. For this purpose, hydrogeological, lithological, hydrological, meteorological and other relevant data have been collected. The IWM customized software "depth-storage" model has been used for producing hydrostratigraphic columner sections, which represent the nature and extent of aquifer system and determining specific yield of sediment formation. From striplogs distribution, it reveals that a productive aquifer exits below the alteration of fine sand aquifer, aquitard and aquiclude layers. The analysis also shows that two major (upper and lower) aquifers exist within the depth from 61m to 95m and from 198m to 216m or beneath throughout the study area. In addition, specific yield of the aquifers varies from 0.07 to 0.20 indicating that the aquifer consists of fine to medium sand. Moreover, a thick aquifer having thickness approximately 30m exists at Isangati and Kamthana Lohagara. However, there is an evidence of prominent aquifer at 200m to 235m throughout the study area containing varying thickness of more or less 36m below the aquiclude. The analysis of storage coefficient and hydrostratigraphic sections shows that the upper aquifer is semi-confined and lower aquifer is confined in nature. The study estimates that groundwater resources for shallow aquifer in Lohagara Pourashava at May 01, 2003 for 6m and 7m depths are 7.48 Mm^3 and 10.00 Mm^3, respectively, whereas at November 01, 2003, it is estimated as 11.29 Mm^3 and 13.81 Mm^3 for 6m and 7m depths, respectively. According to water balance studies, it can be concluded that the groundwater recharge is about 390mm for the deeper aquifer in 2003 and the corresponding annual aquifer storage volume is estimated as 7.25 Mm^3, which mostly comes from the horizontal flow.

Keywords: *Aquifer sustainability, Groundwater modelling, Groundwater resource, Hydrogeological framework, Lohagara Municipality, Narail district*

1. INTRODUCTION

Groundwater (GW) is usually considered as a source of potable water supply in the developing world because it is readily available in nature and naturally protected from the contamination. It is commonly used for irrigation and supplying water for meeting industrial and domestic requirements all over the world (Hoque et al., 2007). Like many other natural resources, GW resource is being exploited at an increasing rate all over the world and thus, GW exploitation is an important issue in the field of modern hydrogeology (Hoque et al., 2007; Balek, 1989). It is evident that the sustainability of existing GW supply structure and supplementary use of surface water depends on the reliable assessment of available GW resources for the area under consideration. For this purpose, an integrated distributed geohydrological model is often required. The model serves as a tool to simulate the recharge, flow and potential head of GW within the defined hydrogeological boundary (Kumar, 2011; Anderson and Woessner, 1992). The numerical model helps in estimating the present available resources and subsequently helps in predicting the potential effect on GW changes under different options of water allocation scenarios. Aquifers are generally very convenient sources of water because they are naturally occurring underground reservoirs, which usually have an enormous storage capacity. Since many aquifers contain high quality waters, their application for drinking water production and human consumption is obvious and they are used as a source of potable water source all over the world. As their storage capacity is quite large, they can provide a continuous supply of water, even in dry seasonal periods when rainfall is very less and/or nearly absent and surface water is fast depleted.

GW is the principal source of drinking and irrigation water supplies in Bangladesh. Until recently, 97% of the population used to rely on GW for drinking purposes while more than 70% of the total irrigated area is served with GW sources (Hasan et al., 2007; BGS and DPHE, 2001). In Bangladesh, GW occurs in the extensive alluvial aquifers at shallow depths and meets most of the water demands for drinking and agricultural uses. More than 10 million shallow hand pumped wells, locally known as hand tube wells (HTWs), are used to provide drinking water to the almost entire rural areas and in urban areas of the country, where there is no piped water supply (Hasan et al., 2007). Development of the GW resource for irrigation and other uses is a vital component of the government's agricultural strategy to achieve food self sufficiency in Bangladesh (Wahid et al., 2007), which is also highlighted in the national water management plan of the country (WARPO, 2001). Thus, there is an urgent need to formulate a strategic plan for appropriate assessment and development of its groundwater resources (GWR) for an aquifer system under consideration. Balek (1989) and Karanth (1987) reported that the component of groundwater study generally contains two major sub-components such as hydrogeological studies and groundwater modelling and assessment. Hydrogeological study has been carried out to understand the regional and local hydrogeological setting, hydrostratigraphical framework, delineation of the underlying aquifer system, status of groundwater level (GWL) fluctuations and groundwater quality (GWQ). The GW model is developed for understanding GW flow dynamics and assessment of existing GWR at the present as well as future development scenarios. In the present study, the hydrogeological setting and available GWR in Lohagara Municipality under Narail district of Bangladesh have been studied. Although Lohagara Municipality is largely dependent on GWR for domestic and irrigation purposes, currently there is no management plan in place and large scale uncontrolled GW exploitation is taking place (IWM, 2011). Lack of appropriate understanding of the GW system in terms of resource utilization, is one of the major limitations to the effective management of this resource. Therefore, the present study has been undertaken to investigate the existing hydrogeological framework and underlying aquifer characteristics of Lohagara Municipality and to quantify the available GWR in relation to natural recharge and extraction options in Lohagara Municipality under Narail district of Bangladesh.

2. MATERIALS AND METHODS

2.1 The Study Area

The study area for the present study, Lohagara Municipality, is situated in Lohagara upazilla (sub-district) under Narail district of Bangladesh (Figure 1). The area is about 16.16 sq. km for the present case study. It is bounded by the Ganges River on the North, by the River Padma on the East and by Western border upazillas on the west. The southern boundary of the study area is defined as the boundaries of the upazillas of Sharsha, Jhikargacha, Jessore Sadar, Narail Sadar, Lohagara, Kashiani and Muksudpur.

2.1.1 Climate

The study area experiences a typical tropical monsoon climate, with hot wet summers from May to September and cool dry winters. The rainy season occurs approximately from May to October and almost 90% of the total annual rainfall occurs during this period. Both temperature and relative humidity remain high in this season. Mean daily temperature is fairly constant between months of April and September and show little variation across the region, being of the order of 28^0C. From October, temperature begins to decline, and mean daily temperature reaches to a minimum of about 19 to 19.5^0C in January. In April, maximum daily temperature in the region can often exceed 35^0C, while in January minimum daily values can be below 10^0C. The rainfall distribution in the study area is not uniform. The lowest mean annual rainfall is about 1800mm, which is observed in northwest stretched strip and increases towards eastward and reaches about 2100mm and 2400mm in Narail and Bhanga, respectively.

2.1.2 Topography and Soil Characteristics

The study area has an almost flat topography and characterised by a fairly plane land except the water bodies such as rivers, ponds, depressions and beels etc. Peat occurs extensively in the Gopalganj-Khulna Beels and locally in some Haors of the Sylhet basin. The soils contain organic matter at the surface or buried under a mineral soil layer below at a depth of up to 40cm. The organic material that forms the Histic horizon varies from dark brown, fibrous peat to semi-liquid black muck. They have been included as Histosols. Soils of this area are result of deposition of Ganges alluvium. Ganges alluvium is calcareous when deposited, but most basin clays and some older ridge soils have been decalcified and acidified in their upper layers; lime is found only in the subsoil or substratum of such soils. Clay soils predominate in basins and on the middle parts of most ridges, with loamy soils (and occasionally sands) occurring mainly on ridge crests. The cut-off parts of the Meghna floodplain have a smooth relief and predominantly silty soils, which are deeply flooded by rainwater in the monsoon season. The unit covers most of the districts of Rajshahi, Natore, Pabna, all districts of Khulna division, and parts of Manikganj, Narayanganj, Munshiganj, Shariatpur, Madaripur, Barisal, Gopalganj. This physiographic unit is almost triangular in shape and bounded by the Ganges tidal floodplain on the south. On its southern end it traps the Gopalganj-Khulna Beels. pH of soil ranges from 7.0 to 8.5.

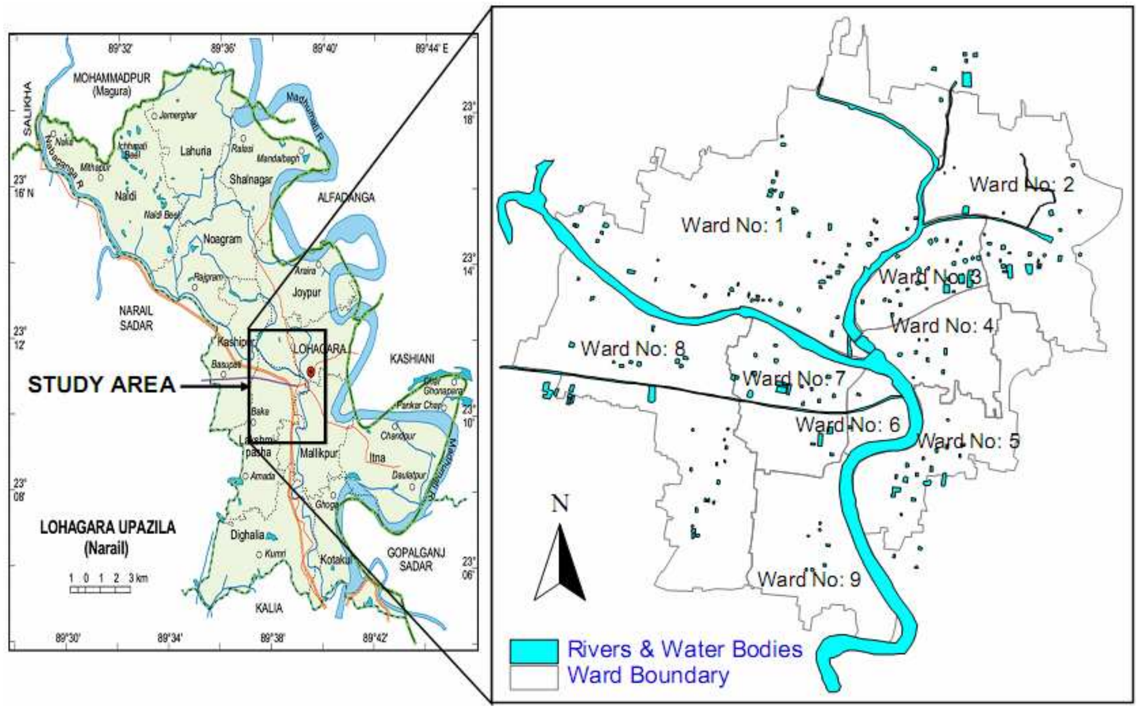

Figure 1: The study area (Lohagara Municipality) in Lohagara Upazilla of Narail district.

2.1.3 Geomorphological and Hydrogeological Setting

Geomorphologically, Lohagara upazilla belongs to Ganges River floodplain which comprises the active floodplain of the Ganges and the adjoining meander floodplain. The latter mainly comprises a smooth landscape of ridges, basins and old channels. The relief is locally irregular alongside the present and former river courses, especially in the west, comprising a rapidly alternating series of linear low ridges and depressions. The Ganges channel is constantly shifting within its active floodplain, eroding and depositing large areas of new char land each flood season, but it is less braided than that of the Brahmaputra-Jamuna. Seasonal flooding is mainly shallow in the west and north, with the highest ridge crests remaining above normal flood levels, but flood depths increase towards the east and the south. Flooding is mainly by accumulated rainwater and the rising GW table except on the active Ganges floodplain and close to distributary channels which cross the meander floodplain. Hydrogeological investigations had been undertaken to assess aquifer information of sedimentary formation. Past studies (BWDB, 1993) give an idea about the depth and thickness of the aquifer layers in the study area. Hydrogeological parameters of this area are governed by the litho-stratigraphic and prevailing tectonic activities, which is part of regional hydrogeological setting and tectonic features. The Quaternary sequence provides good aquifers, which have been extensively exploited in Bangladesh. The aquifers are generally thick multilayered with high transmissivity and storage coefficient. In addition, the aquifer systems can broadly be distinguished in the study area is recent sand forming both confined and semi-confined aquifer.

2.1.4 Existing Agricultural Practices

The major part of the study area is agricultural land. It has homestead, pond and beels also. The crop calendar reveals that T. Aman, HYV-Aman, HYV-Boro, and Potato are the main crops in the study area. Land use and vegetation are used in the model to calculate actual evapo-transpiration depending on the actual crops grown in the project area. Under the present study, spatial distribution of crops has been determined from a comprehensive field campaign. However, for the model input, these cropping types and cropping pattern have further been simplified considering the major crops that require irrigation water. A crop database for each crop, which defines leaf area index, root depth and other properties of each crop are developed based on FAO publications (FAO, 1979) and used in the model. Paddy is grown almost everywhere through the year in each of the Karif-I (March-June), Karif-II (July-October) and Rabi (November-February). Pulse, oilseeds, vegetables, jute, wheat, sugar cane, potato, Mango, etc, are major non-paddy crops. With all availability of irrigation water, growers of the study area shifted towards growing more productive Boro-Fallow-HYV T. Aman.

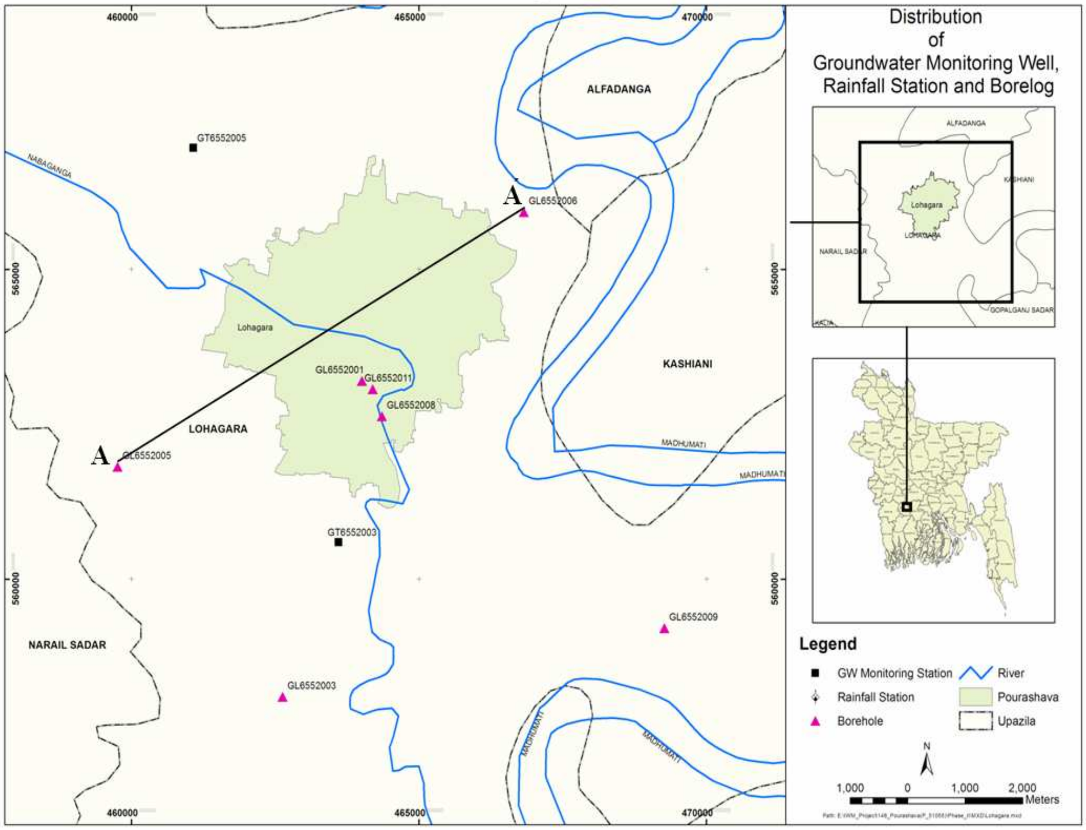

Figure 2: Borelog location and hydrostratigraphic section alignment A-A of Lohagara Pourashava area

2.2 Establishment of Hydrogeological Setting

2.2.1 Topography

A well-prepared Digital Elevation Model (DEM) is essential for visualizing the floodplain topography and for accurate modelling. A DEM of 300m resolution has been developed to define the topography of the study area and used in the model development. Topographic data for the study area has been extracted from the topographic database developed by FAP-19 based on irrigation planning maps available at IWM and also from the topographic survey conducted for Lohagara Municipality by IWM. Roads alignment and homestead coverage also have been collected from Roads and Highway Department (RHD) and LGED. Utilizing these data, a DEM of 300m resolution has been developed to define the topography of the study area. DEM shows that elevation of the area varies from 0.60 to 9.00 m PWD (meters above Public Works Datum of Bangladesh).

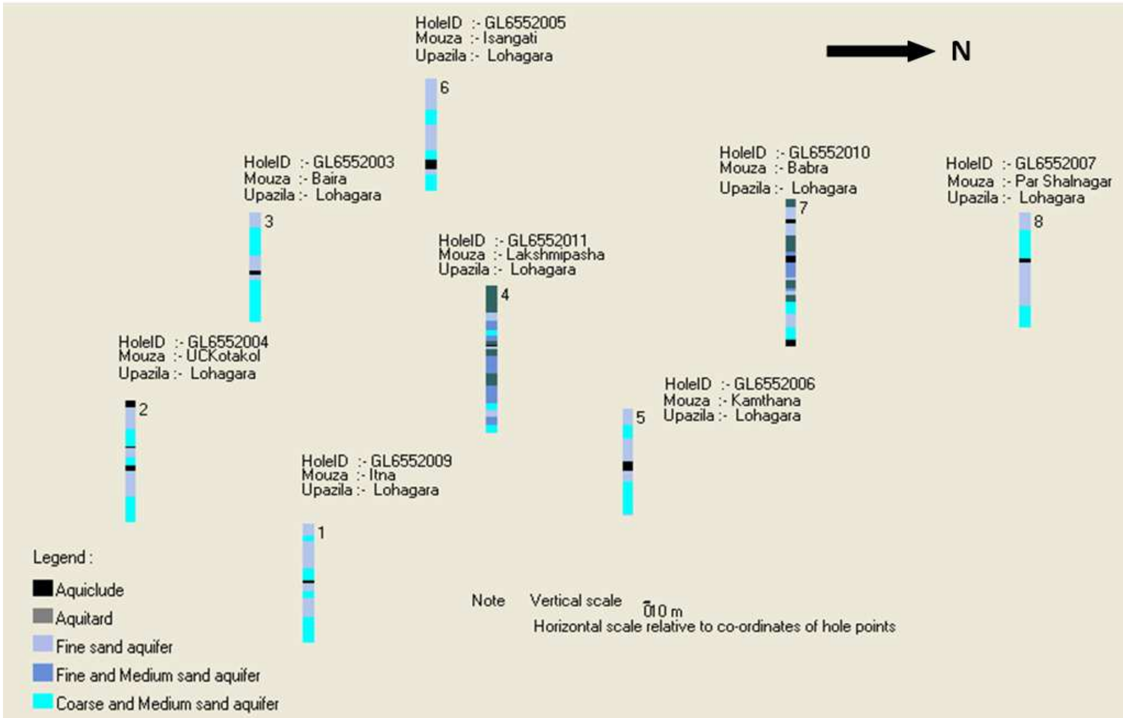

Figure 3: Striplogs and distribution of borelog location in and around Lohagara Municipality

2.2.2 Stratigraphic Analysis and Aquifer Delineation

Hydrogeological investigation for the Lohagara Municipality has been carried out to define the hydrostratigraphic layers. Eight lithological borelogs were collected from DPHE and BWDB for lithological characterization of the sub-surface sedimentary formation, producing hydrostratigraphic cross-sections and for preparing input data of GW model. Among the borelogs of the area, maximum depth of the available borelog is 305m. Sub-surface lithological characterization of the study area and configuration of the hydrostratigraphic units has been prepared by analyzing the individual lithological units and depth of different aquifers. IWM customized software "depth-storage" model has been used for producing the columnar sections and determining the specific yield of sediment formations. Spatial distributions of eight striplogs and their locations are projected all over the study area (Figure 2 and 3). Hydrostratigraphic columnar sections represent the nature and extent of aquifer in and around the municipality area. From striplogs distribution (Figure 3), it reveals that a productive aquifer exits below the alteration of fine sand aquifer, aquitard and aquiclude layers.

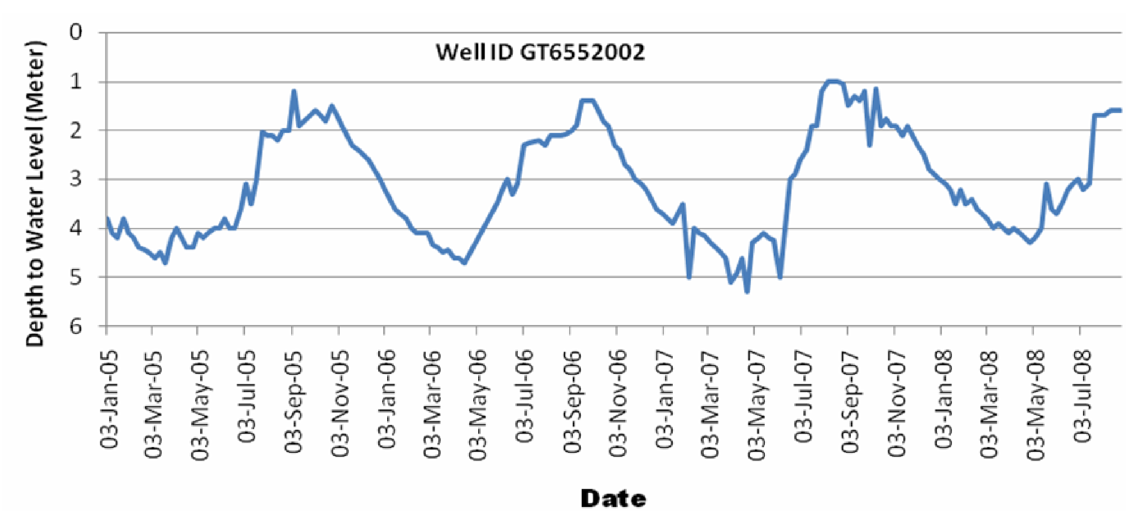

Figure 4: Hydrograph of Lohagara Municipality and its surrounding area

2.2.3 Groundwater Table Fluctuations

Groundwater level (GWL) variation over the period is the primary indicator, which reveals the aquifer response on recharge, abstraction, and aquifer discharge as gravity release. GW hydrograph has been developed from the analysis of the collected data for assessing annual GWL fluctuation. GWL data were collected from the Bangladesh Water Development Board (BWDB) and long term GWL hydrograph (Figure 4) is constructed. It reveals that GWL fluctuates between 2.5 m and 5 m with annual recharge–discharge conditions. From the hydrograph, it is also observed that there are fluctuations in the water level of the study area but having no declining trend.

2.3 Groundwater Modelling and Assessment

For hydrogeological study and GWR assessment, specific emphasis has been given for the municipality area and its vicinity at least the area of Lohagara upazilla. For the GW modelling, a larger study area is generally considered to avoid the boundary influences in model computations (Kumar, 2011; Anderson and Woessner, 1992). Based on these criteria, the developed GW model domain covers an area of about 9,582 sq. km. in this study, which includes Meherpur, Kushtia, Chuadanaga, Jhenaidah, Magura, Rajbari, Faridpur and Part of Gopalganj, Narail & Jessore districts of Bangladesh.

2.3.1 Data Acquisition and Analysis

For the purpose of groundwater source identification and resource assessment, large numbers of hydrogeological and meteorological data have been collected from BWDB, DPHE, BMD. From these secondary data and also from IWM's own data-bank, relevant data to the study area have been used in hydrogeological and GW modelling studies. For hydrogeological study of the Lohagara Municipality and its surrounding area, secondary data has been collected for analysis. Collected secondary data first assessed through necessary quality control procedure and subsequently used for further analysis. There is no aquifer test data is available from secondary source (BWDB, 1997) for the target location of the study area. As such, aquifer properties could not be determined for the Lohagara Municipality and its surrounding area. The model has been developed based on the secondary data collected from the field and also with the data available in IWM. To quantify the resources precisely, the primary data such as geophysical investigation to identify the geological formation and also pump test data to determine the aquifer properties is needed. The model has been calibrated based on secondary data which needs to be validated with the primary data collected in the field.

2.3.2 Model Setup and Simulation Specification

GW model setup involves a geometrical description and specification of physical characteristics of the hydrological system of the study area. In this study, the model has been developed using MIKE-SHE mathematical modelling software tool, developed by DHI Water and Environment Pty Ltd. (DHI, 1999). MIKE-SHE is a comprehensive mathematical modelling system that covers the entire land-based hydrological cycle, simulating surface flow, infiltration, flow through the unsaturated zone (UZ), evapotranspiration and GW flow; it is designed to address dynamic exchange of the water between these components. Major components of the model setup include evapotranspiration, unsaturated zone, saturated zone, overland flow and river systems. The default time step control and computational control parameters for overland flow (OL), UZ and saturated zone (SZ) have been used for entire simulation period. However, simulation periods of the calibration, validation and prediction models were different and user specified. The study area has been discretized into 1 km square grids in the horizontal plan. The model has 9,998 grid cells, where 420 grids are the boundary cells and the rest are computational cells. The grid cells are the basic units to provide all the spatial and temporal data as input and to obtain corresponding data as output. The coupling of surface water (SW) and GW model involves a number of specifications. The river reaches where the coupling will take place have been defined in the river model. In the present study, all the major rivers and khals within the study area have been coupled with GW system. Type of river-aquifer exchanges and the flooding conditions have also been defined. The flow exchange between the SZ component and the river component is mainly dependent on head difference between river and aquifer and properties of riverbed material such as leakage coefficient (Nobi and Das Gupta, 1997). For river-aquifer exchange, leakage coefficients along with the hydraulic conductivity of the SZ are taken into account for most of the river reaches.

2.3.3 Calibration and Validation

The purpose of model calibration is to achieve a satisfactory agreement with the measured data by adjusting the input parameters within acceptable range (Anderson and Woessner, 1992). As a coupled SW-GW model contains huge numbers of input data, the parameters to adjust during the calibration could be numerous. During the calibration, it is therefore important to adjust the parameters within the acceptable range determined from field measurements, and also to minimize the number of adjusted parameters. In this study, the initial input parameters have been obtained from field measurements and other secondary sources. The model has been calibrated for the period of 1997 to 2003. During calibration overland leakage coefficient, vertical hydraulic conductivity, storage coefficient and river leakage coefficient have been adjusted. In the present model, calibration was done against observed GWL and a total of 62 observation wells were used for the calibration and validation purposes. It is customary that the calibrated model should be verified outside the calibration period (Anderson and Woessner, 1992). In order to increase the reliability of the model, it should be validated using another set of data. The period of validation was taken as 2004 to 2007.

2.3.4 Groundwater Resources Assessment

The data analysis suggests that only two geological layers exist within 7m depth. Saturated thicknesses of these two layers have been calculated based on the following considerations:

- Case (a): if thickness of first layer exceeds 6m or 7m depth, entire saturated thickness lies only in first layer.
- Case (b): if thickness of first layer remains above GWL, entire saturated thickness lies only in second layer.
- Case (c): if case (a) & case (b) do not occur, then saturated thickness lies in both first and second layers. To find out the thickness of first layer within the saturated thickness, simply depth of water table is subtracted from the thickness of first layer. Then, part of first layer within the saturated thickness is subtracted from the entire saturated thickness to find out the thickness of second layer within the saturated thickness.

The availability of GWR within the 6m and 7m depths are estimated based on the available saturated thickness up to 6m and 7m depths multiplied by the specific yield (S_Y) of the area under consideration. The volume of the available GWR is calculated using the equation as = area $\times$ Δh $\times$ S_Y, where Δh is the saturated thickness within 6m and 7m depths. By using this procedure, model grid wise GW availability on 2003 has been estimated. Saturated thickness of first and second layers are multiplied by the corresponding specific yield (S_Y) values and summed up to get the depth of water availability of a grid. Availability in volumes is calculated by multiplying the depth of water availability by the area of the grid. Finally, the available GWR is estimated based on the number of grids that lie within the study area.

3. RESULTS AND DISCUSSIONS

3.1 Delineating Aquifer System

The developed columnar section and estimated specific yield of three borelogs are presented in Figure 5, which indicates that fine sand dominated all over the study area from the top most layers varying in depth from place to place. Two prominent aquifers (e.g. upper and lower) are present within the depth from 63m to 90m and from 198m to 216m or more throughout the study area. Specific yield of the aquifers varies from 0.07 to 0.20, which describes that the aquifer consists of fine to medium sand. Hydrostratigraphic section along A-A´ (Figure 2) represents the hydrostratigraphic layers distribution of the Lohagara Municipality and its surrounding area and is presented in Figure 6. It reveals that top most layer is aquitard. Below the aquitard layer, alteration of fine sand dominated and medium sand dominated aquifer is evidenced within different depths and separated from the lower productive aquifer by the clay layer. A thick aquifer having thickness approximately 30m exists at Isangati and Kamthana Lohagara. However, a prominent aquifer is evidenced at 200m to 235m throughout the study area at varying thickness having thickness more or less 36m below the aquiclude. From the analysis of storage coefficient and hydrostratigraphic section, it appears that the upper aquifer is semi-confined and lower aquifer is confined in nature. By analyzing the stratigraphy of the study area, major hydrostratigraphic units are delineated and average thickness of hydrostratigraphic unit are summarized in the Table 1.

Table 1: Summary of hydrostratigraphic units and their extents

Hydrostratigraphic Unit	**Depth (m)**		**Average Thickness (m)**
	from	to	
1st Aquitard	0	30	30
1st Fine Sand Aquifer	31	60	30
1st Aquifer	61	95	36
2nd Fine Sand Aquifer	96	150	55
1st Aquiclude	170	190	21
2nd Aquifer	200	235	36

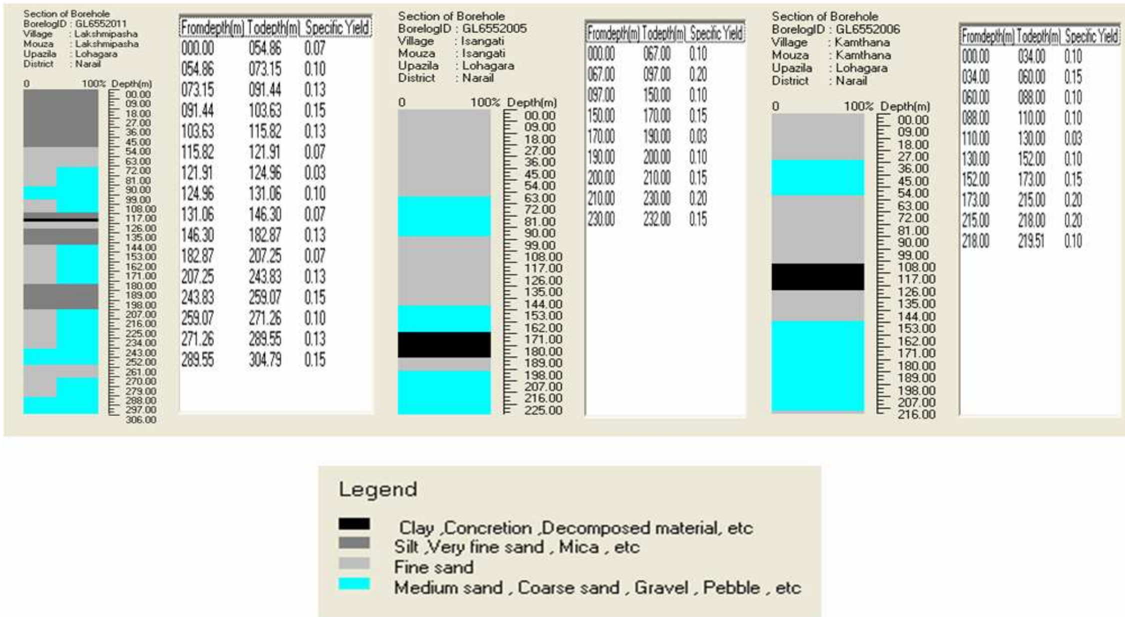

Figure 5: Three columnar sections and specific yield of borelog in and around the Lohagara Municipality

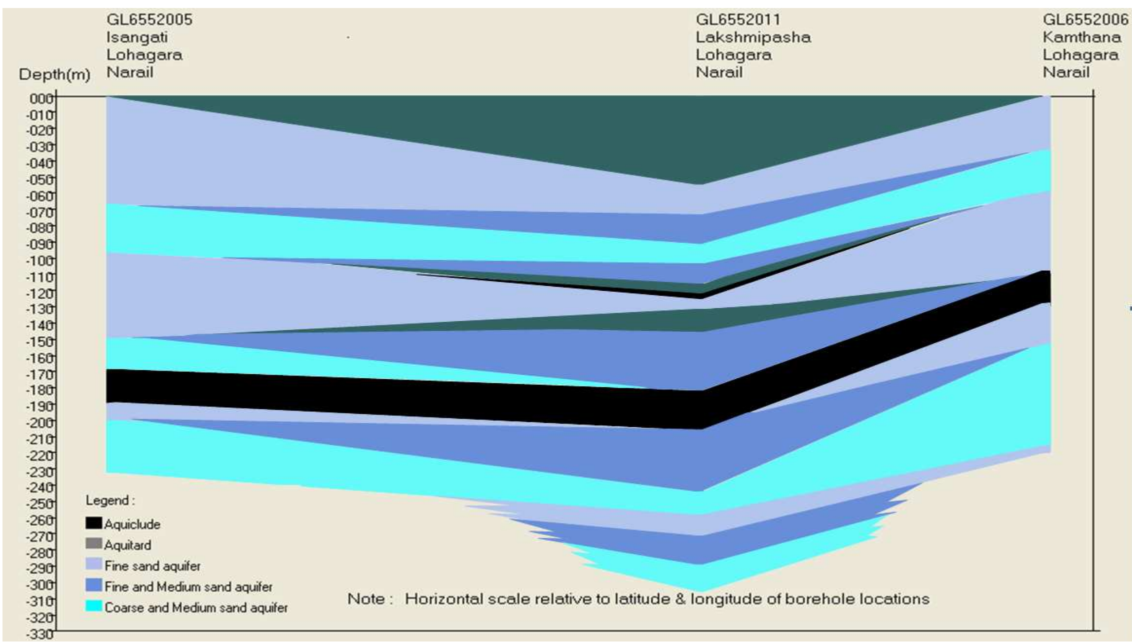

Figure 6: Hydrostratigraphic section along A-A´ of Lohagara Municipality

3.2 Calibration and Validation of GW Model

The developed GW model is calibrated and validated against the observed GWL from a total of 62 monitoring wells in and around the study area. The sample calibration and validation plots in the representative monitoring wells are presented in Figure 7 and Figure 8, respectively. Overall, validation results show similar trend of GWL fluctuations and good matching of GWL between observed and simulated values for both of the validation periods. From the results of the model validation, it could be concluded that the parameters used in the calibrated model are acceptable. Thus, the model can be used for prediction purpose. Hydrographs of simulated groundwater tables (GWT) were obtained at some pre-selected locations, which show that the maximum and minimum depth to GWT occurs at the end of April and end of October respectively. Hydrographs of observed GWT also supports the above findings. Based on these findings, spatial distribution maps of depth to phreatic surface on May 01, 2003 and November 01, 2003 within Lohagara Municipality area (Figure 9). It is observed that during the peak time, GWT almost regains to its original positions.

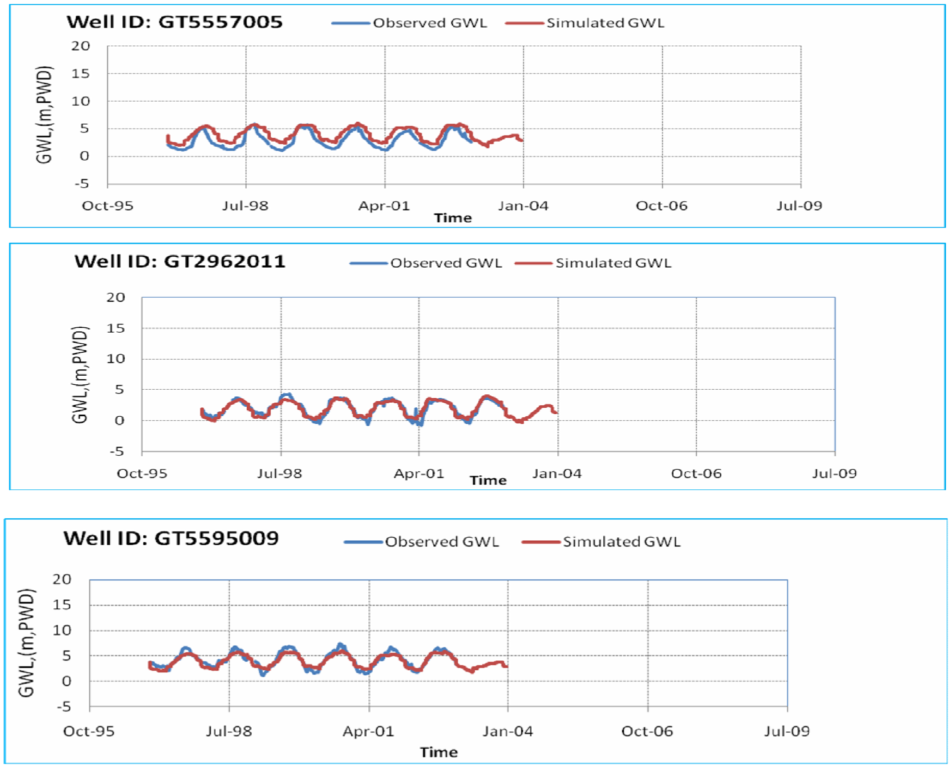

Figure 7: Observed and simulated plots of GWL showing calibration of GW model

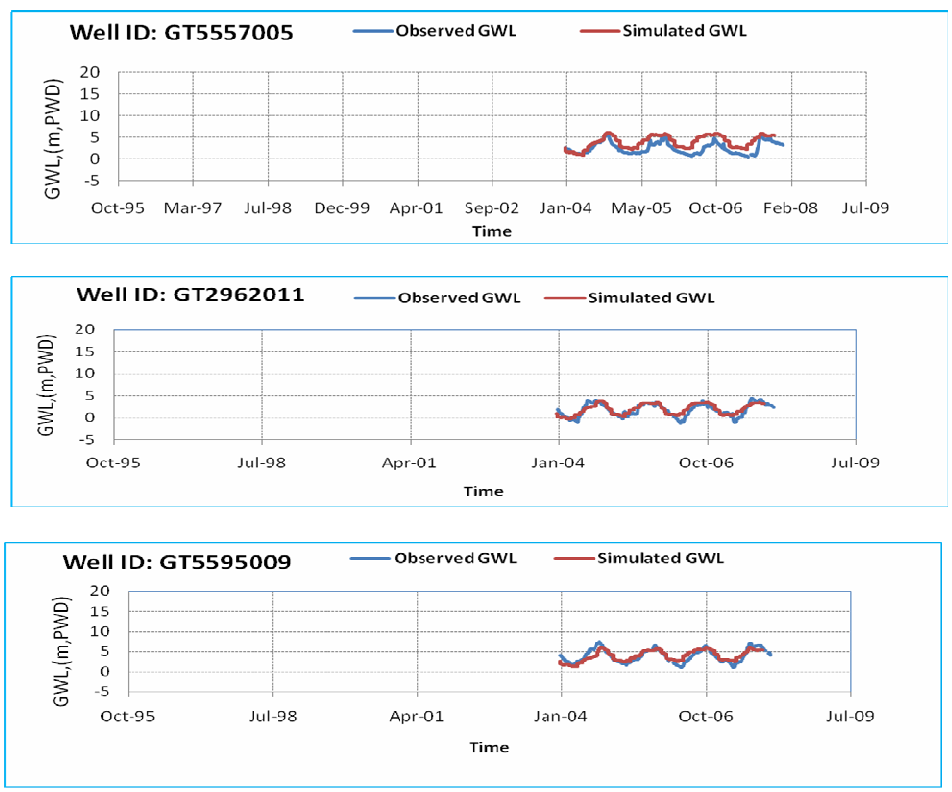

Figure 8: Observed and simulated plots of GWL showing validation of GW model

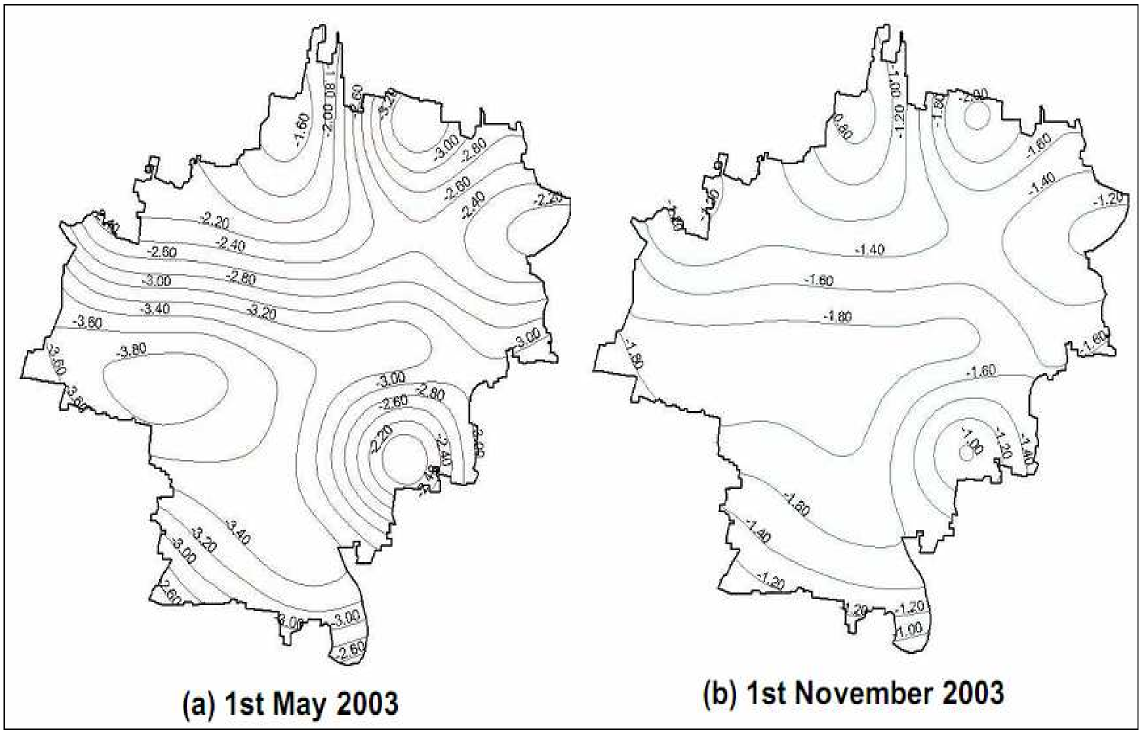

Figure 9: Contour map of depth to phreatic surface for Lohagara Municipality for base condition

3.3 Assessment of Groundwater Resources

3.3.1 Groundwater Resources in Shallow Aquifer

The base condition refers to the design year (year 2003 in this study) hydrological situations and other existing conditions that prevail in the practical field. The main purpose of simulating the base option is to understand the present state of the project under design year in terms of volume of water presently being used, whether it crosses the potential recharge of the aquifer, present state of GWT, crop coverage and scope for further irrigation expansions considering the water availability. Crop coverage under different crops at existing conditions has been considered in base option. Total amount of water required for irrigation is abstracted from the underlying aquifer of the study area. In the study area, GW is being used to meet the irrigation as well as domestic demands. Thus, reliable assessment of GWR is essential for effective water resources management and preservation of environment. Accordingly, GWR of the study area has been estimated based on the available resources before irrigation period.

The availability of GW in Lohagara Municipality of Narail district has been estimated on May 01, 2003 and November 01, 2003 for two different depths of 6m and 7m. It has been estimated that GWR in the shallow depth for Lohagara Municipality at the date of May 01, 2003 for 6m and 7m depths are 7.48 Mm^3 and 10.00 Mm^3, respectively. In addition, the GWR in the shallow aquifer at the date of November 01, 2003 are found as 11.29 Mm^3 for 6m depths and 13.81 Mm^3 for 7m depths.

3.3.2 Groundwater Resources in Deep Aquifer

The GWR in the deeper aquifer is calculated based on the water balance study for the individual layers, since most of the water supply for domestic uses mainly depends on the availability of water resources at the deeper aquifer, A trial and error method is applied to find out the maximum abstraction without any negative changes of storage for the deeper aquifer. The net recharge for the deeper aquifer is estimated based on the final water balance study and presented in Figure 10. However, this assessment is based on the secondary data by assuming that the aquifer properties and boundary data for the deeper aquifer is similar to the upper aquifer. The main limitation of this assessment is that this obtained result need to be verified with the availability of field pump test data, which is not available at hand right now. The net recharge for the deeper aquifer is calculated as net recharge = 0 mm (from upper layer) – 23 mm (for outflow) + 413 mm (for inflow) = 390 mm. This amount of recharge mostly comes from the horizontal flow as presented in Figure 10. For this amount of GW recharge, the corresponding annual aquifer storage volume is estimated as 7.25 Mm^3.

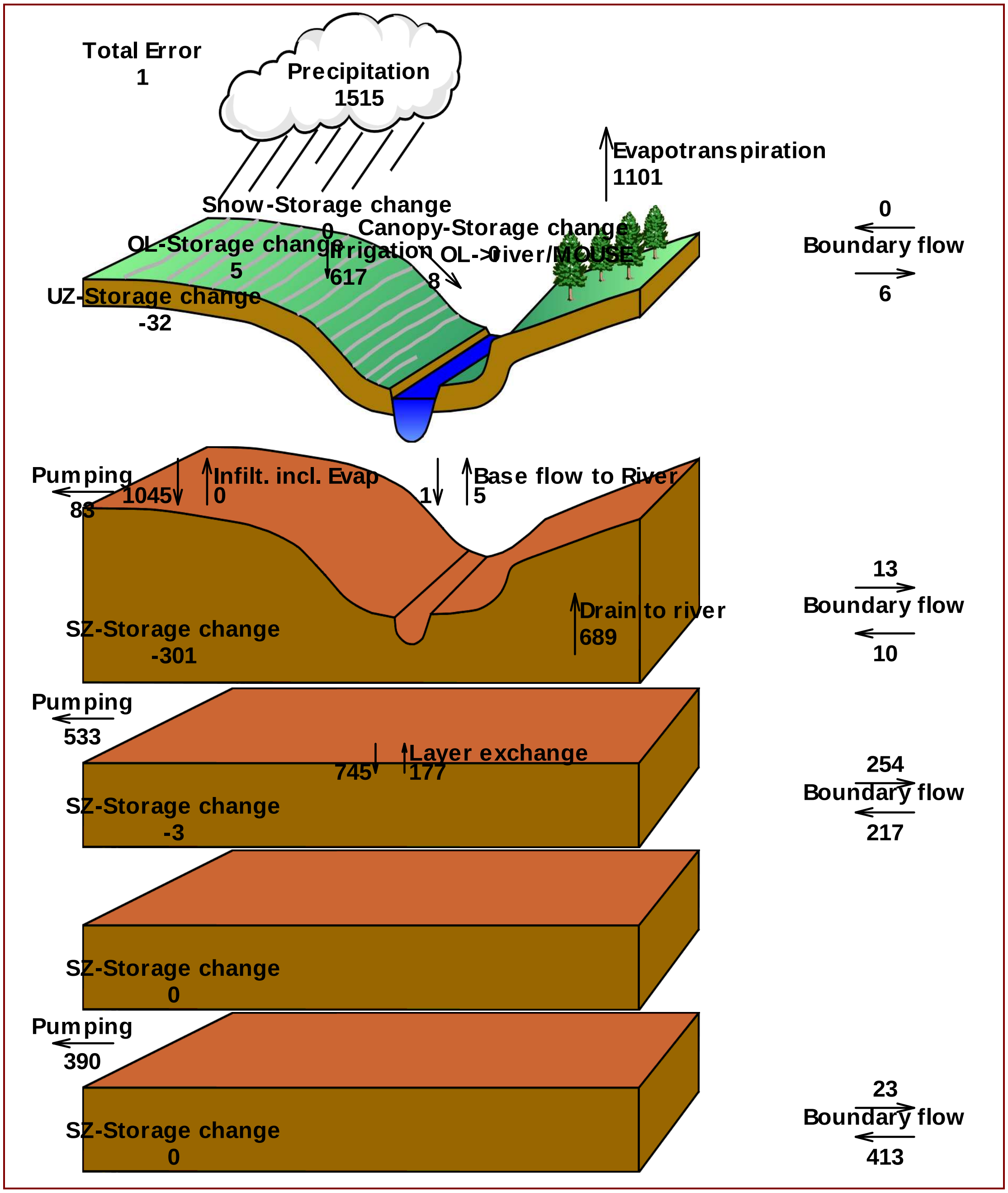

Figure 10: Water balance (in mm) for base condition on 2003 in Lohagara Municipality area

4. CONCLUSIONS AND RECOMMENDATIONS

From the developed hydrographs and water balance studies for different periods in the present study, the following conclusions can be drawn for the Lohagara Municipality of Narail district of Bangladesh:

- It can be concluded from the stratigraphic analysis that two major (upper and lower) aquifers are evidenced within the depth from 61m to 95m and from 198m to 216m or beneath throughout the study area. In addition, a thick aquifer having thickness approximately 30m exists at Isangati and Kamthana Lohagara. However, there exists a prominent aquifer at 200m to 235m throughout the study area containing varying thickness of more or less 36m below the aquiclude.
- The analysis of storage coefficient and hydrostratigraphic sections shows that the upper aquifer is semi-confined and lower aquifer is confined in nature. In addition, estimated specific yield of the aquifers indicates that the aquifer mainly consists of fine to medium sand.

- Maximum depth to GWT occurs at the end of April mainly due to irrigation abstraction and natural drainage which, however, recovers almost to its original position due to natural recharge at the end of September.
- The present study also estimates that GWR in the shallow aquifer for Lohagara Municipality at the date of May 01, 2003 for 6m and 7m depths are 7.48 Mm^3 and 10.00 Mm^3, respectively whereas the GWR at the date of November 01, 2003 found as 11.29 Mm^3 for 6m depths and 13.81 Mm^3 for 7m depths.
- From the water balance studies, it can be concluded that the GW recharge is about 390mm for the year 2003 for the deep aquifer, which comes mostly from the horizontal flow. For this amount of GW recharge, the corresponding aquifer storage volume is found to be 7.25 Mm^3 annually.

For the purpose of aquifer sustainability, following recommendations are suggested:

- Conjunctive use of SW and GW to meet the domestic, irrigation and industrial demand should be promoted and a conjunctive water allocation plan should be established.
- GWT and quality monitoring should be conducted regularly. The production well monitoring should also be done periodically to minimize the operation and maintenance cost of the project as well as to optimize the resource utilization.

ACKNOWLEDGEMENTS

The authors express their thanks to the Water Resources Planning Division of the Institute of Water Modelling (IWM), Dhaka, Bangladesh, Lohagara Municipality and Department of Public Health Engineering (DPHE) for extending all sorts of supports to complete the study. Special thanks to S.M. Mahbubur Rahman, Director of WRP Division, IWM for his continuous encouragements and supports for this study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DEVELOPMENT OF AN INNOVATIVE WATER TREATMENT TECHNIQUE USING SOLAR STILL CUM SAND FILTER

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ABSTRACT

Salinity in water causes many problems in coastal areas of many developing countries. In coastal areas, water sources usually contain many harmful bacteria and dissolved solid which is not safe for drinking. In the areas where potable water is limited and groundwater is highly contaminated with saline, desalination process is necessary in that areas. This study has been carried out to improve the water quality by using tubular solar still along with sand filtration to provide adequate and safe water supply in coastal areas. In this study, solar desalination with a tubular solar still along with sand filter (TSS-SF) was designed and constructed. The laboratory experiment was carried out on the top roof of the Civil Engineering Department of Khulna University of Engineering and Technology (KUET). The distilled water quality and quantity was monitored. Various water quality tests were performed between raw water and treated water and based on the analysis proposal is given for the modification of TSS-SF for the application in coastal regions.

Keywords: *Water treatment, salinity, water purification, low salinity, potable water.*

1. INTRODUCTION

Water is essential for human being. Water demand is increasing day by day due to increasing population. According to World health Organization (WHO), one third of the world's populations are without access of clean drinking water. In 2000, 1.1 billion people worldwide lacked sufficient water resources due to contamination (Sobsey 2002). Although water is one of the abundant resources on earth but only about 3 % of it is potable and remaining 97% is saline water which is lying in the sea. This small percentage of the earth's water, which supplies most of human and animal needs, exists in ice caps, glaciers ground and surface water source (Ahmed and Rahman 2000). According to Oron et al. (2008), 60-90% and according to Food and Agriculture Organization (2006), 69% of the all freshwater sources are used for agricultural irrigation. Groundwater has been used for various purposes such as drinking, domestic purposes, irrigation, industrialization and so on. So, water level is lowering day by day. Surface water is being polluted by various organisms, organic and inorganic compounds. Generally, surface water bodies are more contaminated than groundwater (Burch & Thomas 1998; Wright, Gundry & Conroy 2004), and even though these two systems are interconnected, groundwater is naturally remediated as it flows through underground filtration media (Barker 1998). Groundwater also has less exposure to airborne contaminants as well as animal faeces. Surface water bodies receive runoff from households which is likely to be contaminated (Wright, Gundry & Conroy 2004). Generally, groundwater is clear, colourless with very little or no suspended solids and it is free from disease producing micro-organisms which normally present in surface water. For these reasons groundwater is the main source of water supply in urban and rural areas of many developing countries but in some areas groundwater is contaminated with arsenic, and excessive dissolved iron. Besides, in coastal areas groundwater contains high level of salinity. Solar desalination is the process that removes excess salts and other organic compounds from the water. Solar desalination is the process that removes excess salts and other organic compounds from the water. Though basin type solar still is most popular method in solar desalination, but tubular solar still is easy for construction, operation and maintenance than the basin type solar still. It is the process for water purification where solar energy is used as input energy, raw water is evaporated from the storage channel of solar still and finally accumulated into the outlet. It creates distilled water in undeveloped places that have access only to sea water, brackish or contaminated water.

Among many processes of water purification, filtration is a process in which water is allowed to pass through a bed of filtering media, usually sand and gravel. The major risk to drinking water in developing countries is microbiological contamination (Peter Varbanets et al. 2009). The main forms of contamination are through chemical inputs and pathogens (Brikkle & Bredero 2003). Removing microbiological contamination is especially important for developing countries (Clasen, Nadakatti & Menon 2006). Among various techniques of filtration slow sand filtration or biological filtration is the process where raw water passes through a bed of sand. Slow sand filtration is a suitable method for water treatment in developing countries because it does not require any complex electrical and mechanical equipment or coagulating chemicals. The major advantages of slow sand filter are it has very high removal turbidity, color and bacteria; cleaning of filter bed by scraping and removal of a top layer of sand and low cost of operation and maintenance. It reduces the number of micro-organisms and other physicochemical compounds present in the water. The fundamental requirements of drinking water are that it should be free from disease producing pathogenic micro-organisms; contain no elements or compound in concentrations that can cause acute or long-term adverse effect on human health; be fairly clear and aesthetically attractive (i.e. low turbidity and color); contain no compounds that can cause an offensive taste and odor; not cause corrosion, scale formation discoloration or staining. There has been a belief that if the technology is simple, low-cost and run by the local community; then it will be effective in promoting a better quality of life (La Frenierre 2008). So, the TSS-SF unit could be a good source of producing potable water because of simple construction, locally available materials and semi-skilled or unskilled operators for operations and maintenance. The main objective of this study is to develop a novel water treatment technique using solar still cum sand filter which would increase the production of treated water and retain the salinity within a permissible level.

2. METHODOLOGY

A low cost Tubular Solar Still cum Sand Filter (TSS-SF) was designed and constructed. It was constructed of tubular frame covered with a transparent normal polythene paper and a black rectangular tray used as channel for storing saline water along with sand filter. The sand filter consists of a circular filter box and an outlet. The filtration materials were clean sand free from clay, gravel and charcoal. The influent raw water quality in both TSS and SF was remained same. The effluent treated waters from these treatment operations were blended together and stored in a container. The schematic diagram of the TSS-SF unit is shown below:

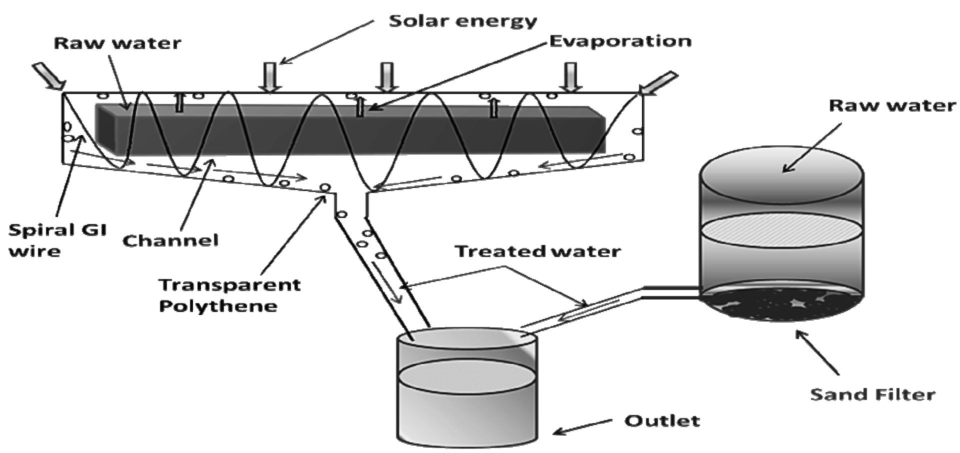

Figure 1: Schematic diagram of TSS-SF unit

2.1 Construction of channel

A black rectangular tray of length 4 feet, width 10 inch and height 6 inch is used as a channel which is made by cartoon paper wrapping black polythene and it is reinforced by 18 no GI wire inside the polythene.

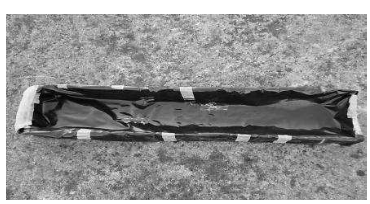

Figure 2: Black rectangular tray

2.2 Construction of tubular solar still

A circular tubular frame of 12 inch diameter was constructed by using 10 no GI wire & a tray was used as a channel inside it. It was supported by a wooden stand of 4ft length, 10 inch width and some bricks around the bottom of the stand also with a simple outlet.

Figure 3: Tubular Solar Still (TSS)

2.3 Construction of sand filter

A circular slow sand filter was constructed using 8cm thickness of gravel and charcoal layer, thickness of sand layer (sieve 100#, 50#, 30# retain, 16# pasing) 55cm, diameter of sand filter 16cm and a simple outlet pipe. The surface area of sand filter 200 sq cm. The schematic diagram of the sand filter is shown below:

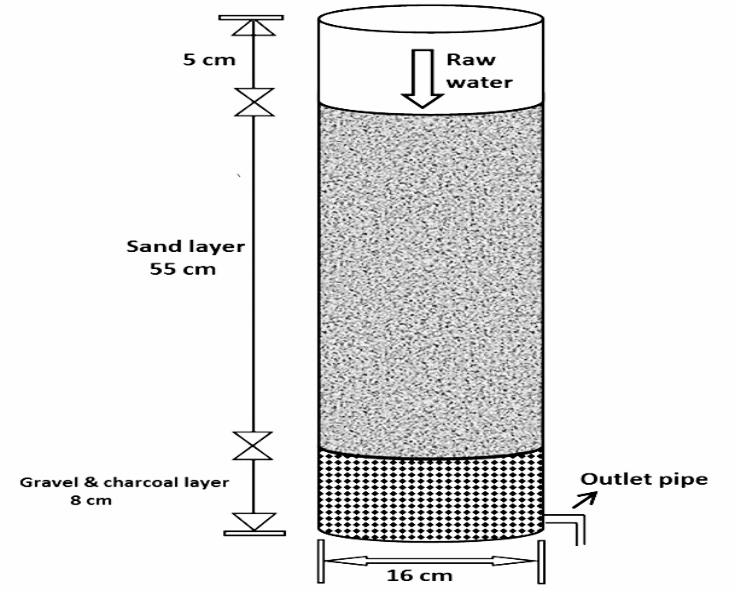

Figure 4: schematic diagram of the sand filter

3. RESULTS AND DISCUSSION

At the beginning of the treatment process, a sample of 15 litres water was taken into the solar still and made its salinity artificially about 1800 mg/L by adding sodium chloride. After that, the same quality of 2 litres water was taken into the sand filter in each day. When the influent raw water of TSS was finished, then it was again filled up. The output of treated water from the TSS-SF unit was collected between 7th November to 4th December, 2011. Each seven days total output from the period 7th November to 4th December, 2011 is shown in the table 1 below:

Table 1: Treated water from TSS-SF unit

Observation Period (Week)	Quantity of Treated Water (L)	
	TSS	TSS-SF
Nov (07 ~ 13): 1st	24	38
Nov (14 ~ 20): 2nd	28	42
Nov (21 ~ 27): 3rd	29	43
Nov 28 ~ Dec 04: 4th	30	44

The quantity of distilled water from TSS was collected everyday. There was variation of productivity because the solar radiation was not same in each day. The quantity of distilled water from the TSS depends upon the intensity of solar radiation. In tubular solar still, many types of heat transfer occur inside the tubular cover and outside such as evaporative heat transfer from the saline water to the tubular cover; condensate heat transfer from the saline water to the tubular cover, the tray and the water, the tray and the tubular cover, the cover and the atmosphere; radioactive heat transfer between the water surface and the tubular cover, the tubular cover and the atmosphere. The evaporative water is transferred to the tubular cover and then finally condensed on the tubular cover inner surface, releasing its latent heat of vaporization. It was observed that solar radiation increased gradually from the morning 10:00 to noon 15:00 and decreased in the afternoon so the evaporation rate was higher in that time. The opening of TSS was closed very tightly so that the inner temperature was not interacted with the environmental temperature. The storage channel was placed properly on the wooden stand. In tubular solar still moisture rises, condenses on the cover and runs down into a collection outlet, leaving behind the salts, minerals and most other impurities including germs and bacteria. So, the storage tray and tubular cover was washed when the storage channel was emptied. Sand filter was cleaned when the fine sand became clogged. The operational process was performed during the winter season. As 15 litres of water was taken into the storage channel of solar still but its maximum capacity is 20 litres. If the experiment would be done during the dry season and make the best use of storage channel, the productivity will be found very higher.

According to Molla and Biswas (2009), the average daily production rate of distilled water from TSS was 3.54 L/m^2. According to Bokshi and Khatun (2010), the average daily production rate of distilled water from TSS was 2.38 L/m^2. According to Saha (2011), it was 3.04 L/m^2 and according to Rab (2011), it was 1.89 L/m^2 and the average daily production was 0.302 L. According to Raihan (2011), the average daily production rate of distilled water from the inclined basin type solar still (BSS) was 3.78 L/m^2 and the average daily production was 1.85 L. But in this above TSS, the average daily production rate was 12.9 L/m^2 and average daily production was 4 L. According to Rais (2012), flow rate of slow sand filter was 0.55 m^3 per m^2 per hr. Flow rate of this above sand filter was 0.71 m^3 per m^2 per hr. There was great difference in the production rate because it depends on the design and maintenance.

According to Bangladesh standards, water requirement for drinking purposes in rural areas is 2~3 lpcd. In the TSS-SF unit, the average production is 6 L/day. In many areas of many developing countries where potable water sources are very limited. Carrying of water is very difficult, occasionally requires high transportation cost and is greatly affected on water consumption. So, people are becoming influenced to use tube-well, rivers, ponds and lakes water for drinking purposes. As a result, they are suffering from various water borne diseases like arsenicosis, dysentery, diarrhea, cholera etc. In the flood season, these diseases spread very rapidly. So, the construction of TSS-SF unit will meet the demand of water consumption in these areas.

The salinity value of treated water stored into outlet from TSS was found to be around 180 mg/L and from TSS-SF, it was found around to be 450 mg/L; though the salinity of influent raw water was around to be 1800 mg/L.

The variation of salinity among influent raw water, effluent water of TSS and TSS-SF is shown in the figure 5 below:

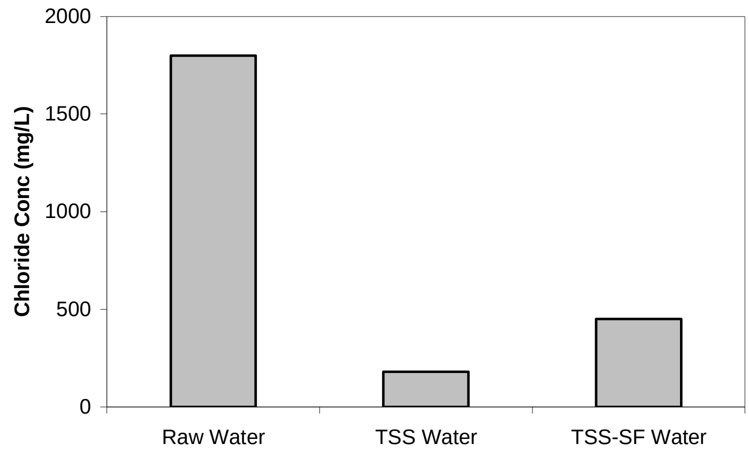

Figure 5: Variation of salinity among influent raw, effluent water of TSS and TSS-SF

According to World Health Organization standards (WHO 1993), the salinity value of drinking water is 250 mg/L. According to Bangladesh standards (BDS 1997), the salinity value of drinking water is 150 to 600 mg/L and in coastal areas of Bangladesh the chloride content up to 1000 mg/L is acceptable. So, the salinity of treated water of TSS-SF unit is within the permissible limit.

The total dissolved solids (TDS) value of influent raw water was 3000 mg/L. The total dissolved solids (TDS) value of treated waer from TSS and TSS-SF was found to be 350 mg/L and 800 mg/L. The variation of TDS value among influent raw water, effluent water of TSS and TSS-SF is shown in the figure 6 below:

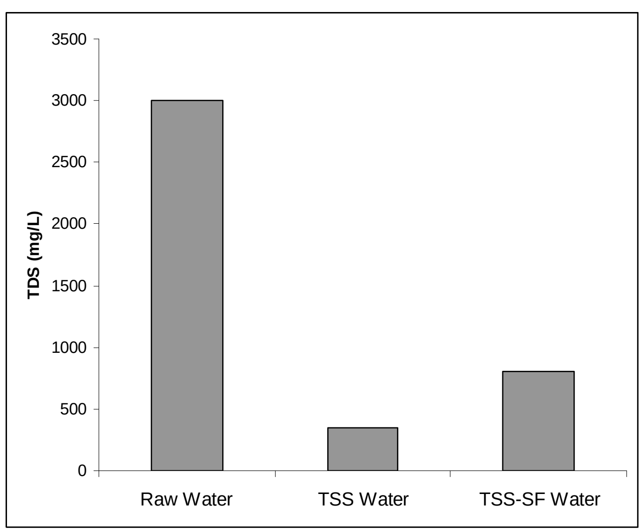

Figure 6: Variation of TDS among influent raw, effluent water of TSS and TSS-SF

According to WHO guideline (1993) and Bangladesh standards (BDS 1997), the TDS value of drinking water is 1000 mg/L and TDS value lying below 300 mg/L, the water is considered as excellent water. So, the TDS value is within the permissible limit.

The total coliform (TC) in the influent raw water was 260 no/L. The TC value of treated water from TSS and TSS-SF was found to be 0 no/L. The variation of TC value among influent raw water, effluent water of TSS and TSS-SF is shown in the figure 7 below:

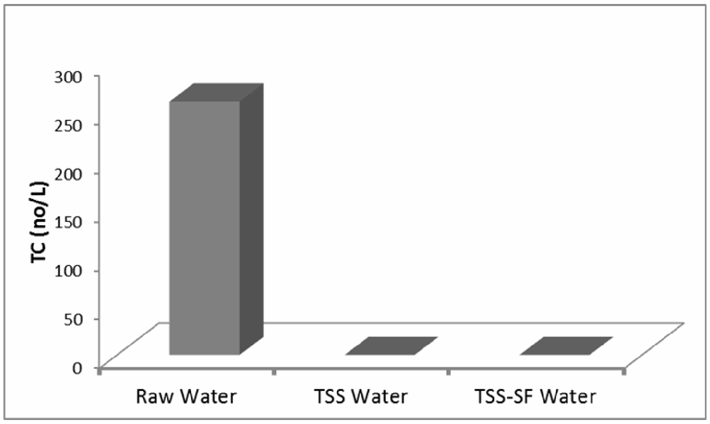

Figure 7: Variation of TC among influent raw, effluent water of TSS and TSS-SF

According to WHO guideline (1993) and Bangladesh standards (BDS 1997), the TC value of drinking water is 0 no/L. So, the TC value of treated water is within the permissible limit. So, it can be said that the treated water found from the TSS-SF unit is safe for drinking purposes. As, water borne diseases are caused by ingestion of pathogens with drinking water so this treated water could be an effective preventive measure against transmission of water borne diseases.

The construction cost of TSS was 500 Tk (BDT) and including it, the cost of storage channel and tubular frame was 250 Tk (BDT) and 120 Tk (BDT) respectively. The storage channel was made of good quality cartoon paper with high durable polythene so its construction cost was semi-high relative to the TSS. The construction cost of SF was 900 Tk (BDT). By adding the operational cost, the total cost of TSS-SF unit was 1750 Tk (BDT). The TSS-SF unit was placed strongly on a safe place so that high wind velocity could not do any harm to it. The TSS-SF was operated very carefully and maintained continuously.

The production rate of TSS-SF depends on the design of it and operating climatic conditions. For getting the high production rate field experiment should be conducted in summer season. The intensity of sunlight falling on the TSS is the most important factor affecting production. So, the TSS-SF unit should be placed where maximum sunlight is available and it should be safe against the environmental constraints. The sand should be proper sieving to discard fine and coarse fractions. The sand should not be too fine because of possibility of high initial head loss. There could be different losses in TSS like leakage of polythene and absorption of channel surface. So, it should be designed and operated very carefully. For better productivity TSS-SF unit requires constant maintenance.

4. CONCLUSIONS

Today, the world demand for potable water is increasing continuously due to industrial development, intensified agriculture, improvement of standard of life and increase of the world population. The supply of pure drinking water is a serious problem for most parts of the world. In dry season, this problem becomes very acute. But in dry season, the productivity of TSS-SF unit is higher. The TSS-SF unit is suitable for producing potable water from mostly available saline water in the coastal belt of Bangladesh. Though desalination is a slow process, but in the TSS-SF unit the production rate is higher than desalination process. The construction of the TSS-SF unit is very simple and one can use this technology easily in the house. So, it is concluded that the application of TSS-SF tecnique can fulfill the demand of potable water in the coastal areas of many developing countries like Bangladesh.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

HYDRO- MORPHOLOGICAL STUDY FOR REHABILITATION OF OLD MADHUMATI RIVER USING MATHEMATICAL MODEL

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ABSTRACT

Restoration of any river is indispensable to conserve and restore the surrounding environment. River restoration through dredging is one of the primary solutions for rehabilitation of rivers those get silted up causing adverse impacts on the state of the river channel and its morphology. Restoration of old channel now dead but once a flowing channel is a very complex tasks and need cautious attention regarding selection of its alignment, re-excavation volume and sustainability of dredge channel. Uses of modeling tools are very crucial for understanding the scenarios on future performance after rehabilitation and its sustainability. An in-depth study has been performed to restore the Old Madhumati river, which is situated in the south west region of Bangladesh. Once it was a free flowing river now became silted up causing serious environmental degradation in the surrounding area. Under this obstinately changing morphological condition of Old Madhumati river, the study included review, collection of time series satellite images, scenario development for cross-sections, velocity, setting the alignment of dredging, estimates of dredging using mathematical modeling which will authenticate sustainability. Careful investigation was conducted to examine whether the dredged canal needs supplementary measure to keep it hydraulically alive throughout the year or not including preliminary study on the environmental impacts.

Keywords: *Rehabilitation, River morphology, Dredging, Mathematical modeling, Sustainability*

1. INTRODUCTION

Bangladesh is a land of rivers and the environment and the life and livelihood of millions of people of Bangladesh are dependanr on riverine emvironment. Environment is the aggregate of conditions affecting the existence or development of life and nature. The overall global environment has undergone degradation due to the various development efforts and in Bangladesh it is declining faster for many obvious reasons, the main being due to the population pressure. Many silted up rivers have been occupied by the growing population and/or increasing food production as a result of population increase. Re-excavation of silted up rivers and channels bring immense benefit to maintain land and water equilibrium of an area provided that the re-excavated earth is disposed properly. It also brings various socio- economic benefits to the local people, although it involves quite a lot of finances. Proper re-excavation of rivers and channels also contributes to environmental development and restoration of an area.

Reduction of fresh water flow, over the years, through the Gorai has imparted significant impact on the river system of the south west region of Bangladesh. Prior to implementation of the Farakka barrage, south west region of Bangladesh did not face acute shortage of flow since the flow of the Gorai and other tributaries of the Ganges towards this region were sufficient to meet up the requirement for maintenance of environment and sustenance of livelihood of the people of the region. Another important river which also contributes to fresh water to this region is the Arial khan from which Kumar, Madaripur Beel Route (MBR) and other important tributaries take off. In the recent past, on account of declining flow of the Ganges, Gorai offtake has been silted up and flow through this river towards the South west region started to face shortage of flow, especially during dry season when no flow condition exists. On the contrary, flow of the Arial Khan did not show any indication of declining tendency, but flow of the Kumar reveals receipt of less amount of flow (SWMC 2002). It is known that within the basin of the Arial Khan-Kumar, some irrigation projects have been implemented during last few decades. Due to shortage of flow, overall hydro-morphological situation in the rivers of this region has been degraded. During dry period, tidal flow dominates and the sediment, which comes along with the tide, deposits. No significant flow comes from upstream to push the sediment load towards the sea that ultimately piles up degrading overall morphological condition at the downstream. Such relentless degradation might cause total closure of the flow through a branch of the Madhumati River entering into the Gopalganj town by isolating the Chapail area that meets the Madaripur Beel Route canal. Few years back, this linking branch channel was free-flowing River which gradually srank to a seasonally flowing river and is now locally named as Old (Mora) Madhumati River. The channel is now a leftover and saturated with water hyacinths during monsoon. Other than monsoon, it remains dry for most of the time. Every year during monsoon, both banks of the channel gets inundated due to siltation on the channel bed. Water of the river Mora Modhumati becomes polluted mainly due to disposal of sewerage wastes and other city wastes, which become severe during the dry period. Boat communication in the Mora Modhumati River has been limited due to siltation. Water quantum, flow and tidal prism have been reduced due to siltation mainly and they affected the growth of aquatic flora and fauna both qualitatively and quantitatively. At present people of the areas, specially living beside the river suffer from availability of surface water seasonally, mainly used for bathing, bathing of livestock and domestic purposes. Fish production and availability has also been reduced due to drying and pollution of habitat (IWM 2006).

2. STUDY OBJECTIVES

In view of the above background information, rehabilitation of Old Madhumati River through re-excavation has become a necessity to meet the public demands. Investigation intended to cover the hydro-morphological condition of the Madhumati canal integrating the Madhumati-Madaripur Beel Route (MBR) river system in order to determine the suitability of re-excavation, verify the suitability of existing alignment, sequence of re-excavation and required re-excavation volume to maintain the stable hydraulic regime section for continuous flow round the year. Investigations of these hydraulic issues have been carried out with the application of mathematical models. The core of the mathematical modeling study is the application of the MIKE21C modeling system, which is an advanced two-dimensional mathematical modeling. The tool helps assessing the various options on selection of dredging alignment, dredging quantity, depth, width, possible velocity and quantum of flow through the dredged channel etc which helps determining the sustainability of the rehabilitation works of the Old (Mora) Modhumati River.

3. REVIEW

Gorai flow has been interrupted and redueced during lean season, causing a lot of harm and degradation of the environmental and socio economic conditions along the river due to implementation of diversion of lean flow water at the Barrage in Farakka since 1975 (Hossain 1989). In the Southwest Area Water Resources Management Project (FAP4, 1993) executed in the period of 1991 to 1993, different scenarios for augmentation of the dry season flow in the Gorai river were investigated by one dimensional model. One of the schemes was dredging of a deep channel along the thalweg of the upper 30 kilometers of the Gorai river. Guide banks and groynes were suggested for the scheme. A two dimensional pilot model of the Gorai offtake including part of the Ganges was developed by the River Survey Project (FAP24 1996) using a large set of coherent data collected during 1995 and 1996 under FAP24 project. Following the signature of the Ganges water Sharing Treaty, studies were initiated under the Task Force by BWDB to steer the implementation of a scheme to restore the flow in the Gorai, based on the options identified by the Southwest Area Water Resources Management Project. As a follow up of this the Government has implemented GRRP phase I and also Phase II and currently the Ganges Barrage study is going on with a view to augmenting the Gorai River flow and to restore the ecosystem of the Southwest Bangladesh. Hossain et al (2003) and Hossain and Zaman (1996) studied the sediment transport and morphological aspects of the river system of the region including Gorai and Arial khan river which givers a very clear overview on the water management of the area. In addition to reviews mentioned above, studies on hydraulic process behind the siltation at the mouth of the old Madhumati canal from different literature, study, reports and paper from internet have been consulted (Hosain1992a, 1992b; Edmund and Slingerland 2007). The study area including the adjoining river system is shown if Fig. 1.

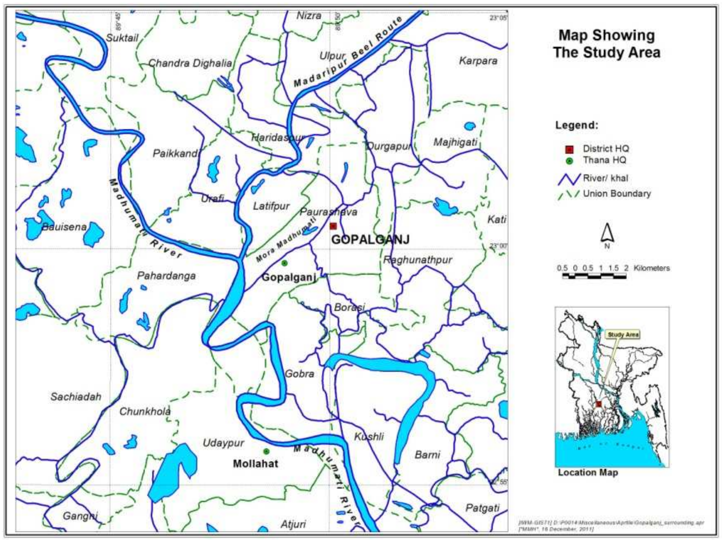

Figure 1: Map Showing the study area and adjoining river system

4. DATA AND METHODOLOGY

Various kinds of hydrometric data of recent and previous years have been collected and compiled. Main source of these data is IWM survey/measurement carried out during 2009 and 2010. Other sources are BWDB and CEGIS. During 2009 and 2010, water level and velocity data have been collected by survey group of IWM. As a part of updating the south west region model (SWRM) of Bangladesh to support flood forecasting and warning centre, BWDB, IWM routinely collects bathymetric data of the major rivers of this region. during December 2009 and June 2010 bank line data has been surveyed by IWM. Sediment data during June 2010 were collected at two locations; at Chapail ghat near the downstream end of the old Madhumati canal and the other at Haridashpur Bridge by IWM. Planform characteristics of the Old Madhumati and MBR-Madhumati confluence have been analyzed with the help of available satellite imagery of 2001, 2003 and 2005 collected from CEGIS.

Prior to development of the mathematical modeling, data analysis have been done extensively from the basis of hydraulic process of the Madhumati and MBR river system. The analysis of the data focused on the present and past morphological condition at the vicinity of the old Madhumati canal the sediment process, evolution of the channel and changes in the channel geometry etc. The study has also assessed the necessity of suitable type of structural interventions in order to keep the river in flowing stage. To achieve this, collection and compilation of all existing hydraulic and morphological data in relation with the river system concerned have been made. The hydrometric data includes water level and discharge data. These data are necessary to define model boundary and to compare the model generated results with the observed data. It is worth mentioning here that within the Old Madhumati channel, there is no discharge or water level gauge station. Since the flow of this channel is governed by the flow in the MBR and the Madhumati River, so available data for these two rivers have been collected. It is necessary to obtain more densely spaced bed level data in order to develop two-dimensional model. These data is needed to be collected during pre- or post-monsoon when the morphological activity is less than during the monsoon. To generate the computational grid for the two-dimensional model Bank line data were collected. Sediment data at Chapail ghat near the downstream end of the old Madhumati canal and the other at Haridashpur Bridge was also collected.

To analyse the sustainability of dredging , two dimensional morphological model has been developed by using MIKE 21C modelling tool. MIKE21C, two-dimensional modeling (hydrodynamic and morphological) is used as the main tool which simulates full hydrodynamic and morphological process of erosion of rivers with movable bed. MIKE21C applies curvilinear computational grid; it incorporates fully unsteady flow, bed load as well as suspended load including dynamic bed level changes through simple sediment continuity, updates the planform through bank erosion model and includes the computation of the secondary currents, and thus provides the 3D effects (IWM 2010).

5. ANALYSIS OF MORPHOLOGICAL CONDITION

With the help of available satellite imagery of the study area plan form characteristics of the Old Madhumati and MBR-Madhumati confluence have been analyzed (Figure 2). From the analysis, it is observed that the amount of siltation has increased in the Madhumati-MBR confluence. This is probably due to the reduced flow through the Gorai River. A char is observed in the downstream of the Madhumati-Old Madhumati confluence and the char is more pronounced in 2003 and 2005 than in 2001. This morphological change might be due to siltation in the Old Madhumati since this downstream char retards the flow through the Old Madhumati attracting more siltation. Reduction of the char periphery might be due to thrust of huge flow during 2004 in which flow was higher than that in 2003 and 2005. Width of the MBR is 185 m near the bridge and after meeting with the Madhumati at downstream, it increases. In the vicinity of the confluence of the Old Madhumati canal and the Madhumati, width is in the order of 600 m. Figure 3 shows the study area covering the MBR and the old Madhumati canal illustrating the bed levels at some of the important locations. It is seen from the figure that lowest bed level is at immediate downstream of the mouth of the old Madhumati canal and thalweg is located at opposite bank from the mouth. Lowest bed level is nearly -19.9 mPWD whereas near the mouth, it is around -9 mPWD. Analysis of the cross-sectional patterns in the vicinity of the bridge reveals the impact of the constriction scour and deposition of the scoured materials. Figure 4 shows the observed cross-sections taken near the bridge area. Cross-section at the bridge (line number J48) shows bed scour due to constriction made by the bridge. Going immediate downstream, sediment laden flow, getting un-constricted, is retarded and velocity is reduced. Due to reduction of flow velocity, these scoured materials are deposited on the bed which has been reflected on the downstream cross-sections, specially the cross section number J59, which is near the mouth of the old Madhumati canal.

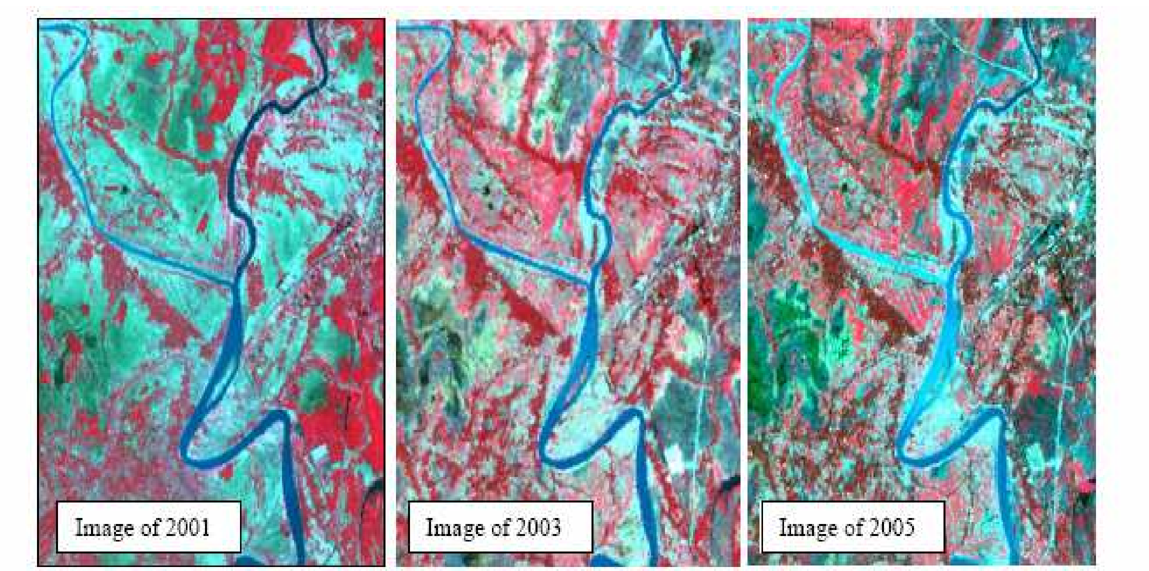

Figure 2: Satellite imageries of the study area indicating almost no change in shifting of bank lines of the MBR and Old Madhumati

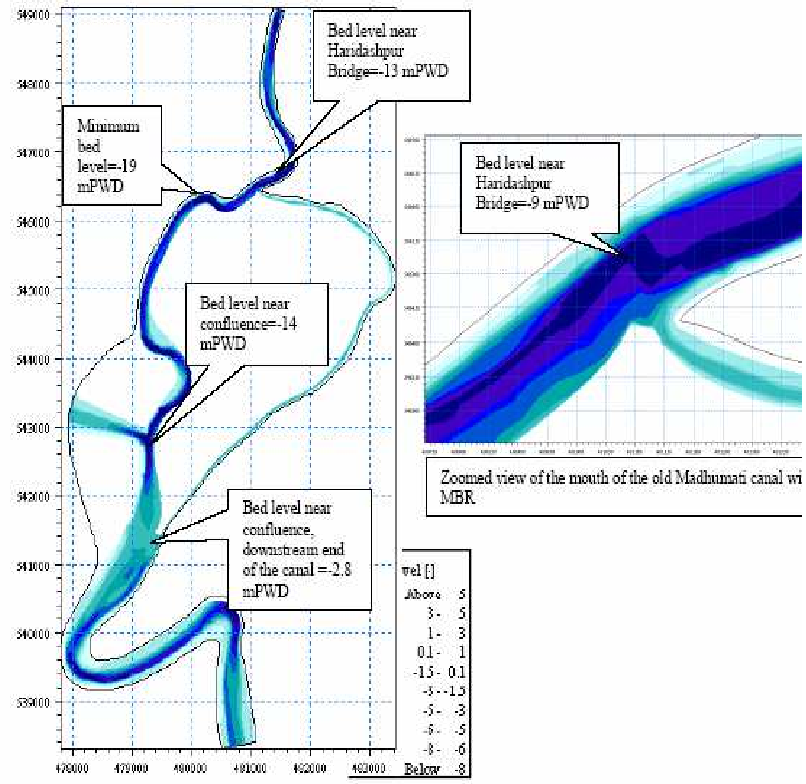

Figure 3: Study area of the MBR and Old Madhumati canal with description of bed levels

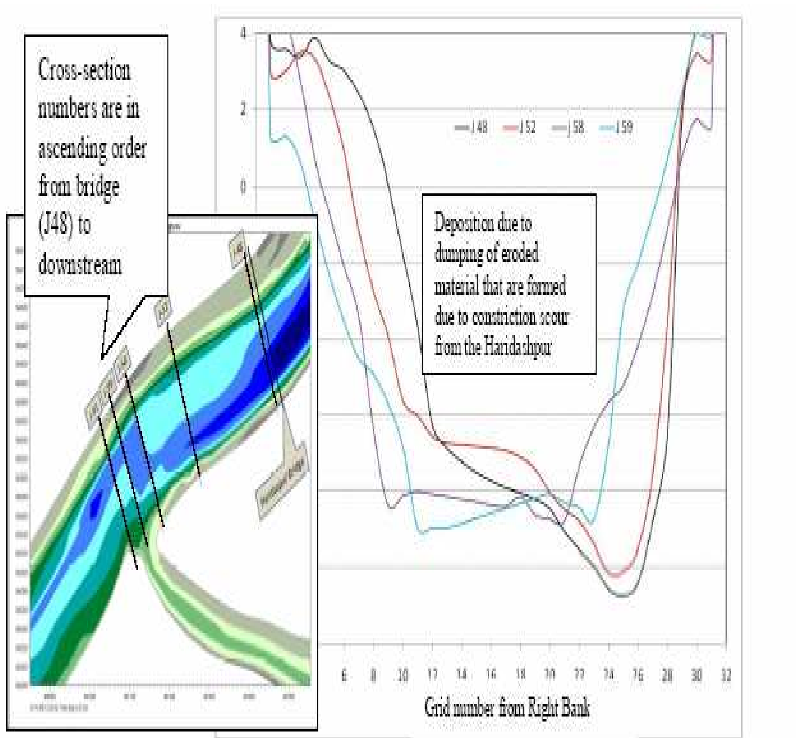

Figure 4: Cross-sectional changes in the vicinity of the Haridashpur Bridge

6. RESULTS AND DISCUSSIONS

Two dimensional-model incorporating the design section of the old Madhumati canal as shown in Figure5 and hydraulic parameters as shown in Table 1 indicate that the amount of flow within the old Madhumati canal varies from 30 to 74 cumecs (Table 2) and model results for low flood event indicate that the flow within the canal would remain around 100 cumecs and water depth more than 1 m persists in the canal. Design velocity of the section varies from 0.5 m/sec to 1 m/sec as per model results for 1998 high flood event. Figure 6 show that the velocity within the canal is in the range of 0.25 m/sec to 0.75m/sec which does not exceed the maximum permissible velocity. However, near the mouth of the old Madhumati canal, more than 1 m/sec velocity has been found along the outer bend (Figure 7). Such high velocities might threaten the stability of the bank at this point. During peak of the high flood event 1998, other parts of the canal have been seen to experience water level from 4 to 4.5 m PWD and the water level at the mouth is 4.45 to 4.5mPWD (Figure8). As per the design of the section, highest water level of the canal at the bank is 4m PWD, which is higher if compared with the highest water level of 1991 but if compared with the water level of 1998, then it is lower. However, considering the dyke height, possibility of inundation is not likely. For low flood event of 1991, model results show that within the outer bend of the canal near the mouth, erosion takes place whereas at the other part of the same section and other parts of the canal, deposition is observed (Figure 9). On account of the inclination angle, i.e. curvature, such formation is observed. At this location, deposition varies between 0.2m to 1m. Middle part of the canal also indicates deposition slightly higher (1.5 m). Some deposition at other parts in the order of 1 m is also noticed. Deposition at the downstream end and at the mouth generates unfavorable situation which ultimately hinders the free flow through the canal. Tendency of bank erosion has been observed from model simulations at Old Madhumati offtake to nearly 500 m downstream. Investigating the hydraulic phenomena behind the deposition at the mouth and at the confluence point, three sensitivity tests (options) have been considered for improvement, i.e. for smooth passage of flow through the canal. These tests have been carried out following flood event of 1991. Option 1 has been prepared by shifting the existing starting location of the mouth of the canal with the MBR towards 200 m upstream (Figure10), option 2 considered lowering the existing bed level at the mouth from -1 mPWD to -5 mPWD (Figure11), and Option 3 dealt with two inclined groynes along the right bank of the MBR, opposite to the mouth of the canal (Figure 12). If comparison is made among the flow conditions and de-silting tendency within the canal for different options, then it is seen that lowering the bed level near the mouth of the canal poses favorable situation (Figure13).

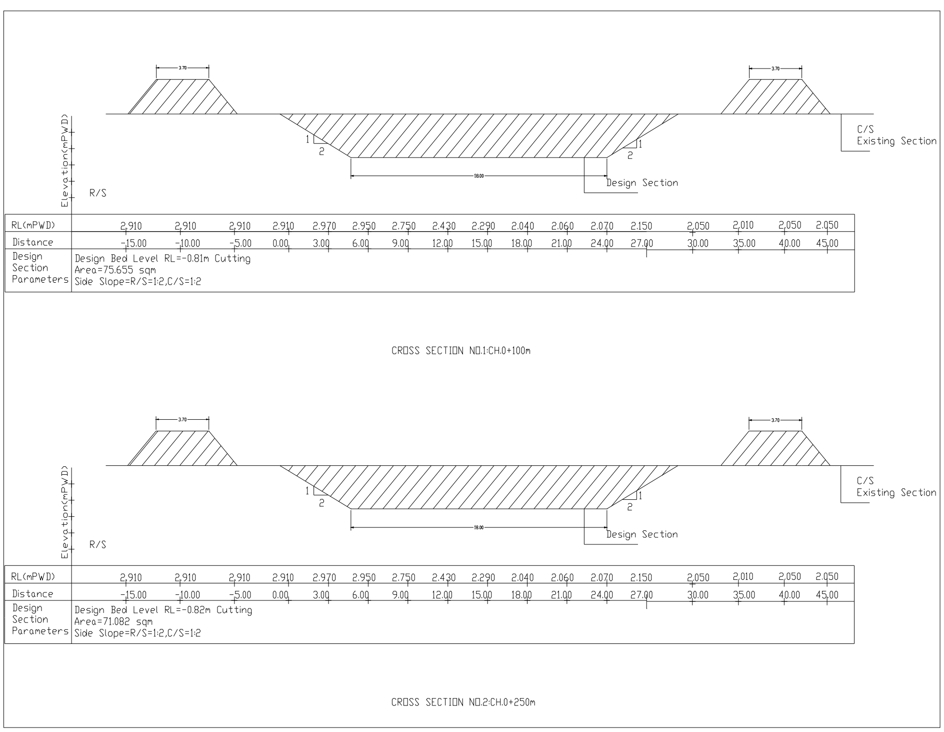

Figure 5: Dredging Cross section

Table1: Existing hydraulic parameters of the Old-Madhumati canal

			Existing				
Chainage	Reach	Cat Area(Ha)	Bed Level(mPWD)	BW (m)	TW(m)	Bank.GL (mPWD)	Bed Slope
Name of the Khal:Old Madhumati Khal,1900Km							
0			2.67	18	25	2.69/2.76	-0.0003
1900	1	1900	3.17	18	25	4.23/4.20	
Name of the Khal:Old Madhumati Khal,1100Km							
1900			3.17	18	25	4.23/4.20	
3000		1100	0.33	20	30	4.62/4.70	0.0026
Name of the Khal:Old Madhumati Khal,2000Km							
3000			0.33	20	30	4.62/4.70	
5000	2	2000	0.93	22	32	4.37/4.40	-0.0003
Name of the Khal:Old Madhumati Khal,3320Km							
5000			0.93	22	32	4.37/4.40	
8320	1	3320	0.56	25	35	3.48/3.50	0.0001

Table 2: Design hydraulic parameters of the Old-Madhumati canal

Design											
Drainage Rate(mm/day)	Q(m^3/s)	BasinWL (mPWD)	Outfall RWL(m)	WS	Side Slope	B(m)	d(m)	Chacek for V & Q	WL(mPWD)	BL(mPWD)	Depth of Cutting
Name of the Khal:Old Madhumati Khal,1900											
	3000			0.0001	1.5	18	2.9	V=0.495	3	0.1	2.57
	0	3	2.31					Q=32.058	2.81	-0.09	3.26
Name of the Khal:Old Madhumati Khal,1100											
	4500			0.0002	1.5	20	2.8	V=.894	2.81	-0.21	3.26
	0	3	2.31					Q=47.033	2.59	-0.81	
Name of the Khal:Old Madhumati Khal,2000											
	5500				1.5	22	3	V=.729	2.59	-0.21	0.54
	0	3						Q=57.942	2.19	-0.81	1.74
Name of the Khal:Old Madhumati Khal,3320											
	7400				1.5	25	3	V=0.889	2.19	-0.81	1.74
	0	3						V=8.701	1.227	-1.773	2.33

Morphology of the river system within the study area is governed by the combination of fresh water flow from upstream and tidal flow from downstream. Both the MBR and Madhumati rivers are associated with distinct meandering characters. Morphology of the MBR and old Madhumati canal has been assessed mainly with the analysis of the channel alignment and cross sectional changes. The hydrology of the study area is quite complicated because of the presence of tidal effect in the MBR and Madhumati River.

On the basis of the model results for different scenarios and analysis of available data it is clear that annual dredging is required for maintaining the canal since siltation is observed from the model results, at the mouth as well as other parts of the canal that generates unfavorable condition for water to flow from the MBR into the old Madhumati canal. Starting location of the mouth of the canal is required to be shifted towards upstream as much as possible keeping smooth alignment with the MBR. Lowering of bed level should be done in order to avoid abrupt change at the distributary point, where bed level in the MBR is -6 mPWD whereas in the old Madhumati canal, it is in the order of -1 mPWD. This difference needs to be changed along with smoothening of bed level from MBR to the old Madhumati canal needs to be done. Construction of 500 m bank protection works (Revetment) is required from Old Madhumati off take to downstream at right bank to counter such possible bank erosion. Downstream end of the canal also needs to be monitored since the design for this part experiences higher velocity of flow. Generation of higher velocity may also cause bank de-stabilization.

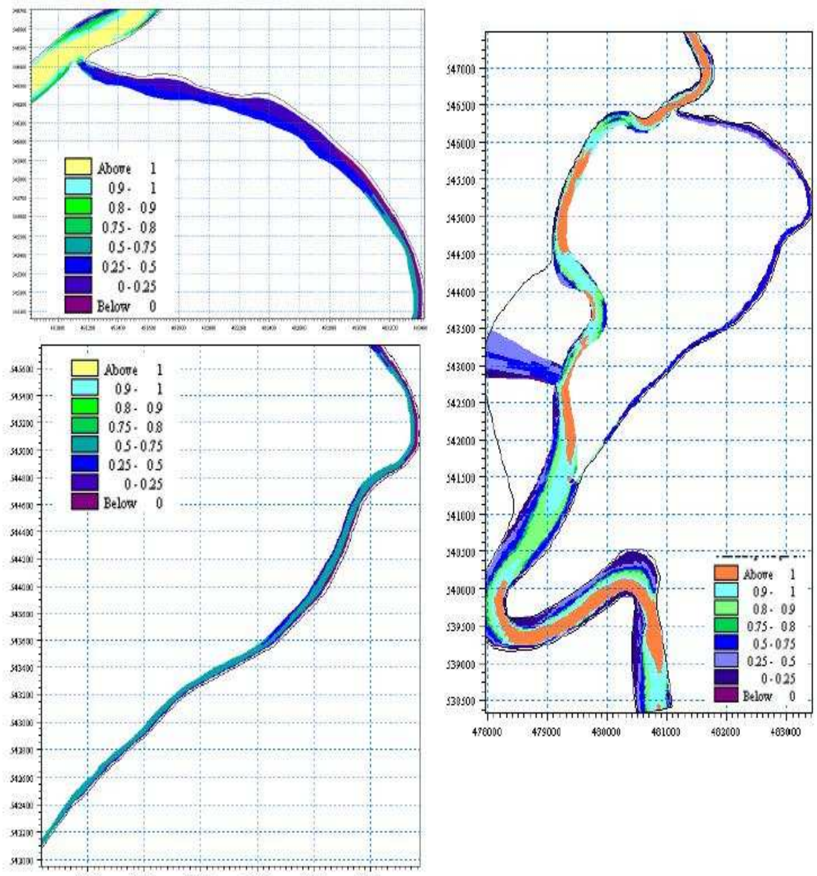

Figure 6: Simulated Peak velocities of the MBR-Madhumati river system and old Madhmati canal for extreme flood event of 1998.

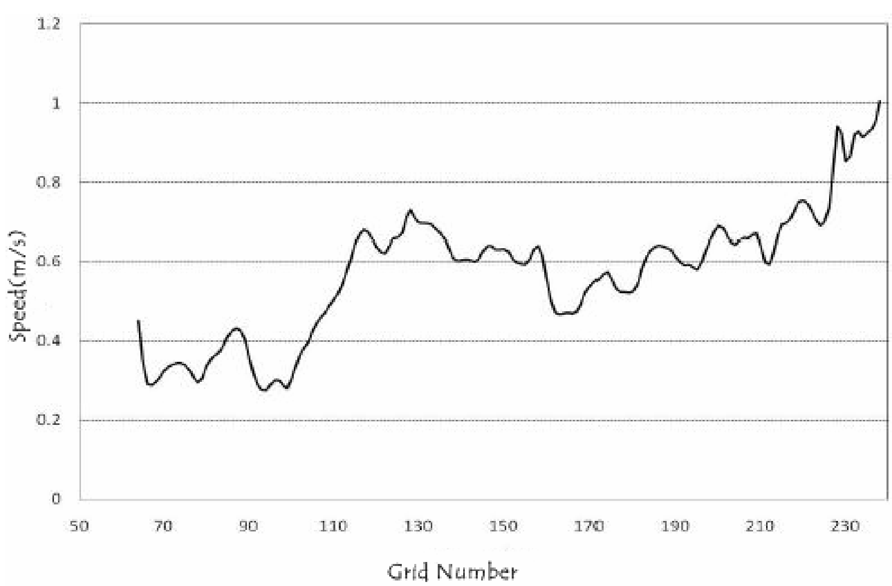

Figure 7: Simulated longitudinal velocity of the old canal during peak of 1998 flood event

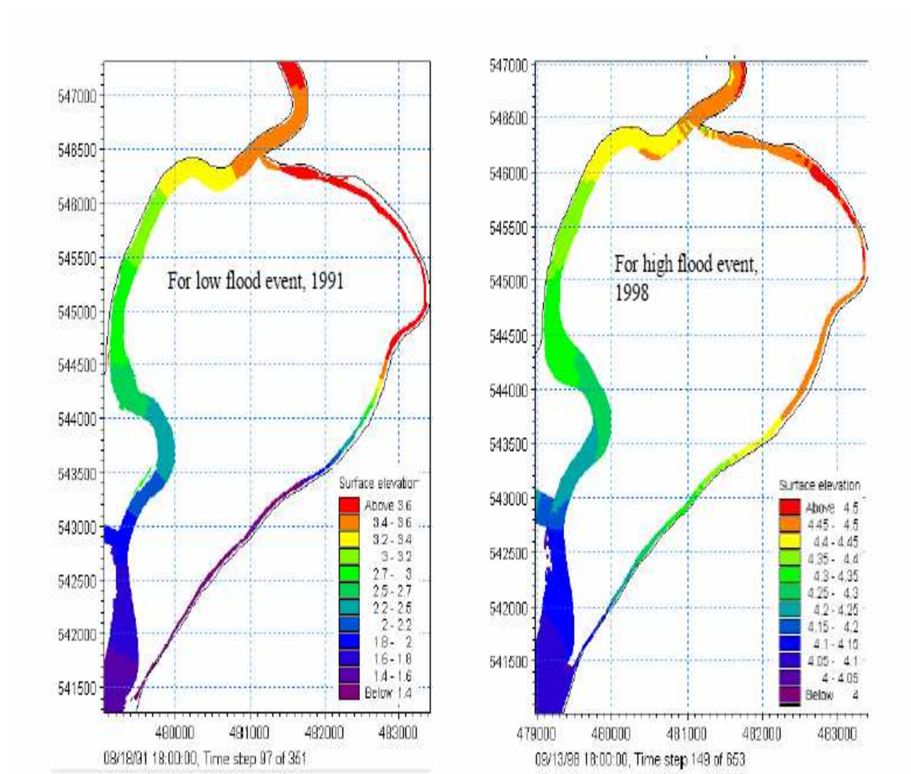

Figure 8: Simulated water levels for different events within the old canal

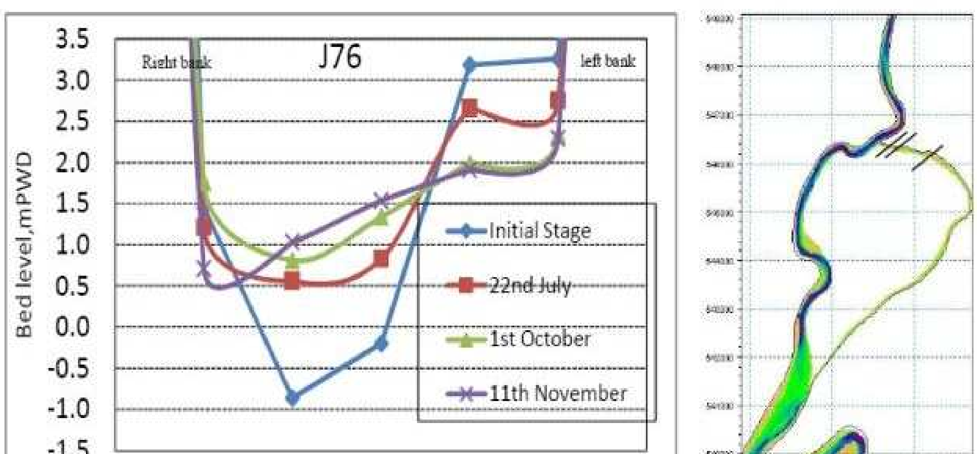

Figure 9: Cross sections at different stages of low flood event of 1991 indicating erosion at outer bend and deposition at mid part.

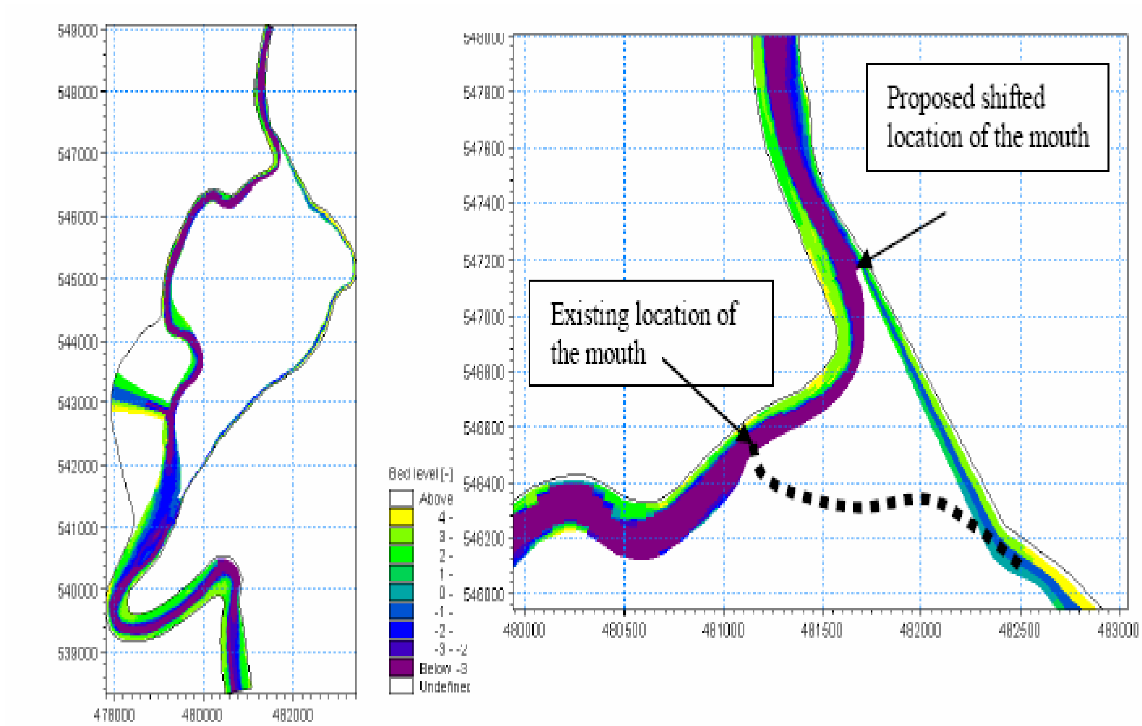

Figure 10: Proposed alignment of the canal at 200 m upstream from existing one

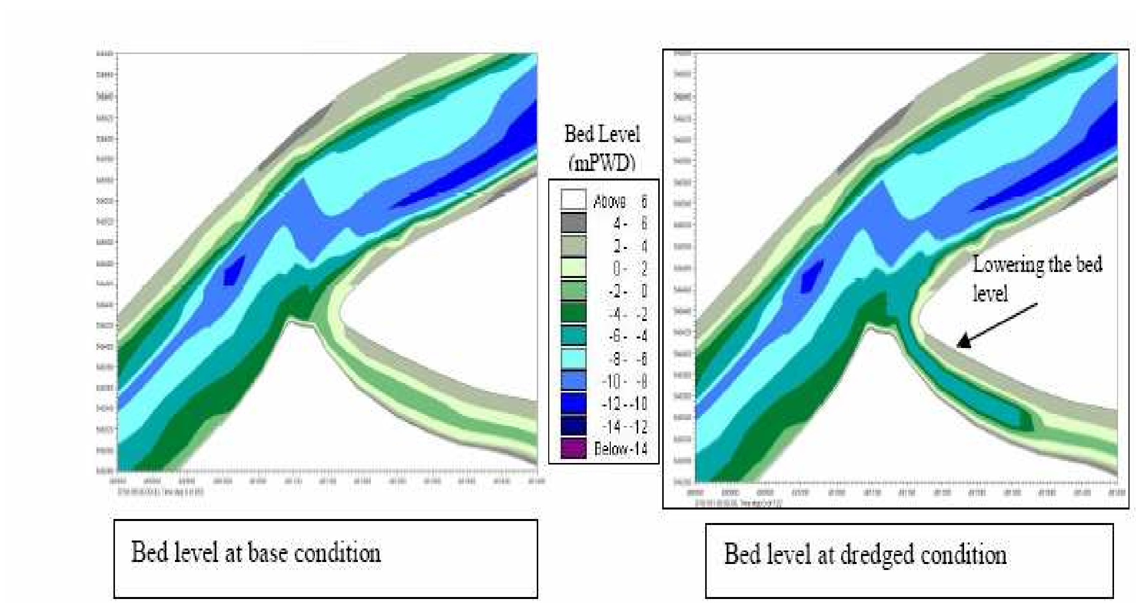

Figure 11: Bed Contour for two different set up;base condition (left) and dredged at mouth(right).

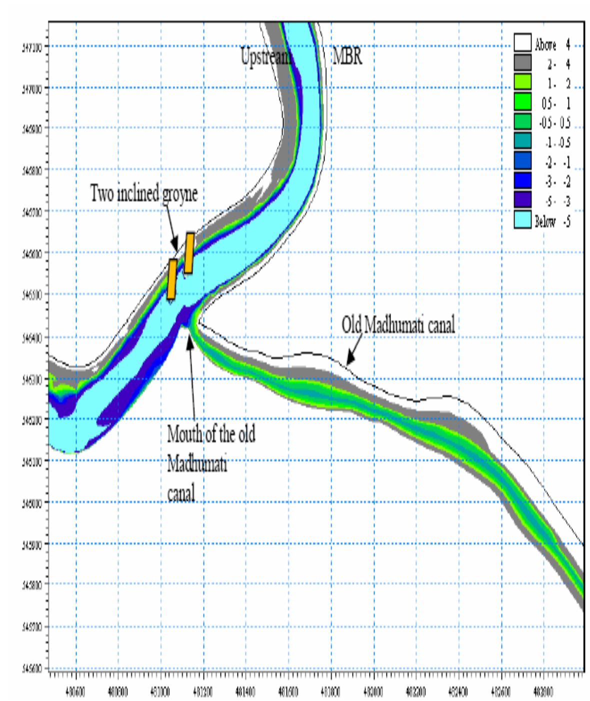

Figure12: Location of the proposed two groynes as incorporated in the model

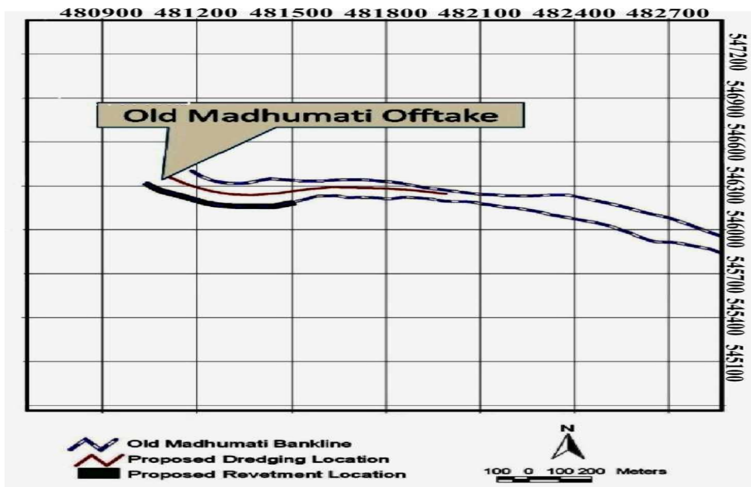

Figure 13: Proposed dredging and revetment locations on old Madhumati

According to the Environment Impact Assessment (EIA) the proposed intervention will bring some positive impacts such as improvement of flooding situation and drainage conditions. De-siltation of river bed is expected to remove water logging situation, salinity status, and improvement of surface water quality. Aquatic flora and fauna including fish, terrestrial flora and boat communication/navigation will develop. Agricultural production and irrigation for crop production will develop. Status of water pollution, communication, health hazard situation, aesthetic conditions and scope of recreation and scope of employment will improve. Some negative impacts like loss of land, displacement of some settlements/ people, enhancement of poverty for some displaced population, lack of awareness of the proposed development among the local people, creating worries and uncertainties among the local population affecting people's participation.

7. CONCLUSIONS

Required amount of flow within the old Madhumati canal varies from 30 to 74 cumec during lean season and model results for low flood event indicate that the flow within the canal would remain around 100 cumecs and water depth more than 1 m in the canal. Design velocity of the section is 0.5 m/sec to 1 m/sec and the model results for 1998 (high flood event) show that the velocity within the canal is in the range of 0.25 m/sec to 0.75 m/sec which does not exceed the maximum permissible velocity. However, near the mouth of the old Madhumati canal, more than 1 m/sec velocity has been found along the outer bend. Such high velocity might threaten the stability of the bank at this point. At this flood event, other parts of the canal have been seen to experience water level from 4 to 4.5 mPWD and the water level at the mouth is 4.45 to 4.5 mPWD. As per the design of the section, highest water level of the canal at the bank is 4 m PWD, which is higher if compared with the highest water level of 1991 but if compared with the water level of 1998, then it is lower. However, considering the dyke height, possibility of inundation is not likely to occur. For low flood event of 1991, model results show that within the outer bend of the canal near the mouth, erosion takes place whereas at the other part of the canal, deposition is observed. Presence of curvature influences the erosion and deposition which varies between 0.2m to 1 m. Middle part of the canal also indicates deposition slightly higher (1.5 m). Tendency of bank erosion has been observed from model simulations at Old Madhumati off take to nearly 500 m downstream. Environmental Impact Analysis shows that major improvement is expected to occur after implementation of the rehabilitation of the Old (Mora) Madhumati River.

ACKNOWLEDGEMENT

Support and cooperation obtained from Institute of Water Modeling (IWM), Dhaka is deeply acknowledged.

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THE POSSIBLE EFFECT OF SEA LEVEL RISE ON THE LIVLIHOOD OF BEEL DAKATIA: LOCAL PEOPLES' PERCEPTION

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ABSTRACT

The South-West coastal region (Beel Dakatia) of Bangladesh is inversely affected by SLR. To justify the impacts of sea level rise in Beel Dakatia region, eight villages named Shiromoni, Dakatia, Saravita, Salua Bazaar, Thukla, Shahpur, Bejerdanga and Raripara were surveyed and peoples opinion regarding SLR was taken. 12.2% of the total population which includes 680 people and 137 families were surveyed. The livelihood parameters are categorized in four social dimensions; they are physical, human capital, natural and socio-economical. Education is considered as human capital parameter, transportation and housing systems are physical parameter, water resources as natural parameter and profession is considered as socio-economical parameter. Considering the present resilience as 100%, the resilience of people at human capital, natural, physical, and socio-economical conditions are estimated to be reduced by 45%, 50%, 40% and 60% respectively due to the effect of 1ft SLR. For 2ft SLR the resilience of people for the same dimensions are estimated to be reduced by 95%, 95%, 88% and 85% respectively. To adapt with the 1ft increase of water level, local people think that they have to raise plinth level of their houses, raise their tube well base height, raise the school or college premises and sanitary latrines. If water level rises 2ft and if the water stands for a long time then they think that they have to leave the place. Peoples are thinking to mitigate the adverse effect of SLR locally, which is insufficient and is very limited corresponding to their needs.

Keywords: *Climate Change, Sea Level Rise, Beel Dakatia, Livelihood Parameters, Adaptation.*

1. INTRODUCTION

Climate change associated with sea level rise (SLR) is one of the major environmental concerns of today. Climate change was projected to impact tropical countries more negatively than the temperate ones. In the Third Assessment Report (TAR) of the Intergovernmental Panel on Climate Change (IPCC) the projected rise from 1990 to 2100 was 9 to 88 cm. The SAARC Meteorological Research Centre (SMRC) analyzed sea level changes of 22 years historical tide data at three tide gauge locations in the coast of Bangladesh. The study revealed that the rate of sea level rise during last 22 years are many fold higher than the mean rate of global sea level rise over 100 years. A study by Ahmed and Alam (1998) mentioned one meter change of sea level by the middle of 21st century; it combines a 90 cm rise in sea level and about 10 cm local rise due to subsidence. Pilot study of Department of Environment (DOE, 1993) mentioned a potential future sea level rise for Bangladesh is 30-50 cm by 2050. An increasing tendency in sea level rise from west to east along the coast has also been found. Bangladesh is highly vulnerable to sea level rise, as it is a densely populated coastal country of smooth relief comprising broad and narrow ridges and depressions (Brammer et al., 1993). World Bank (2000) showed 10 cm, 25cm and 1 m rise in sea level by 2020, 2050 and 2100; affecting 2%, 4% and 17.5% of total land mass respectively. Milliman et al. (1989; cited in Frihy, 2003) reported 1.0 cm per year sea level rise in Bangladesh. UNEP (1989) showed 1.5 m sea level rise in Bangladesh coast by 2030 affecting 22,000 Sq. km (16% of total landmass) area with a population of 17 million (15% of total population) affected. Since this scenario was calculated in 1989, the expected rate of sea level rise has been modified because of uncertainty. At present expected rates, this situation will occur in about 150 years from now. However, number of potential population affected by the projection of World Bank by one meter sea level rise (17.5 million) and that of UNEP by 1.5 meter sea level rise (17 million) is similar. Beel dakatia is the second largest beel (smallest depression in floodplain) in Bangladesh. It stands on the South-West coastal region of Bangladesh with an area about 17400 hectors or 174000000 square meters under Dumuria, Phultala, and Avaynagar thanas in coastal district of Khulna and Jessore. Beel dakatia includes five union named Fultola, Gilatola, Raghunathpur, Rangpur and Damalia. It lies between longitude $89^0 29'$E and $89^0 35'$E. The elevation of this area is low and topography is almost flat. The surrounding rivers of this area are Solmari, Hamkura and Salta. During tide and saline water from sea comes to the beel through these rivers and during rainy season the excess rainwater is discharged.

Fig. 1 Location of beel dakatia

The main objectives of the study are to investigate the impacts of SLR on BEel Dakatia based on local people's perception. The present and possible future scenario of polder system and drainage facility (condition of embankment, sluice gates and peripheral rivers) due to sea level rise was also investigated. Finally, people's opinion on the local adaptation systems for the mitigation of impacts of SLR in the study area investigated.

2. METHODOLOGY

A questionnaire survey was conducted in eight villages of Beel Dakatia region to accumulate the local peoples' opinion about the effect of SLR on their livelihood and their local adaptation techniques. The questionnaire consists of questions about effect of SLR and their possible impacts on the livelihood of the local people. Four livelihood parameters were considered: physical, human capital, natural and socio-economical. Education is considered as human capital parameter, transportation and housing systems are physical parameter, water resources as natural parameter and profession is considered as socio-economical parameter. It includes 3 key scenarios: (a) Existing visible impacts**,** (b) Future impacts if 1 ft (30.5 cm) permanent inundation occurs and (c) Future impacts if 2 ft (61cm) permanent inundation occurs. Based on the present scenario and local peoples' past experience about the polder system and drainage facility, a future scenario is prepared for cooping the present trend of sea level rise. Survey was also conducted to investigate the local peoples' opinion about the future adaptation techniques for SLR.

3. FUTURE IMPACTS OF SEA LEVEL RISE

Eight villages named Shiromoni, Dakatia, Saravita, Salua Bazaar, Thukla, Shahpur, Begerdanga and Rari Para in Gilatoli, Rangpur, Raghunathpur and Phultola unions were surveyed to study the impacts of SLR in beel dakatia. Parameters that are considered as peoples' livelihood are profession, houses, water sources, transportation system, education system and sanitation system. Percentages of people in survey at eight villages in beel dakatia are tabulated in table 1.

Table 1 Percentage of involvement in survey at eight villages in beel dakatia

Village	Union	Total population	No. of people surveyed	People surveyed (%)
Shiromoni	Gilatola	920	81	8.8
Dakatia	Gilatola	730	62	8.5
Saravita	Rangpur	650	97	14.9
Salua Bazaar	Rangpur	550	88	16.0
Thukla	Raghunathpur	640	83	12.9
Shahpur	Raghunathpur	520	72	13.8
Begerdanga	Phultola	710	83	11.7
Rari Para	Phultola	856	114	13.3
Total		5576	680	12.2

Future impacts of SLR in beel dakatia are discussed individually for every village at sections 3.1 to 3.8. Overall future impacts of SLR in the study area are discussed in the section 4..

3.1 Study area: Shiromoni, Union: Gilatola

For this case, the number of families was surveyed = 17, which consists 81 peoples (8.8% of total population of this village). If water level rises 1 ft in the study area, 17.65% people think that they will change their profession; among them 66.67% people will move into shrimp culture and 33.33% people will move into business. 41.18% people think that their drinking water sources will be submerged. 35.51% people think that their houses will be flooded; among them 66.67 % will move into other higher places and 33.33% will remain in the same place. 23.5% people think that their roads (transport system) will be submerged. 47.1% people think that their education system (schools, colleges) will be hampered. 35.51% people think that their sanitation system will be flooded.

If water level rises 2 ft from sea level, 70.59% people think that they will change their profession; among them 16.67% people will move into shrimp culture, 50% will move into business and 23.33% will move into others. 76.47% people think that their drinking water sources will be submerged. 88.24% people think that their houses will be flooded; among them 80 % people will move into other higher places and 20% will remain in the same place. 23.5% people think that their roads (transport system) will be submerged. 82.35% people think that their education system (schools, colleges) will be hampered.

3.2 Study area: Dakatia, Union: Gilatola

For this case, the number of families was surveyed = 14, which consists 62 people (8.5% of total population in this village). If water level rises 1 ft in the study area, 42.86% people think that they will change their profession; among them 66.67% people will move into business and 33.33% will move into shrimp culture. 64.28% people think that their drinking water sources will be submerged. 57.14% people think that their houses will be flooded; among them 75% will move into other higher places and 25% will remain in the same place. 23.5% people think that their roads (transport system) will be submerged. 42.85% people think that their education system (schools, colleges) will be hampered.

If water level rises 2 ft in the study area, 85.71% people think that they will change their profession; among them 33.33% people will move into shrimp culture and 66.67% will move into business. 100% people think that their drinking water sources will be submerged. 100% people think that their houses will be flooded; among them 75% will move into other higher places and 25% will remain in the same place. 35.71% people think that their roads (transport system) will be submerged. 78.57% people think that their education system (schools, colleges) will be hampered.

3.3 Study area: Saravita, Union: Rangpur

For this case, the number of families was surveyed = 19, which consists 97 people (14.9% of total population of this village). If water level rises 1 ft from in the study area, 42.1% % people think that they will change their profession; among them 75% people will move into shrimp culture and 25% will move into business. 47.4% people think that their drinking water sources will be submerged. 63.2% people think that their houses will be flooded; among them 71% will move into other higher places and 29% will remain in the same place. 23.5% people think that their roads (transport system) will be submerged. 31.57% people think that their education system (schools, colleges) will be hampered.

If water level rises 2 ft in the study area, 84.21% people think that they will change their profession; among them 15% people will move into shrimp culture and 85% will move into business. 100% people think that their drinking water sources will be submerged. 100% people think that their houses will be flooded; among them 75% will move into other higher places and 25% will remain in the same place. 85.72% people think that their roads (transport system) will be submerged. 100% people think that their education system (schools, colleges) will be hampered.

3.4 Study area: salua bazaar, Union: Rangpur

For this case, the number of families was surveyed = 17, which consists 88 people (16% of total population of this village). If water level rises 1 ft in the study area, 52.94% % people think that they will change their profession; among them 44.44% people will move into shrimp culture and 56.56% will move into business. 41.17% people think that their drinking water sources will be submerged. 64.71% people think that their houses will be flooded; among them 63.64 % will move into other higher places and 36.36% will remain in the same place. 23.5% people think that their roads (transport system) will be submerged. 47.1% people think that their education system (schools, colleges) will be hampered.

If water level rises 2 ft in the study area, 82.24% people think that they will change their profession; among them 71.43% people will move into shrimp culture and 28.57% will move into business. 100% people think that their drinking water sources will be submerged. 88.23% people think that their houses will be flooded; among them 63.64 % of them will move into other higher places and 36.36% will remain in the same place. 84% people think that their roads (transport system) will be submerged. 100% people think that their education system (schools, colleges) will be hampered.

3.5 Study area: Thukla, Union: Raghunathpur

For this case, the number of families was surveyed = 18, which consists 83 people (12.9% of total population of this village). If water level rises 1 ft in the study area, 44.44% people think that they will change their profession; among them 62.5% people will move into shrimp culture and 37.5% will move into business. 50% people think that their drinking water sources will be submerged. 55.56% people think that their houses will be flooded; among them 60 % will move into other higher places and 40% will remain in the same place. 44.44% people think that their roads (transport system) will be submerged. 38.89% people think that their education system (schools, colleges) will be hampered.

If water rises 2 ft in the study area, 88.88%people will change their profession; among them 12.5% people will move to shrimp culture and 87.5% will move into business. 100% people think that their drinking water sources will be submerged. 83.33% people think that their house will be flooded; among them 80% will move into other higher places and 20% will remain in the same place. 77.77% people think that their roads (transport system) will be submerged. 88.88% people think that their education system i.e.; schools, colleges or other media will be hampered.

3.6 Study area: Shahpur, Union: Raghunathpur.

For this case, the number of families was surveyed = 16, which consists 72 people (13.8% of total population of this village). If water level rises 1 ft in the study area, 43.75% people will change their profession; among them 57.15% people will move into shrimp culture and 42.86% will move into business. 31.25% people think that their drinking water sources will be submerged. 37.5% people think that their houses will be flooded; among them 33.33 % will move into other higher places and 66.67% will remain in the same place. 25% people think that their roads (transport system) will be submerged. 31.25% people think that their education system (schools, colleges) will be hampered.

If water level rises 2 ft in the study area, 87.5%people think that they will change their profession; among them 57.14% people will move into shrimp culture and 43.86% will move into business. 81.25% people think that their drinking water sources will be submerged. 93.75% people think that their houses will be flooded; among them 80% will move into other higher places and 20% will remain in the same place. 77.77% people think that their roads (transport system) will be submerged. 81.25% people think that their education system (schools, colleges) will be hampered.

3.7 Study area: Begerdanga, Union: Phultola

For this case, the number of families was surveyed = 16, which consists 83 people (11.7% of total population of this village). If water level rises 1 ft in the study area, 31.25% people think that they will change their profession; among them 80% people will move into shrimp culture and 20% will move into business. 31.25% people think that their drinking water sources will be submerged. 37.5% people think that their houses will be flooded; among them 33.33 % will move into other higher places and 66.67% will remain in the same place. 25% people think that their roads (transport system) will be submerged. 43.75% people think that their education system (schools, colleges) will be hampered.

If water level rises 2 ft in the study area, 93.75%people think that they will change their profession; among them 26.67% people will move into shrimp culture and 73.33% will move into business. 81.25% people think that their drinking water sources will be submerged. 100% people think that their houses will be flooded; among them 87.5% will move into other higher places and 12.5% will remain in the same place. 81.25% people think that their roads (transport system) will be submerged. 93.75% people think that their education system (schools, colleges) will be hampered.

3.8 Study area: Rari para, Union: Phultola

For this case, the number of families was surveyed = 20, which consists 114 people (13.3% of total population of this village). If water level rises 1 ft in the study area, 40% people think that they will change their profession; among them 62.5% people will move into shrimp culture and 37.5% will move into business. 35% people think that their drinking water sources will be submerged. 45% people think that their houses will be flooded; among them 75% will move into other higher places and 25% will remain in the same place. 30% people think that their roads (transport system) will be submerged. 35% people think that their education system (schools, colleges) will be hampered. 60% people think that their sanitation system will be flooded.

If water level rises 2 ft from sea level, 85% people think that they will change their profession; among them 23.5% people will move into shrimp culture and 76.5% will move into business. 95% people think that their drinking water sources will be submerged. 90% people think that their houses will be flooded; among them 77.77% of them will move into other higher places and 22.22% will remain in the same place. 75% people think that their roads (transport system) will be submerged. 90% people think that their education system (schools, colleges) will be hampered.

4. OVERALL IMPACTS ON LIVELIHOOD

Fig. 2 illustrates the change of livelihood parameters against 1ft permanent inundation. If water level rises 1 ft from sea level, 40% people think that they have to change their profession; among them 60% people will move into shrimp culture and 40% will move into business. 35% people think that their drinking water sources will be submerged. 50% people think that their houses will be flooded; among them 75% thinki that they have to move into other higher places and 25% will remain in the same place. 30% people think that their roads (transport system) will be submerged. 45% people think that their education system (schools, colleges) will be hampered. If water level rises 2 ft from sea level, 85% people think that they have to change their profession; among them 40% people will move into shrimp culture and 60% will move into business. 90% people think that their drinking water sources will be submerged. 95% people think that their houses will be flooded; among them 80% will move into other higher places and 20% will remain in the same place. 80% people think that their roads (transport system) will be submerged. 90% people think that their education system (schools, colleges) will be hampered.

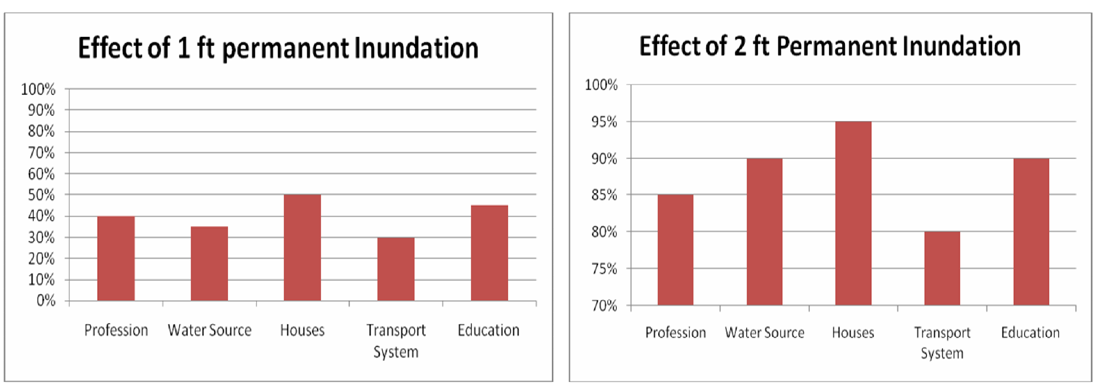

Fig 2: Effect of 1 and 2 ft Permanent Inundation in the Study Area

4.1 Livelihood Tetragons Presenting Peoples Resilience

Livelihood tetragons (fig:3) were drawn to represent the people's resilience of the surveyed area. It have been seen that local peoples resilience are very low with respect to 1 and 2 ft permanent inundation on the study area due to SLR. Stress on peoples livelihood are much more due to 2 ft SLR than 1 ft SLR. So if 2 ft SLR occurs and if it causes permanent inundation than its effects can be much vulneable.

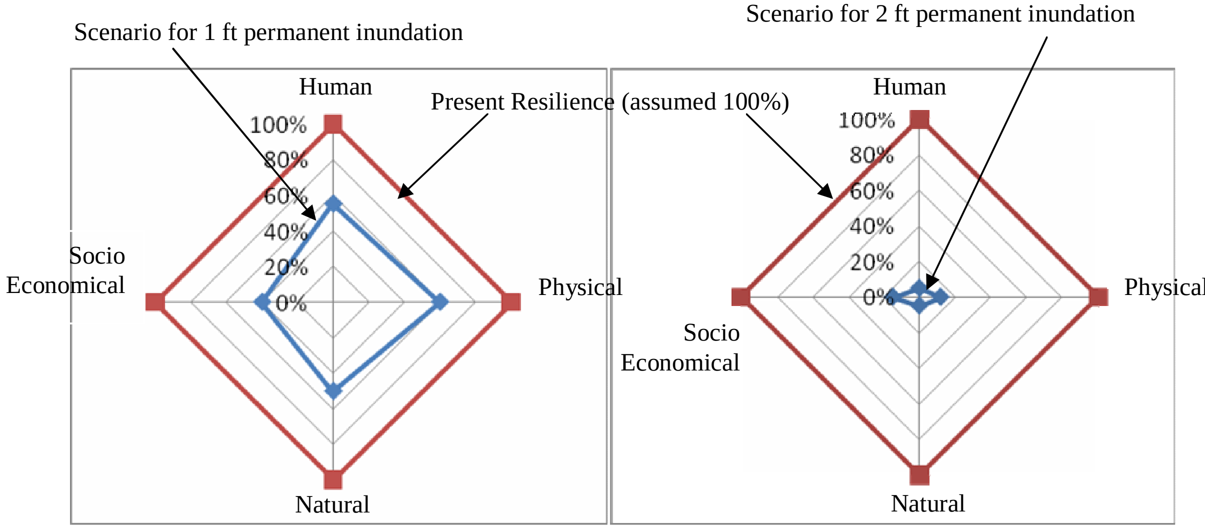

Fig 3: Livelihood Tetragons Presenting Peoples Resilience if 1 and 2 ft Permanent Inundation

5. Present and past Scenario of Polder

The polder system consists of embankments and sluice gates. The present and past condition of embankments and sluice gates are described below:

5.1 Condition of embankments

Fig. 4: Present embankment system in beel dakatia (Raghunathpur, March, 2011)

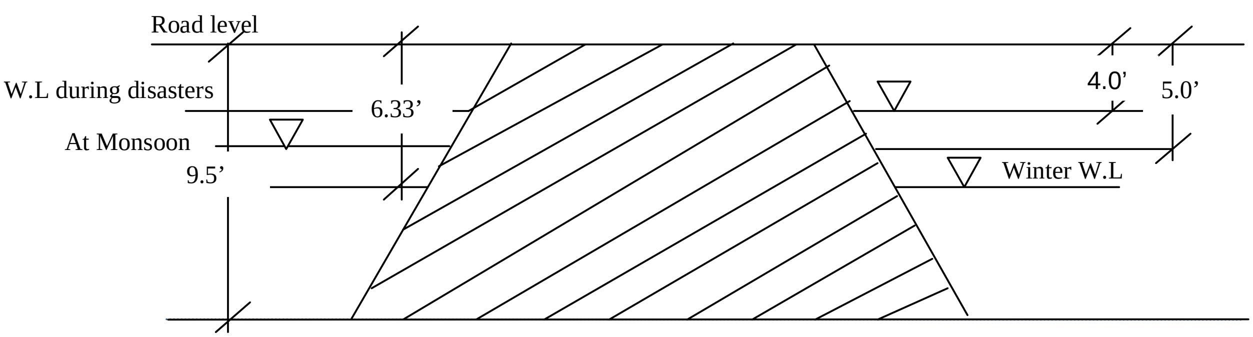

Fig. 5: Cross section of a typical embankment

Whole beel dakatia is surrounded by the embankments. Different types and heights of embankments are found there. Fig. 4 shows the pictorial view of a typical embankment. Fig. 5 represents the cross section of a typical embankment, where average height of embankments are 9.5 ft, in February height of water surface from top was 6.33 ft, at rainy season height of water surface from top was 5.0 ft, during Aila height of water surface from top was 4.0 ft below the top of embankment, at 1988 and 1998 flood water flew about 1.0 ft and 1.5 ft over of the embankments.

5.2 Condition of Sluice Gates

Fig. 6: Pictorial view of sluice gate at Salua Bazaar (February, 2011)

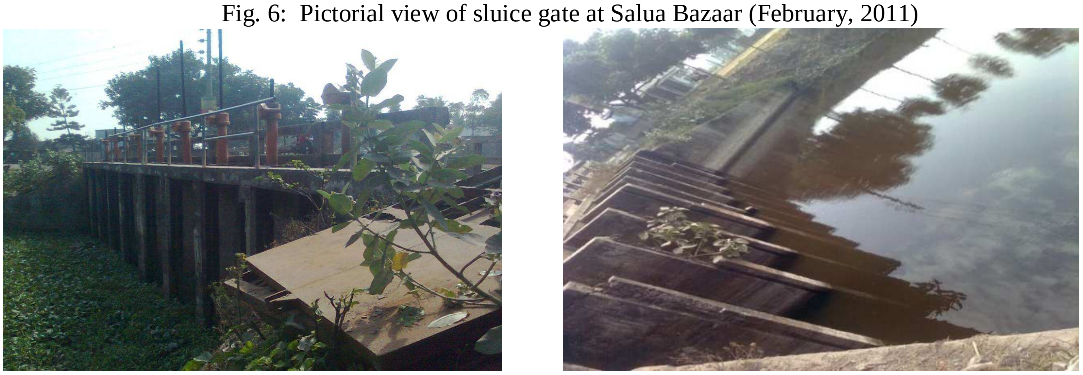

Fig. 7: Pictorial view of sluice gate at Amvita (February, 2011)

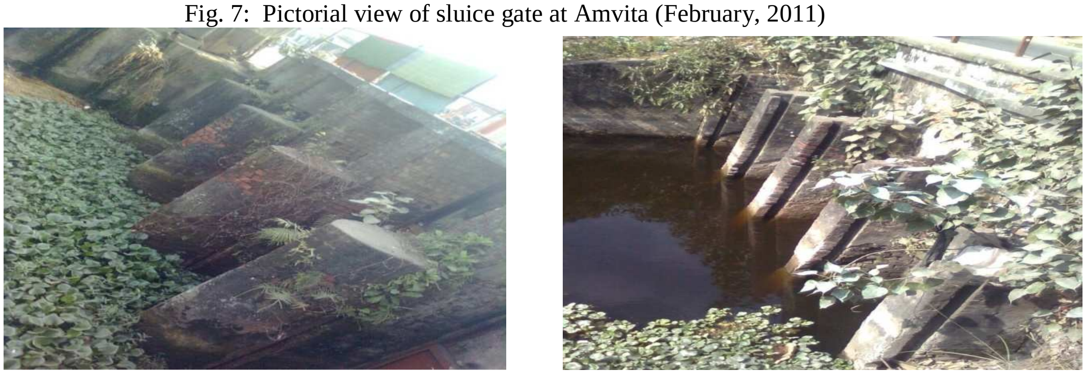

Fig.8: Pictorial view of sluice gate at Thuklar Bazaar (February, 2011)

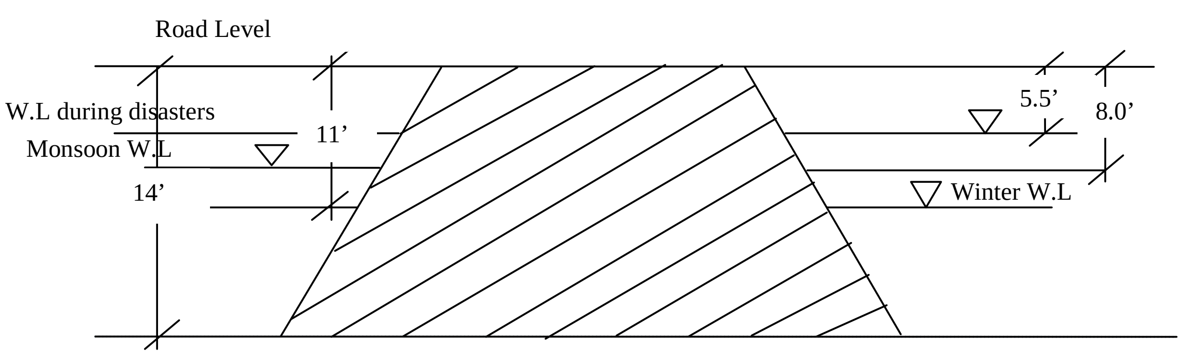

Fig. 9: Cross Section of a Sluice Gate

From their construction in 1960, the three sluice gates situated at Salua Bazaar, Amvita and Thukla Bazaar are used to drain out the excessive water from beel dakatia during rainy season. Fig. 6, Fig. 7 and Fig. 8 represent the pictorial view of sluice gates at Salua Bazaar, Amvita and Thukla Bazaar respectively. These sluice gates are not only used to drain out the excess water but also used to allow the entry of water in high tide period and during low tide period water flows back to the river on the same route. Cross-section of a typical sluice gate is represented by Fig. 9, where average height of sluice gate is 14'. At February height of water surface from top was 11 ft below the top slab. At rainy season height of water surface from top is generally 8.0 ft. During Aila height of water surface from top was 5.5 ft. At 1998 and 1988 flood water flew about 1ft and 1.25 ft over the sluice gates.

5.3 Future Impacts on Polder System

Present scenario of embankments and sluice gates are not satisfactory due to lack of maintenance. Average height of embankments and sluice gates are 10 ft and 14 ft respectively. In rainy season, height of water surface is below 6 ft from top. At present situation 1 ft or 2 ft increase of water level will not create harmful condition because in recent past natural disaster like Aila, Tsunami, flood after 2000 or at rainy season, the water depth were not create any harmful effect. However following three scenarios may create permanent inundation in the area.

If devastating flood as like 1988 or 1998 occurs then the situation will be measurable. The water that will be overtopped the embankments will not discharge out after the flooding period, and may cause permanent inundation. Another possibility is, if sluice gates do not maintain and operate properly, water may enter to the polder enclosed area and cause permanent inundation. When water level rises, the people outside of the polder area as well as the fishermen have an intention to breach out the system. Such scenario was observed in 1990. This will cause permanent inundation. Due to back water effect of SLR, the water will not discharge out properly, once it enters inside the area.

Local adaptation systems on these parameters against the impacts of SLR are discussed below:

5.3.1 Houses

Three types of houses are available in beel dakatia. Most of them are “KACCHA” and “SEMI PACCUA” houses. Different types of houses are illustrated in Fig. 5.1. Resilience capacity of “KACCHA” houses against sea level rise is minimum, and their plinth level is also low. If water level rises1ft, people think that they will raise their houses at a suitable height. If water level rises 2 ft and permanent inundation occurs then local people think that, they will leave the places and shift into higher places.

Fig.10: Pictorial view of some houses in beel dakatia (Shiromoni)

5.3.2 Transport System

In beel dakatia several types of roads such as flexible pavement, rigid pavement, brick soling road and “KACCHA” roads are available. To mitigate the problems that will occur if sea level rises to 1ft or 2 ft Local people think that they will raise their roads and make small types of earthen embankments to protect the roads against sea level rise.

5.3.3 Drinking Water Sources

At present, people of beel dakatia mainly depend on deep tube well for their drinking purposes. They also use shallow tube wells and pond water for different purposes such as cooking, bathing, drinking etc. If water level rises 1 or 2 ft, local people think that they will raise their tube well base height.

6. CONCLUSIONS

Sea level rise may cause really high impacts in beel dakatia. By affecting different livelihood activities and important ecosystem, sea level rise imposes a great threat to the existence. Peoples are thinking to mitigate the effect of SLR locally, which is very limited corresponding to their needs. So, development and implementation of adaptation policies and to take initiatives for mitigation measures are the right ways to respond to sea level rise impacts.

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CHANGING CLIMATE AND SURFACE & GROUND WATER RELATED ISSUES IN DHAKA

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ABSTRACT

Dhaka being a fast growing metropolitan city has undergone severe unplanned urbanization in the last few years. Starting from river encroachment to untreated waste dumping in the river, unplanned industrialization, unauthorized extraction of groundwater, inadequate water supply, lack of adequate drainage and sewer facility and unexpected boost in Dhaka's population due to migration from suburbs and villages in search of work have a long lasting negative effect on Dhaka's future to sustain as an effective capital of Bangladesh. In addition to the challenge of attaining sustainable development, the new concern for Dhaka is the changing climate and variability in climatic parameters like temperature and rainfall. The present paper hence focuses on Dhaka's present condition in water sector considering both climate change and human induced issues in a broader aspect. Meteorological Data have been collected and analyzed to find anomalies in the historical trend of local rainfall and temperature of Dhaka. Future projections of rainfall and temperature for Dhaka in A1B scenario (IPCC SRES scenario) have also been taken into account. Ground water level data for Dhaka has been analyzed for past few years to find the trend in some specific locations. How the peripheral rivers surrounding Dhaka is being encroached and polluted and how these problems, in general, is making the present water resources of Dhaka city vulnerable to the increasing demand and changing climate, is also briefly discussed. Thus the study brings out a picture of Dhaka's scarce water resources in near future considering the present driving forces, both human induced and as an effect of climate change..

Keywords: *Dhaka, Climate Change, Temperature, Rainfall, Ground Water, Surface Water*

1. INTRODUCTION

The present water demand in Dhaka city is 2370 million liter per day. Dhaka Water Supply and Sewage Authority (DWASA) tries its best to afford sufficient water for meeting up demand of the city. By the year 2025 its population will be 21 million. At present supply deficit is 40% of the total demand of Dhaka & the deficit will be 60% by 2025. Depletion of ground water table is very alarming. It is very crucial to diversify the sources of water supplied for the city. Diminution of dependency on the groundwater and to make provisions to meet demand of the city from surface water is now very essential. Wastewater being discharged from major drains/khals contains domestic as well as industrial wastes and is ultimately disposed into the peripheral rivers. Since the peripheral river system gets no upstream inflows during dry season, the high waste loads deteriorate the water quality tremendously. A thorough assessment of water quality in the peripheral rivers is essential for their sustainability and also to rescue the city water supply system from every possible upcoming adversity. The objective of the study is to assess the sustainability of peripheral river system of Dhaka as potential surface water sources for Dhaka city water supply to cope up the increasing water demand of the city.

2. OBJECTIVE OF THE STUDY

This overall objective of the study is to analyze the present scenario of surface water and ground water condition of the Dhaka city, correlate climate change with the water sector of Dhaka, and provide a holistic understanding of Dhaka's future risk to be exposed to a more vulnerable climatic condition in a very constrained resources condition. The specific objectives are-

- Review present condition of Dhaka's water sector, both ground water and surface water
- Relate Climate Change with the present aspects of the water resources of Dhaka city.

3. LOCATION OF THE STUDY AREA

The study area is the Dhaka metropolitan area (Figure 1). The peripheral rivers surrounding the area are Turag, Balu, Tongi Khal and Buriganga. Dholai khal, Gerani khal, Segunbagicha khal, Begunbari khal, Dhanmondi lake, Ramna lake, Crescent lake and Gulshan lake, all are part of the unique wetland system of Dhaka. For rainfall and temperature analysis, the whole area was considered under the Dhaka meteorological station of BMD (Bangladesh Meteorological Department). For groundwater analysis, three areas within the metropolitan were selected-Lalbagh, Motijheel and Mirpur. The whole area was considered to find the extent surface water encroachment.

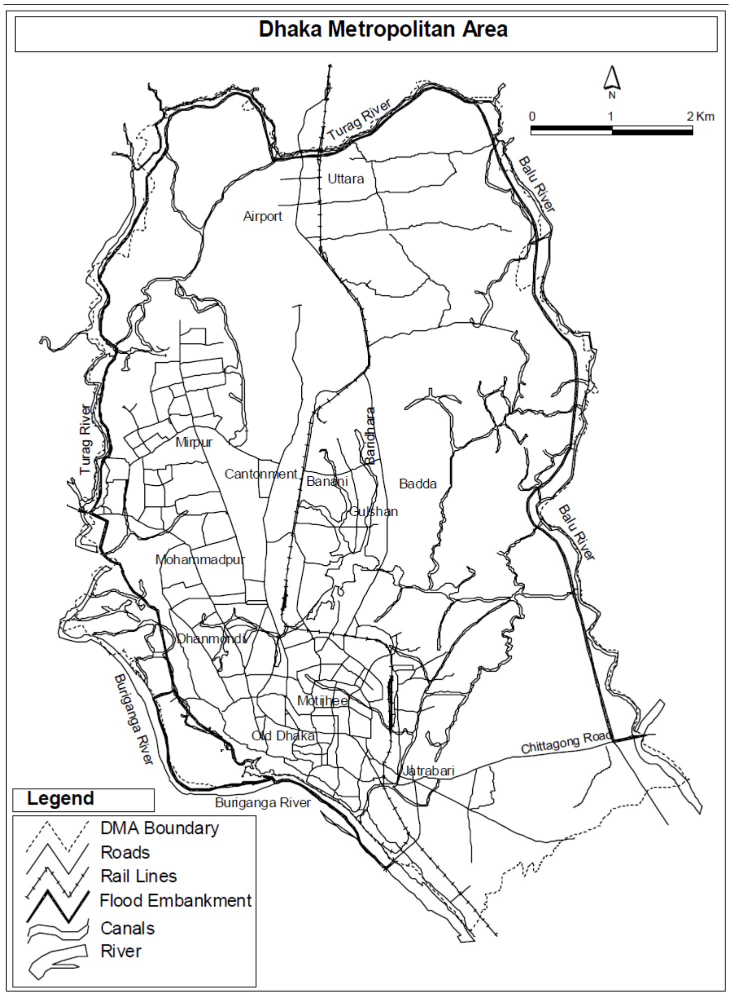


Figure 1: Map of the study area (Dhaka Metropolitan Area) Source: (Tawhid, 2004)

4. METHODOLOGY

Climatic parameters i.e. rainfall and temperature are analyzed to find historical trend. Analysis includes annual average, annual maximum and seasonal average and seasonal maximum of the parameters. The season is distributed such as December-January-February as dry period, March-April-May as premonsoon, June-July-August-September as monsoon and October-November as post monsoon. For the analyses daily data from 1961-2008 is used. Using SPSS tool the correlation value (Pearson) and statistical significance of the correlation (using two tailed test) was found out. Climatic prediction of 2020-2090 (decadal) is collected to further evaluate the future scenario in Dhaka, which was modeled by PRECIS, the regional climate model. Dhaka's present ground water scenario is evaluated by analyzing ground water data at specific points of BWDB well. The selected stations are DHA-009(Lalbagh), DHA-010(Mirpur) AND DHA-013(Motijheel), where recent abstraction of ground water has been extensive. Further focus has been given to the extent of encroachment of the wetlands in Dhaka from collected satellite image analyses of two different years, one from 1967 (Corona Space Photo) and from 2010 (Landsat5 TM Satellite Image), to visualize the change in Dhaka's wetland within 40 years timespan. Some review has also been given considering the surface water quality. Thus, Dhaka's vulnerability in an ever increasing demand of water resources is evaluated in a broader scale, considering human induced changes and climate change.

5. ANALYSIS AND RESULT

5.1 Climatic Parameter Analysis of Dhaka

Dhaka's climate in the last 50 years time span (1961-2008) has been through extensive change. Some graphical representation is given here.

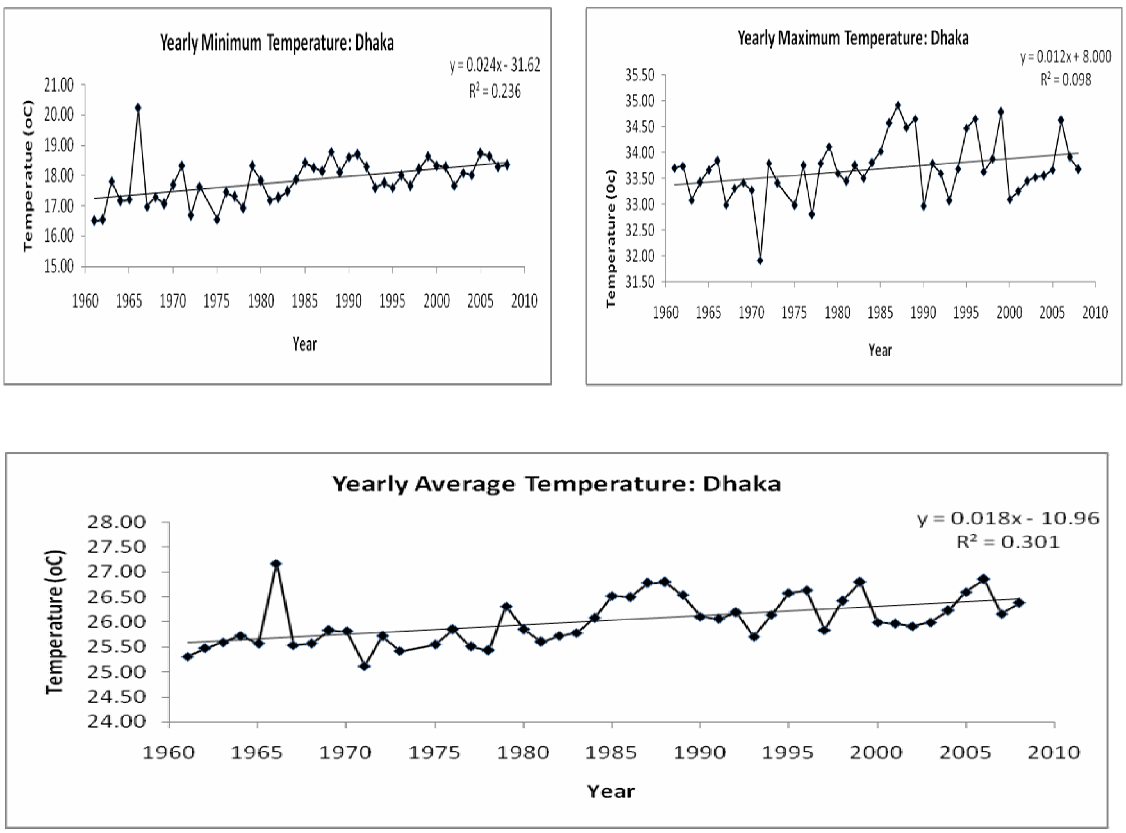


Figure 2: Changes in yearly average, maximum and minimum temperature of Dhaka (1961-2008)

The findings from this trend are shown in a tabular format in Table 1.

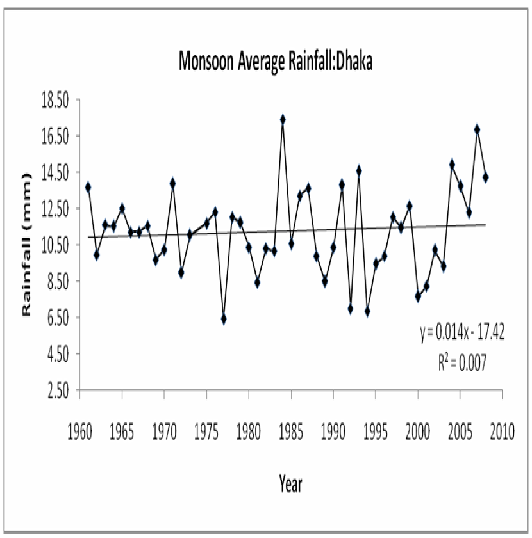
Table 1: Statistical analysis of Temperature Data: Dhaka

Parameters	**Timespan**	**Trend**	**Change per year**	**Correlation (r)**	**Significance (Two tailed Test)**
Maximum Temperature	1961-2008	Increasing	0.012 degree Celsius	0.315	Correlation is significant at 0.05 level
Minimum Temperature	1961-2008	Increasing	0.024 degree Celsius	0.486	Correlation is significant at 0.01 level
Average Temperature	1961-2008	Increasing	0.018 degree Celsius	0.547	Correlation is significant at 0.01 level

Hence it is very clear that global climate change has an impact over regional climate, which is evident from the historical trend analysis. The increase in temperature in Dhaka's climate is on an average 0.018 degree Celsius per year that is an increase of 1.8 degree Celsius in 100 years. The results are statistically significant as significance value is <0.05.

IPCC (International Panel on Climate Change) highlights that Bangladesh is warming with an increasing trend of about 1°C in May and 0.5°C in November during the 14 year period from 1985 to 1998. It also says that Rainfall has also increased - on average, decadal rain anomalies have been above long term averages since 1960s (IPCC, 2007).

From the rainfall analyses done seasonally in this study, season wise variation is seen. Seasonal average and seasonal maximum rainfall trends show an overall increase, ranging from 0.007 mm to 0.626 mm increase per year. Decreasing trend is seen in maximum premonsoon and maximum monsoon rainfall only. As rainfall is not a continuous parameter as temperature and is a discrete event, statistical significance is hard to get. But an overall idea of the historical trend in rainfall is attained.



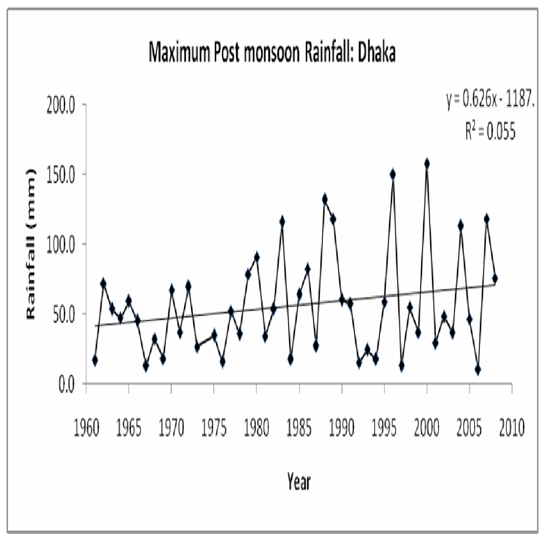


Figure 3: Trend in yearly average and maximum rainfall of Dhaka (1961-2008)

Table 2: Statistical analysis of Rainfall Data: Dhaka

Parameters	Timespan	Trend	Change per year	Correlation (r)	Significance (Two tailed Test)
Maximum Pre Monsoon Rainfall	1961-2008	Decreasing	0.131 mm	-0.096	Not significant
Maximum Monsoon Rainfall	1961-2008	Decreasing	0.097 mm	-0.059	Not significant
Maximum Post Monsoon Rainfall	1961-2008	Increasing	0.626 mm	0.236	Not significant
Maximum Dry Season Rainfall	1961-2008	Increasing	0.097 mm	0.157	Not significant
Average Pre Monsoon Rainfall	1961-2008	Increasing	0.007 mm	0.053	Not significant
Average Monsoon Rainfall	1961-2008	Increasing	0.014 mm	0.084	Not significant
Average Post Monsoon Rainfall	1961-2008	Increasing	0.042 mm	0.235	Not significant
Average Dry Season Rainfall	1961-2008	Increasing	0.007 mm	0.234	Not significant

Dhaka is the most vulnerable to climate change impacts according to a report by World Wild Fund among 11 vulnerable cities (WWF, 2009).The vulnerability is scored with a total consideration of Environmental Exposure, storm threat, sea-level rise, flooding/drought, socio-economic Sensitivity, population, assets threatened and inverse Adaptive Capacity. Dhaka scores an alarming 9 out of 10 which shows the looming future of the city in near future in a changing climate. How changing climate will affect our local climatic parameters has been analyzed in a study using PRECIS regional model in IWFM, BUET. The projection is carried out considering A1B scenario. The projected data is collected and yearly average of the projected data is done in this study to compare the historical trend with future projection.

Table 3: Future projection of temperature and rainfall Data (A1B scenario): Dhaka

Year	Pre monsoon Rainfall (mm)	monsoon Rainfall (mm)	Post monsoon Rainfall (mm)	Dry Season Rainfall (mm)	Average temperature (Degree Celsius)
1990	5.11	11.19	3.36	0.42	23.51
2000	7.91	10.09	3.31	0.71	25.48
2010	3.61	6.4	0.78	0.39	25.38
2020	12.05	6.86	1.59	0.34	26.07
2030	3.37	7.7	2.01	0.4	26.09
2040	5.48	9.61	3.43	0.25	26.92
2050	14.04	12.27	2.23	0.85	27.05
2060	20.78	7.6	0.81	1.22	28.40
2070	22.47	8.86	1.25	0.13	28.05
2080	11.9	8.09	5.73	1.49	28.81
2090	8.98	9.9	4.87	0.32	23.51

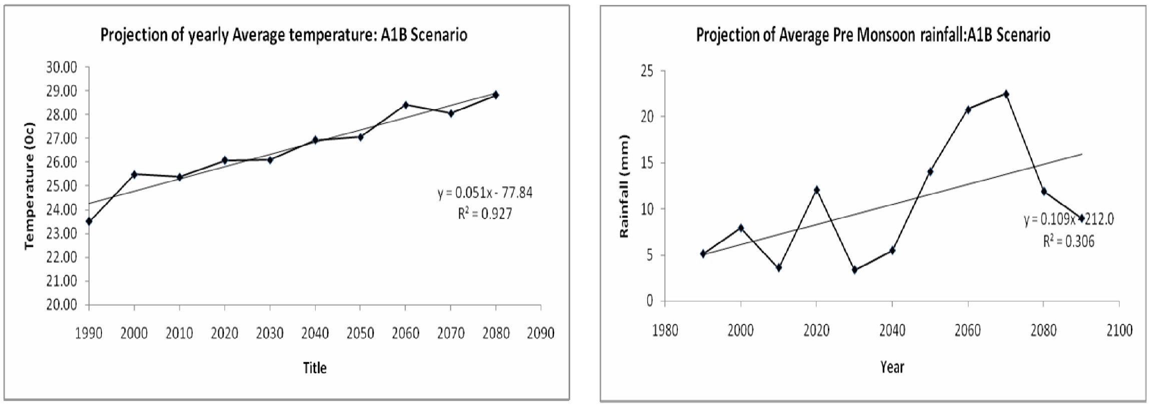


Figure 4: Projection of future average temperature and average rainfall (A1B Scenario)

It is seen from the analysis that temperature and rainfall has an increasing rate in future projection. The projection shows a variation of climatic condition within the 2050s and the 2090s.

5.2 Analysis of Groundwater Condition of Dhaka

As risk of climate change is making Dhaka vulnerable to water related hazards, population increase and increase in demand in water sector from various sectors such as industry, domestic supply, agriculture etc. creates much problem is a sustainable water allocation option. As population has increased and urbanization has occurred, surface water has been subjected to extensive pollution, both by industrial effluents and domestic wastes. Hence ground water has been a target for abstraction and the rate of abstraction has never decreased. At present, DWASA is operating 421 deep tubewells and 4 water treatment plants on the bank of the surrounding rivers. Only 18% of demand is met by these plants and the remaining 88% is met by groundwater abstraction (Akter, Ahmed, & Rasheed, 2009). As demand increases due to high population more abstraction occurs, leading to further depletion of ground water table. Recharge rate is further slowed down by lowered percolation rate due to infrastructural intervention and thick clay layer as aquiclude over the aquifer system. The concrete layer made by human and natural aquiclude is hampering natural percolation of rainfall, flood water and horizontal inflow. In this study three BWDB stations were selected to find trend of groundwater reduction. The selected stations are DHA-009(Lalbagh), DHA-010(Mirpur) AND DHA-013(Motijheel). The analyses are shown in figure 5.

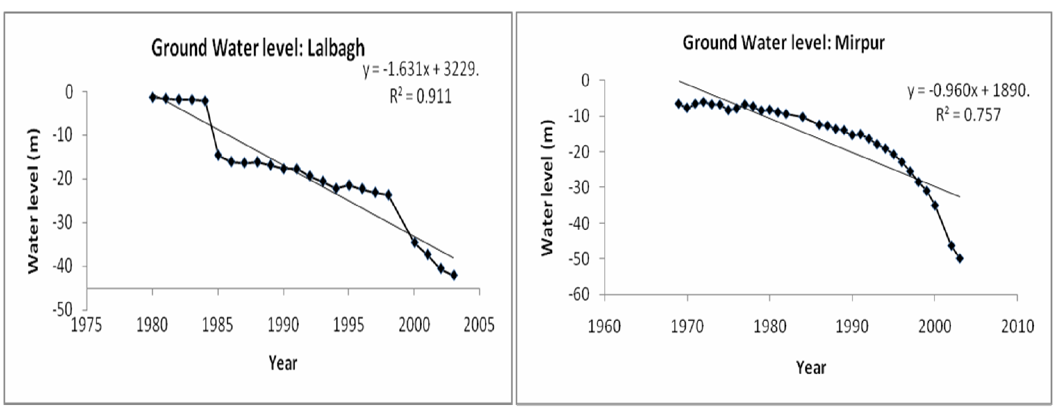


Figure 5: Trend in ground water level reduction in Dhaka city

From the analyses it is seen that ground water is reducing at an alarming rate. The reduction rate is about 1 m per year with a correlation of 0.95 out of 1 in Lalbagh thana. Mirpur also has a decreasing rate of about 1 m per year and BWDB well data of 2003 shows it has gone 60 m down from the R.L. considered which is a reduction of about 50m from the water level of 1970. Such extent of abstraction of ground water has an overall impact on Dhaka's stressed water sector and environmental sustainability.

5.3 Analysis of Surface Water Condition of Dhaka

For better understanding of Dhaka's surface water stresses at present, satellite image analysis of 1967 and 2010 of Dhaka city was collected from CEGIS. The analysis shows significant reduction in the wetland scenario of Dhaka.

Table 4: Change of wetland area in Dhaka city

Year	% of wetland area within total area	Wetland Area (Ha)
1967	38.27	20685
2010	1.37	5519

(*Source:* CEGIS, 2012)

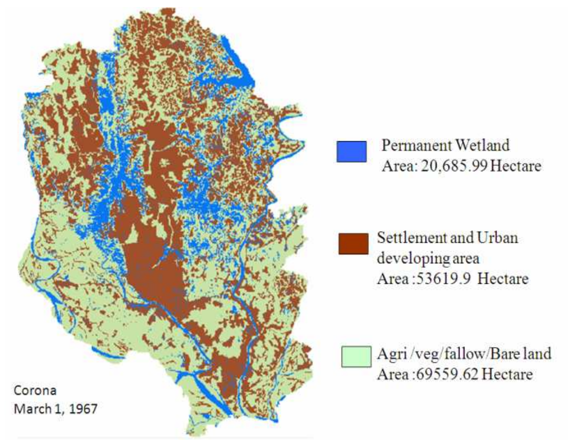


Figure 6: Permanent wetland of Dhaka city (year:1967) (*Source* :CEGIS, 2012)

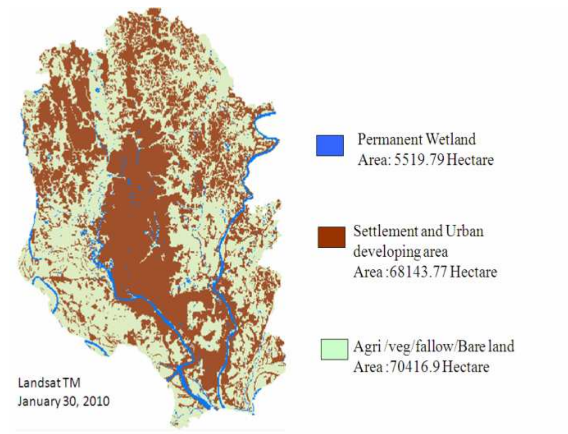


Figure 7: Permanent wetland of Dhaka city (year:2010) (*Source* :CEGIS, 2012)

In another study, the water bodies and lowlands decrease has been shown by 32.57% and 52.58%, respectively during 1960 and 2008 (Islam, Rahman, Shahabuddin, & Ahmed, 2010). Whereas this stidy, shows that the wetland percentage of the total area of Dhaka was 38.27% in 1967 and in 2010 it is only 1.37% of the total area. It is seen from the figures and data that Dhaka has lost most of its wetland. Unplanned construction of water management structure, diversion of water supply for irrigation, industry and domestic purposes reduces wetland water volume, extensive increase of population within the city might have caused this extensive changes. Hence, Buriganga, Tongi Khal, Balu , Shitlakhya and Turag, all are subjected to encroachment. Added to this, huge amount of industrial wastes flow to these rivers every day. Studies show that the level of all the water quality parameters i.e. DO (Dissolved Oxygen), BOD (Biological Oxygen Demand), COD(Chemical Oxygen Demand), Turbidity, Color and pH do not meet the standard level set by Environment Conservation Rules (ECR) 1997, of Government of Bangladesh. Especially in dry season (January to March), concentration of BOD and COD increased and DO content became low (Hasan, Kim, & Hossein, 2009).

6. CONCLUSIONS

To allow natural recharge to occur, we must now switch to surface water use and for that, we need more surface water treatment facilities. In the present context of water resources of Dhaka, stress on available drinking water will be increasing if no measure is taken to recharge the aquifers artificially. To increase surface water usage, treatment facilities must be increased and added to all industries. Strict enforcement of government laws regarding discharge of effluent in the wetlands and waste management must be done. Furthermore, if proper drainage facilities if not constructed within the city area, the increased precipitation rate due to the estimated changes to occur in near future in our climate, will increase drainage congestion as all natural wetlands are being encroached and filled for settlement and agricultural activities. In present rate of increase of population, Dhaka has a very looming future not far from today, as water is becoming the most stressed natural resources in this city surrounded by beautiful peripheral rivers, both due to human intervention and the changing climate.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

MODELLING RIVER-AQUIFER INTERACTION IN A SHALLOW AQUIFER OF SOUTHWESTERN BANGLADESH: A CASE STUDY

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ABSTRACT

The coastal belt in the southwest region of Bangladesh is facing a very difficult situation compared to other parts of the country because of the direct interaction of the saline surface water from coastal rivers and sea with the fresh groundwater in the underlying aquifers. In this study, an integrated river-aquifer model is developed for the shallow unconfined aquifer in the south-western Bangladesh to simulate the groundwater flow characteristics and dynamic flow exchanges between surface water and groundwater systems. The numerical code employed is the MODFLOW and its river package (RIV), in the framework of Groundwater Modeling System (GMS) software and is satisfactorily calibrated against the observed water levels. Simulation result demonstrates that fluctuations of hydraulic heads are dependent on seasonal variations of recharge from rainfall and riverbed leakage. In addition, the river-aquifer relationships proved to be very responsive to water table fluctuations, which indicate the losing or gaining characteristics of the river reaches. The study conclusively reveals that the developed integrated river-aquifer model opens a window for better understanding of the coupled surface water-groundwater system and thereby finding the potential causes of shallow groundwater pollution by the contaminated recharge water particularly in the coastal area of south-western Bangladesh.

Keywords: *Conceptual Model, Dynamic Interaction, GMS, Groundwater Flow, MODFLOW, River-Aquifer*

1. INTRODUCTION

Globally freshwater resources are continuously getting scarce in view of ever increasing demands due to rapid growth of population, uncontrolled urbanization and increased industrial activities. Every year some new regions, which earlier had surplus water, are entering into the domain of water scarce or water stress areas. In the past, efforts were given only on the development of additional sources to meet the rising demand without considering the quality aspect. But the most significant aspect of any water resources planning and management strategy is to ensure adequate supply of water with acceptable quality (Nobi and Das Gupta, 1997). Groundwater (GW) is usually the major source of freshwater in the coastal region because of limited potable freshwater supply from surface water (SW) bodies, where saltwater intrusion is a frequent and widespread problem (Werner and Gallagher, 2006). As rapid urbanization and industrial development continues to expand around the world's rivers and coastlines, so does the rate of unintentional release of contaminants to subsurface and/or surface waters and the need for effective assessment of such environments (Winter, 2000). In the past, groundwater and surface water (GW–SW) systems were considered as isolated components of the hydrological system, which led to the development of different approaches to modelling and management of these systems. However, according to the recent developments in hydrological research, GW-SW are increasingly being treated as part of the same system and both systems are found to be interacting in a variety of physiographic and climatic landscapes (Sophocleous, 2002; Hayashi and Rosenberry, 2002; Winter, 2001). It is now widely recognized that both GW–SW systems are inherently linked to each other (e.g. Simpson et al., 2003; Clement et al., 1996). Thus, uncontrolled development or contamination of one system obviously affects the other (Sophocleous, 2002). Therefore, river–aquifer interaction is significant in that case, where GW is contaminated by polluted SW (for example salinity intrusion) and in situations where degradation of SW is caused by discharge of saline or other inferior-quality GW from underlying aquifers. Therefore, quantitative understanding of the basic principles of dynamic interactions of GW–SW in an integrated river-aquifer system is essential for the effective management of water resources. In this regard, mathematical modelling technique plays a very significant role for evaluating the interaction process.

In any water resource planning and management, the flow exchange between the underlying aquifers and interconnected rivers is an important consideration. During the investigation of river-aquifer interaction, the practical obstacle arises from the difference in response times of GW-SW to dynamic variations in the systems. Due to this difference in time response, numerical modelling of stream-aquifer systems has typically been approached either by using a single model or a coupled model (Rodríguez et al., 2006). In case of single model approach, a groundwater model solves the surface component in a simple way. MODFLOW (McDonald and Harbaugh, 1988), a finite difference GW flow model, has been used in many engineering applications whenever the single model approach is sought (e.g. Sophocleous and Perkins, 1993). In the coupled model approach, a GW flow model is coupled internally or externally to a SW flow model. The physically based GW flow equation and the unsteady open-channel flow equations are solved either as a single time step (Pinder and Sauer, 1971) or as multiple time steps (Swain and Wexler, 1996; Nobi and Das Gupta, 1997). It is thus recognized that the simultaneous solution of the groundwater flow equation and the unsteady open-channel flow equations overcomes many of the drawbacks attributed to the single model approach. However, this approach may demand a computational effort that cannot be fully justified in cases where stream flows show smooth variations or when the objective of the study is just to determine the average system behaviour over a period of time that largely exceeds the response time of the surface component (Rodríguez et al., 2006). In those situations, MODFLOW allows simulation of river-aquifer interactions by means of alternative add-on packages. The original river package (RIV) considered constant river heads and no variation in river flows (McDonald and Harbaugh, 1988). This package was later surpassed by the STREAM package (Prudic, 1989) under the new MODFLOW 2000 structure (Harbaugh et al., 2000), which introduces a mass balance computation for river-aquifer flows to calculate the river stages. MODFLOW continues to be a commonly applied tool for approaching the diverse flow problems of GW–SW systems. Relatively little published work investigates stream-aquifer interactions in an area where the tidally dominated surface water characterized by high salinity and the water is extensively extracted from underground aquifers. Such a condition exists in the south-western coastal region of Bangladesh, where the rivers and underground aquifers are dynamically linked to each other (Nobi and Das Gupta, 1997). Since GW-SW systems are not the isolated components of the hydrologic system, but instead interact in a variety of physiographic and climatic landscapes, therefore, river-aquifer interaction is at the very core of the hydrological cycle. As the river systems in the south-western Bangladesh are already affected by salinity, the dynamic flow exchanges can greatly affect the GW system through dynamic interaction process. Therefore, an understanding of the interaction mechanism between the river and underlying aquifer is indispensable for solving the water resources problems in this region. Keeping this concept in mind, this paper attempts to evaluate the river-aquifer interaction process between the coastal river and shallow unconfined aquifer in the south-western region of Bangladesh. This paper might be a supplementary material for further research on salinity intrusion mechanism, which surely provides useful information for developing the appropriate groundwater resources management strategy in this region.

2. MATERIALS AND METHODS

2.1 Hydrological and Hydrogeological Characteristics of the Study Area

The study area is located in the south-west region of Bangladesh lying between 22.20^0 to 24.12^0 north latitudes and between 88.56^0 to 89.98^0 east longitudes (Figure 1). The area covers about a total area of about 16,985 sq. km., which is surrounded by the Ganges-Padma River in the north, the Gorai-Madhumati-Haringhata-Baleswar River system in the east, the international border between Bangladesh and India in the west and the Bay of Bengal in the south. The area is enclosed by 20 hydrological sub-catchments delineated by the Institute of Water Modelling (IWM), Bangladesh during their regional model development phase (Figure 1). The digital elevation model (DEM) developed from the public domain Shuttle Radar Topographic Mission (SRTM) database reveals that the area is dominated by flat topography. It varies from 18 m to 2 m above mean sea level (MSL) with a gradual drop from northwest to southern coastal-belt directions. At extreme south, the coastal part has very low and flat topography varying between 0 m to 2 m above MSL. The annual average rainfall is 2000 mm of which approximately 75% occurs during the monsoon season (June to September) and almost 90% occurs in the wet period (April to September). The mean annual temperature is 26 ^{0}C. The relative humidity also varies from 70% in March to 89% in July. Depending on these parameters, the average pan evaporation is also high and generally exceeds the rainfall rates in dry season (October to March). The southwest region of Bangladesh is underlain by alluvial sediments of the Bengal Basin, laid down by the Ganges-Brahmaputra River system. The sediments become progressively older with depth and lithologically range from clay and silt, to fine, medium and coarse sand, which are unconsolidated or lightly compacted. Only those sediments down to 300 m depth are of interest hydrogeologically. Previous studies reported that there are no faults or folds within these hydrogeologically significant sediments (Halcrow, 1993). In general, the upper clay layer is comparatively thick in the coastal part. In most of the areas, the composite and deep aquifer sequences are found in the 150 to 300 m depth range. The upper surface layer of mainly clay and silt characterized by high porosity but low permeability, which has poor aquifer properties and thus unsuitable for significant groundwater development. The intermediate layer has moderate to good aquifer properties and capable of producing reasonable amount of water. However, the main aquifer (lower unit) is the most important source of groundwater for irrigation, which is characterized by high porosity and moderate to high permeability and thus, it can provide large quantities of water to shallow and deep tube wells.

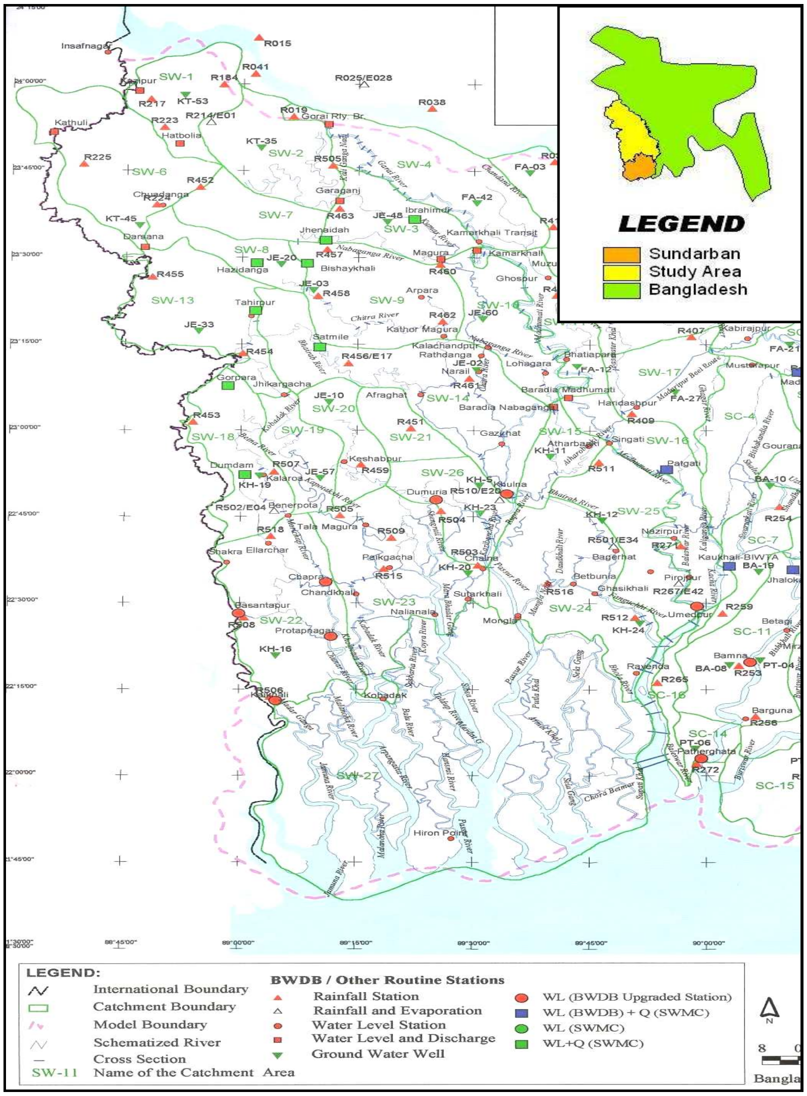

Figure 1**:** Study area showing the river systems and hydrological monitoring stations

2.2 System Conceptualization and Setting of Hydrogeological Framework

The conceptual model helps to simplify the actual field problem and to organize the associated field data so that the system can be analyzed more readily (Elkrail and Ibrahim, 2008). A total of 52 borelogs are available from the groundwater circle (GWC) of Bangladesh Water Development Board (BWDB) covering the whole study area containing individual layer lithology along the depth. This allows taking the full advantage of Groundwater Modelling System (GMS) to build a conceptual model within its software framework. GMS is a graphical user interface (GUI) for using MODFLOW along with its associated add-on packages (Harbaugh et al., 2000; McDonald and Harbaugh, 1988) and stratigraphic modelling options of the aquifer systems. It has a powerful GIS interface for use in different phases of groundwater modelling. Interpretations of individual lithology followed by semi-automatic preparation of cross-sections and automatic fence-diagram reveals that an unconfined aquifer exists up to 150 m below the ground surface characterized by two distinct aquifer layers (Figure 2). All sorts of spatially distributed data such as location of groundwater level observation well, recharge, aquifer hydraulic parameters, top and bottom surface of two aquifer layers, bottom of riverbed etc. is mapped using ArcGIS software by digitizing point data, line or polygon features. The topography of the upper layer is spatially variable from north to south direction and is assigned into the model by integrating the DEM of the study area by krigging technique. The surface topography has influence on the head pattern of the upper aquifer layer and frequently receives rainfall recharge. The hydrogeological setting of the study area is characterized by large horizontal and vertical heterogeneity. Mainly three external sources of water influence the recharge conditions of the flow system such as rainfall recharge, later inflow from west boundary and river bed leakage in some sections.

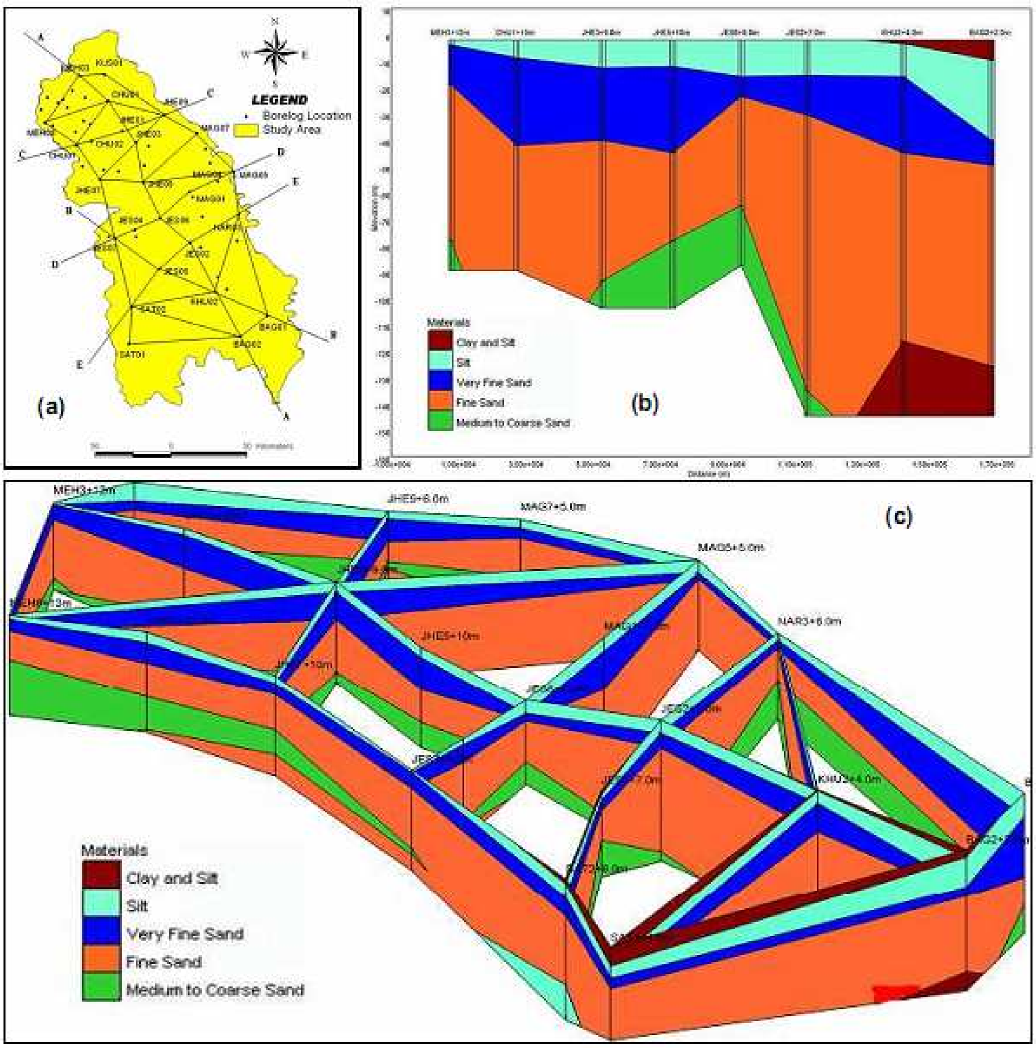

Figure 2. (a) Location of Borelogs (b) Hydrogeological Profile along N-S direction (A-A) (c) Fence Diagram

2.3 Development of Integrated River-Aquifer Model

The objective of the numerical model construction is to check and assess the validity of various interpretations regarding the flow system. These include (i) the location and type of flow-system boundaries, (ii) the location of recharge areas and (iii) variations in interpretation of hydrogeological framework. The three-dimensional groundwater flow model MODFLOW-2000 (Harbaugh et al., 2000) and its add-on river package (RIV) (McDonald and Harbaugh, 1988) are used for developing the integrated river-aquifer model in the numerical environment of GMS software framework. MODFLOW computes the hydraulic heads and cell-by-cell fluxes during the groundwater flow simulation. In order to prepare the finite-difference model grid, the aquifer system domain was divided into 232 rows and 162 columns and each cell having 1 km x 1 km grid size in the horizontal plane containing a total of 75168 cells in the two layers of the model domain. The model consists of two aquifer layers (up to 150 m depth) based on the stratigraphic analysis and simulating the principal hydrostratigraphic units presents the upper regional unconfined aquifers. The top layer (layer no. 1) is characterized by composite mixtures of clay, silt, very fine sand and fine to medium sand and lumped into one unit (0-60m). The bottom layer (layer no. 2) is represented by relatively coarser materials like fine, medium and coarse sand lumped into it (60-150m) (Figure 2). Thus, both layers are hydraulically well-connected to each other (Nobi and Das Gupta, 1997). Surface and bottom elevations of both layers are interpolated from DEM and boreholes data. The model boundaries are represented by the available hydrological features adjacent to and within the model domain (Figure 3). A time dependent specified head boundary is applied along the Ganges River in the north side and Gorai-Madhumati-Haringhata-Baleswar River systems in the eastern side of the study area using water level data of different gauging stations. In the south side, the model boundary is extended up to Hiron point to establish the sea boundary and a time dependent specified head boundary is provided using the Hiron Point water level measurement. The west boundary of the model domain is located along the international border between Bangladesh and India and a general head boundary is assigned along this side. For this purpose, the known hydraulic heads from the observed groundwater level of several monitoring wells located along the border is used. Only a coastal river named Passur (Figure 3) is incorporated in the model domain by using the River Package (RIV) of MODFLOW. This river is one of the distributaries of the Gorai-Madhumati River, which flows over the study area and finally flows to the sea. The location of Passur River in the model domain is presented in Figure 3. For integrating river with the aquifer model, the bottom elevation of the river bed is assigned from the bathymetry data of the Passur River. River is integrated in the model domain in the top layer only based on its bathymetry data. The recharge rates as in (Halcrow, 1993) were assigned on the top layer only using the MODFLOW recharge package. There was a lack of enough pumping test data in the area to be used as aquifer hydraulic property. So, the hydraulic conductivity is initially distributed on the basis of available pumping test data and lithological knowledge obtained from the collected borelogs data.

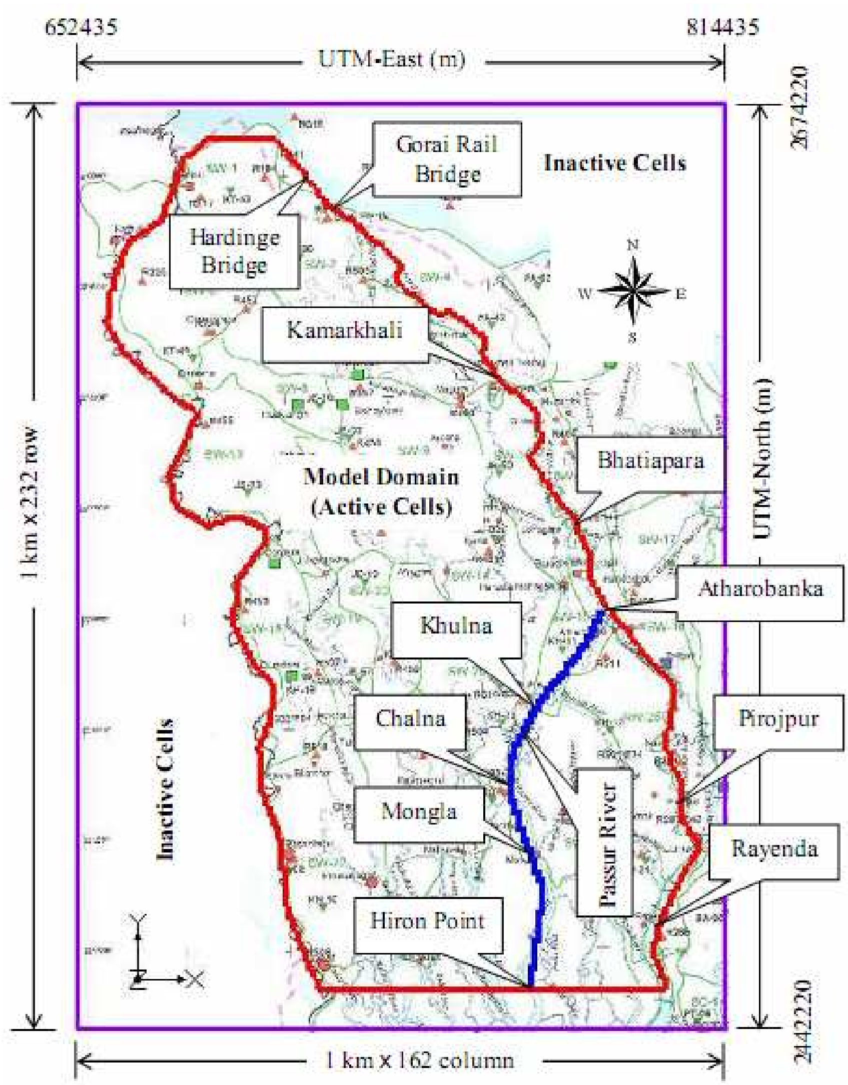

Figure 3: Location of model domain in the study area

2.4 Modelling River-Aquifer Dynamic Interaction

Assessment of dynamic flow exchanges between the river and aquifer is based on the concept that usually, the rivers and streams contribute water into the underlying aquifer system or get water from it depending on the head gradient between the rivers and the groundwater regime. The mechanism of this interaction process can be described by using Darcy's law. Assuming uniform flow between rivers and aquifer systems, the dynamic interaction can be mathematically represented by applying the concepts of Darcy's law expressed as:

$$q_n = C_n(H_n - h_n) = \frac{K_n A_n}{D_n}(H_n - h_n) = \frac{K_n W_n L_n}{D_n}(H_n - h_n) \tag{1}$$

Where,

q_n = Simulated flow rate at a single cell (L^3T^{-1}) (negative for flow out of the groundwater system)

C_n = Conductance of the bed material separating the surface water body (river) from the groundwater system (L^2T^{-1})

K_n = Hydraulic conductivity of the river bed material (LT^{-1})

D_n = Thickness of river bed material (L)

A_n = Area of water body within the finite difference cell (L^2)

W_n = Width of the river in the finite difference cell (L)

L_n = Length of the river reach in the finite difference cell (L)

h_n = Computed hydraulic head for finite difference cell n (L)

H_n = Water level in the river or surface water body (L)

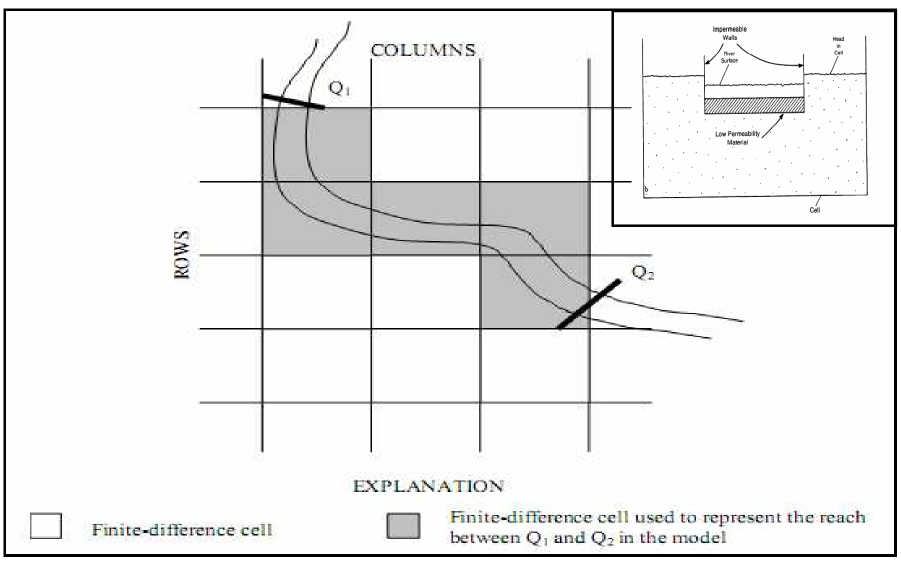

Figure 4: Conceptual representation of river reach by finite difference cells in MODFLOW

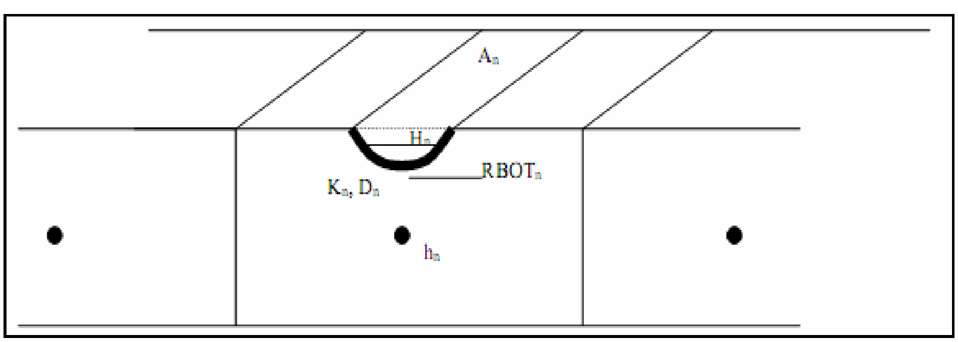

Figure 5: Basic principle of flow exchange between river and aquifer system

The dynamic flow exchange between rivers and aquifers can be computed by coupling the river (RIV) package with MODFLOW model. The conceptualization applied in river package (RIV) of MODFLOW (McDonald and Harbaugh, 2000) is presented in Figure 4. In the RIV package, flow at each finite difference cell specified is computed by using equation (1) and the calculation principle is presented in Figure 5. In the river package (RIV) of MODFLOW, mathematically the equation (1) is expressed as

$$q_n = C_n\left(H_n - h_n\right) \qquad \text{for} \qquad h_n > RBOT_n \qquad (2)$$

$$or, \qquad q_n = C_n\left(H_n - RBOT_n\right) \qquad \text{for} \qquad h_n < RBOT_n \qquad (3)$$

The river package (RIV) of MODFLOW simulates the flow exchanges for the integrated river-aquifer model, which is representing the dynamic interaction between the aquifers and rivers. Water levels in the rivers were used from the different gauging stations within it. The developed integrated river-aquifer model is first calibrated and validated against the observed water level from a total of 114 monitoring wells having 71 wells in top layer and 43 wells in bottom layer of the aquifer. At the same time, river bed conductance parameter is also calibrated. Finally, the dynamic flow exchanges between the river and the aquifer system is simulated for the whole simulation period from 1992 to 2005 and dynamic interaction is evaluated.

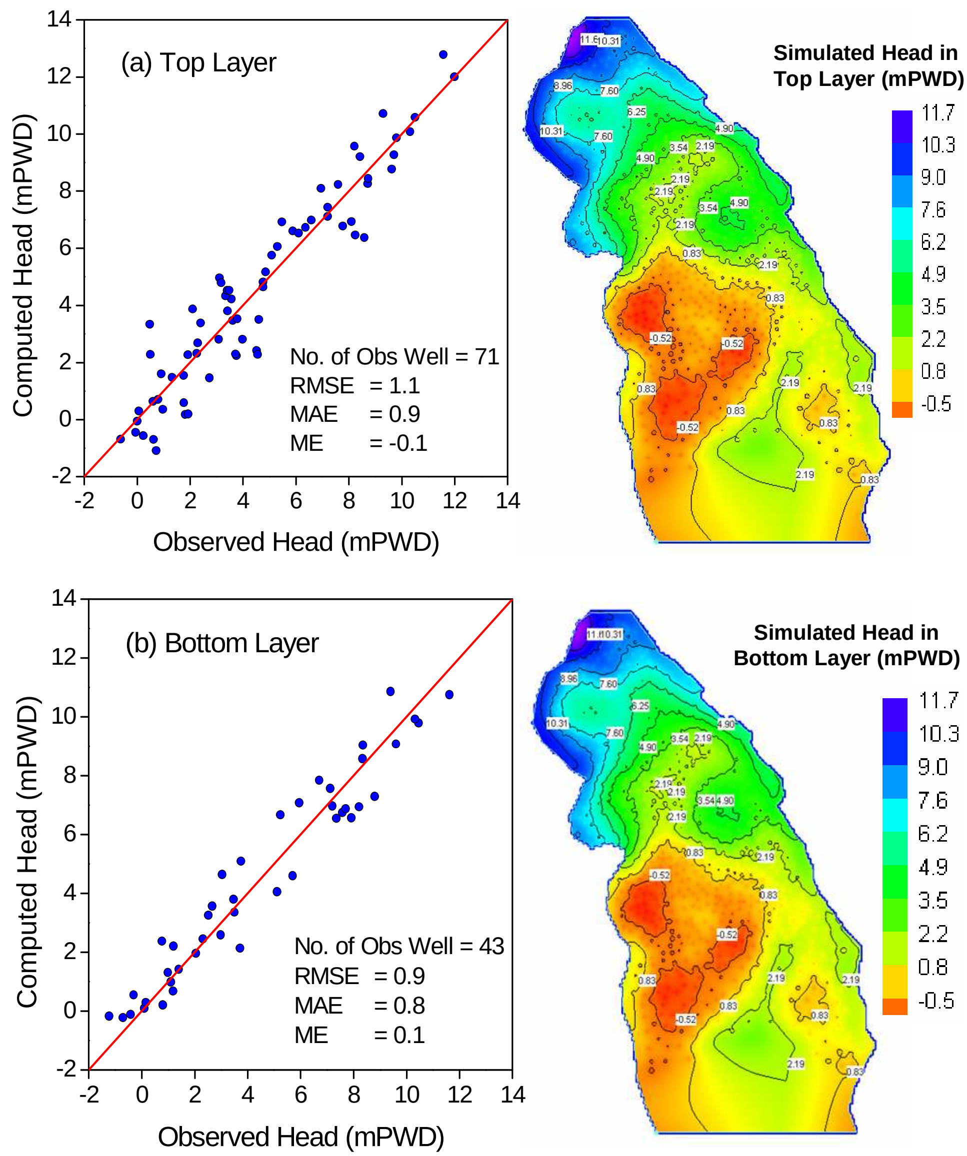

Figure 6: Simulated head distributions in the top and bottom layer in the developed numerical model

2.5 Model Calibration in Steady State Condition

Initially, the developed groundwater flow model is simulated and calibrated under steady-state condition for the average base condition in 1992. The observation data used in the calibration process was the water table elevations from observation wells. Although, steady-state condition does not occur in the practical field situation, but it gives a very initial estimates of model parameters. It also helps to check the mass balance and to check the assigned boundary conditions in the model. Steady-state simulation was run to estimate the initial head distributions to be used in the transient simulation (Anderson and Woessner, 1992). Model calibration in groundwater modelling is often required because of the unavailability of reliable field measured data to estimate the aquifer flow characteristics. During calibration, traditional trial and error technique is followed. GMS has powerful graphical user interface (GUI) with strong GIS linkages for assigning model parameters either into cell-by-cell or zonal based approach. Parameters are assigned into the model by dividing the whole model domain into a number of zones for both layers assuming similar hydrogeological properties as the cell-by-cell input is a complicated task especially in a large groundwater model. In steady-state calibration, the initial values were adjusted within a reasonable limit to obtain a good agreement of the computed and observed water table in 1992 and quantified by statistical means. The calibrated hydraulic conductivities (20 zones) vary from 9 to 14 m/day for top layer and 18 to 40 m/day for bottom layer. During model calibration, the riverbed conductance is required which has been conservatively assumed as 50% of the hydraulic conductivity of the underlying aquifer cells. It can be mentioned here that according to Yaouti et al. (2008), large values of river bed conductance can be assumed in absence of actual field data to ensure a good hydraulic connection between river and aquifer system. In this study, the same recommendation given by Yaouti et al. (2008) has been followed. To evaluate the calibration performance, scatter diagrams are prepared for each layer and a 45^0 line drawn to represent the perfect correspondence between observed and simulated values. For this model, the estimated RMSE for top and bottom layer was 1.1 m and 0.9 m respectively (Figure 6), which is very small. Once these criteria were satisfied, the model was considered calibrated. It can be mentioned here that these criteria of calibration were met with the residual mean being close to zero.

2.6 Model Calibration in Transient State Condition

The transient simulation was run to replicate the flow characteristics in the aquifer by introducing the pumping well. The discharging through the pumping well was simulated as specified flow boundary using specified pumping rate. The pumping well is considered a sink and is represented in the model by a node. Initially, model calibration is performed using the available observed water table data for a period of 1992 to 2002. The remaining data from 2003 to 2005 is used for validation purpose. The whole simulation period is divided into 28 stress periods containing two periods (dry and wet) in a year. Recharge is assumed to occur only in wet period (May to October) and no recharge is considered in dry period (November to April) based on rainfall data analysis. Each stress periods are further divided into 6 time steps with a total of 168 time steps. Thus, each time step consists of 30 days with a total simulation time of 5113 days. In transient analysis, the head distributions or aquifer response at different simulation periods under existing stresses are found out. Transient calibration is carried out using the same model parameter structures and initial head distributions computed in steady state calibration. During transient calibration, storage parameters are assigned in the same zone for both layers. The observed and simulated heads are then carefully compared and parameters are adjusted by trial and error method to obtain a good agreement between them. Initially, storage parameters are adjusted to obtain good match. Also, vertical conductivity is adjusted very little. The horizontal hydraulic conductivity was not changed. However, only one parameter is adjusted at a time because it facilitates to understand the effect produced by the change of each parameter. After modifying of parameters every time, the model is re-run. Several simulations have been carried until a good match between observed and simulated heads are obtained. From transient analysis, it is observed that without modifying the horizontal hydraulic conductivity values good match between observed and computed heads is obtained, which depicts that reasonable calibration was achieved in the transient-state. It is found that most of the observation wells show better agreement with the computed heads and the calibrated hydrograph follows the average trend of the observed hydrograph. The calibrated groundwater level (GWL) hydrograph for two representative monitoring wells are presented in Figure 7. The calibrated values of storage coefficient vary from 0.002 to 0.004 for both layers. However, there are some monitoring wells where poor agreement between observed and simulated heads is noticed. The study area constitutes a large number of small rivers and water bodies which have influence on groundwater level. But practically, it is quite impossible to incorporate all these in the model domain. It may also be due to aquifer anisotropy and heterogeneity. Sensitivity analysis shows that response of the modelled system is more sensitive to recharge variations than hydraulic conductivity and storage parameters.

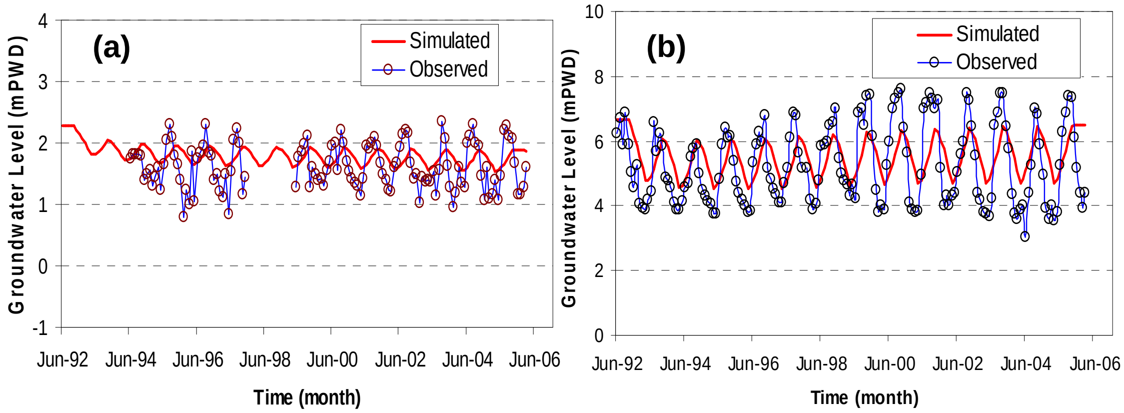

Figure 7: Simulated and observeede GWL hydrograph for (a) top layer (BAG 04) and (b) bottom layer (JHE 17)

3. RESULTS AND DISCUSSIONS

The dynamic flow exchange between river and its underlying aquifer is essential in many water resources problems. As the rivers system in the study area is already affected by the saline water, the dynamic flow exchanges can greatly influence the salinity of the groundwater system by contributing salinity through dynamic interaction. Keeping this in mind, an attempt has been made to estimate the river-interaction process in the present study. During the simulation of dynamic interaction, the riverbed conductance value is required which has been conservatively assumed as 50% of the hydraulic conductivity of the underlying aquifer cells (Nobi and Das Gupta, 1997). However, according to Yaouti et al. (2008), large values of river bed conductance can be assumed in absence of actual field data to ensure a good hydraulic connection between river and aquifer system. From the simulation result, it is observed that most of the observation wells show good agreement with the computed heads and the calibrated hydrograph (Figure 6 and Figure 7) follows the average trend of the observed hydrograph. Thus, the developed integrated river-aquifer model satisfactorily simulates the flow characteristics of the regional groundwater system.

The river-aquifer dynamic flow exchange is simulated for the whole simulation period from 1992 to 2005 and the dynamic interaction is found to be quite significant. In dry season, there is no surface runoff into the rivers and dry season flow comes into the river from the underlying aquifer. The simulated mean monthly flow exchange for the whole simulation period (14 years) are presented in Figure 8. Positive sign refers that flow is entering from the river into the aquifer and negative sign refers that the flow is contributed by the aquifer into the river system. Although some findings are qualitative, the model provides a better understanding of the coupled river-aquifer system and the potential causes of shallow groundwater pollution by contaminated recharge water in the coastal area. Modelling result also demonstrates that fluctuations of hydraulic heads are dependent on seasonal variations of recharge from rainfall and riverbed leakage. In addition, the river-aquifer relationship is proved to be very responsive to water table fluctuations, which interprets the losing or gaining characteristics of river reaches.

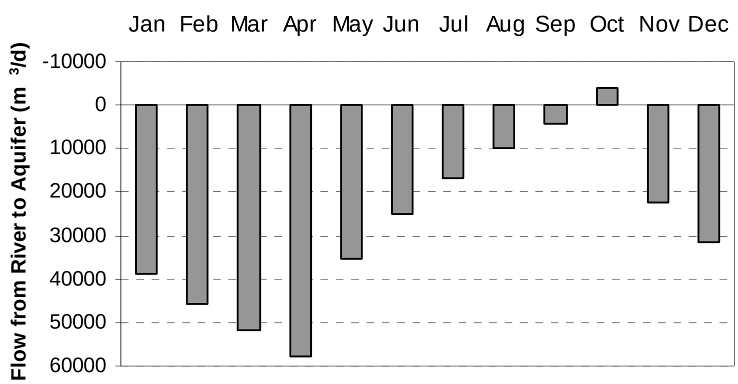

Figure 8: Mean monthly flow exchange between the Passur River and underlying aquifer

4. CONCLUSIONS

This study concludes that the methodology illustrated in the study will open a window for better understanding of the integrated river-aquifer system and the potential causes of polluting the shallow groundwater system by the saline recharge water for probable solutions particularly in the coastal area of south-western Bangladesh. The study conclusively proves that in the integrated river-aquifer systems in the coastal region, the interactions between the rivers and the underlying aquifer systems have significant influence on the flow and salinity intrusion into the overall system. The findings of this study are expected to be useful for surface water and groundwater resources management program in the coastal zone. The developed model may be used in the thrugh understanding of the salinity intrusion mechanism in an integrated river-aquifer system especially in the coastal area of Bangladesh and in further salinity intrusion assessment in future.

ACKNOWLEDGEMENTS

This research has been conducted in the first author's postgraduate study in the Water Engineering and Management field of study at Asian Institute of Technology (AIT), Bangkok, Thailand and is financially supported by the Government of the Netherlands through a fellowship grant. The authors gratefully acknowledge the Government of Netherlands for providing the financial supports as well as to AIT for providing the technical and educational supports during the study. The authors wish to thank them for their cooperation and supports in this study. Special thanks to Aquaveo, LLC., USA for providing the much expensive GMS software for this study. The authors are indebted to Bangladesh Water Development Board (BWDB) and Institute of Water Modelling (IWM), Bangladesh for their data supports.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ONSLAUGHTS ON THE RIVER BURIGANGA: FALLOUT OF POOR GOVERNANCE

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ABSTRACT

The river Buriganga is recognized to be the lifeline of Dhaka the capital city of Bangladesh. Dhaka started growing on its banks being attracted by its sweet nature of water. With the unplanned growth of Dhaka waste management sector started facing manifold problems. Majority portion of the liquid wastes and some solid wastes almost untreated ultimately found way to the surrounding rivers of Dhaka including the Buriganga. The water quality of Buriganga has now reached nearly septic condition where aquatic life can seldom exist and survive. Water quality analysis reveals the fact that both municipal and industrial wastes are the major contributor to pollution along with wastes from various other sources. Buriganga River is also onslaught by filling up part of its sides thereby restricting its usual flow. Various development activities are going on and some are on board to restore the good physical environment of the river. These activities are yet to be proved to be sustainable. On the contrary, the river itself has proven the capacity of being rejuvenated almost every year naturally. Stopping and limiting the sources of pollution depending upon its nature, preventing illegal sides filling and sludge removal from the river bed can only bring back the originality of river environment. Various activities, little or major, that are responsible for Buriganga river pollution are identified. Preventing or limiting most of those activities fall under the purview of some identified existing governance system and a few activities are due to lack of awareness and motivation. This paper emphasizes the callousness and poor performance in the concerned governance as the major cause of various onslaughts on the river Buriganga and shows the ways forward by proposing some changes and reforms in the concerned governance with a view to reviving the life of the river.

***Key words**: Pollution, Municipal wastes, Industrial effluents, Pollution control, Governance.*

1. INTRODUCTION

The Buriganga river, which is about 29 kilometer long, originating from the Dhaleshwari river reunites again with Dhaleshwari river. Dhaka, the capital city, started growing on its northern bank since Pre-Mughal period before 1610, being attracted by sweet nature of its quality. The river was an exclusive source of water for the city dwellers, used to serve as recharge areas for the city groundwater aquifer and as natural assimilator of various effluents including storm water. The tide of the Buriganga river has now become almost "dead slow". The water of the majority portion of the river in dry periods gets abnormally polluted. Increasing pollution has turned the water into a thick black fluid resembling septic condition. Long rowing or traveling across some reaches and living on and near the bank of Buriganga River are now difficult because of the stinking bad smell of the water.

Various policies, acts and rules related to the governance of the rivers directly or indirectly, predominantly for the rivers surrounding Dhaka city, including the river Buriganga, have been formulated for pollution control, regulation of development activities and control of encroachments. There have been a number of organizations in place entrusted with the responsibility of managing the rivers. However, it appears that the onslaught activities are continuing unabated mostly on the river Buriganga. The river is continuously shrinking and the quality of the water is deteriorating. This paper underlines the facts of poor performance of the concerned agencies related to governance of the Buriganga River as the root cause of its physical and environmental degradation.

2. ONSLAUGHTS ON BURIGANGA RIVER

Along with the unplanned and fast growth of Dhaka the supporting civic facilities could not grow in the proportionate pace. Ever increasing population growth in Dhaka since independence created high demand on residential, commercial and industrial establishments which are mostly developed violating concerned rules and regulations and ignoring the inadequacy of supporting services particularly the waste management system and availability of adequate land for particular projects. This phenomenon of unplanned and unhealthy development happened taking undue advantage of poor governance in the concerned authorities and absence of stringent regulatory measures. As a result environmental resources particularly the water resources are badly affected. Among the rivers around Dhaka, Buriganga river is highly abused. The onslaughts on the river are taking place in diversified following ways.

2.1 Discharging treated and untreated domestic and industrial effluents

According to the department of environment (DoE) everyday about 3,500 cubic liters of wastes from about 330 industrial units and 2,700 cubic liters of wastes from other sources are being thrown into the Buriganga. Every day, about 15,000 to 21,000 cubic liters of liquid waste, 19 tons of solid wastes and 7.5 tons of organic wastes fall into the Buriganga river from the Hazaribagh area, where there are about 500 tanneries and leather processing units. According to the Institute of Water Modelling (IWM) and the World Bank survey report in 2007, there are more than 300 various effluent discharge outlets along the rivers in the capital and Narayanganj. Of these, 19 outlets carry mixed flows of industrial and household wastes into the rivers around the capital. Over 300 major industrial units are still dumping their liquid wastes into the rivers. It is estimated that industrial effluent accounts for about 60 percent of the total pollution of watershed around Dhaka and the rest 40 per cent pollution is for municipal wastes. For Buriganga though it is not exclusively calculated but can be considered almost same as calculated for the watersheds around Dhaka.

According to Dhaka WASA, the Pagla Sewage Treatment Plant receives only about 30 per cent of the sewage generated from the city, where only BOD level is reduced to 60 ppm and then directly discharges into the river Buriganag. It is informed that COD is not addressed in that treatment plant.

2.2 Discharging wastes from activities on riverbank

Various activities are carried out alongside the banks of the river. Solid wastes from kitchen and fruit markets such as degraded vegetables, fruits and various other biodegradable wastes are thrown into the river. Cloth washing and dying, various cleansing activities, bathing etc. are carried on the banks of Buriganga River.

2.3 Drawing water from river

Though the required environmental flow for Buriganga River is yet to be ascertained, the flow at dry periods fall far below the desired level. It is found that during the dry season, the river posses a flow of only 50 cubic meters per second. Nevertheless at present through Chandnighat Water Treatment Plant, about 1.15 crore liter of treated water is being supplied to the old part of Dhaka. Drawing water from an almost zero flowing river will result in further reduction in flow.

2.4 Increasing river traffics

Innumerable steamers, launches, trawlers, mechanized boats and traditional boats ply on the river Buriganga. Due to ignorance about the environmental consequences and lack of regulatory system various solid wastes, burnt oil and excrement wastes from toilets of vessels etc. are disposed of into the river. As the number of river traffics on Buriganga River is increasing, volume of discharged wastes is also increasing.

2.5 Dumping solid wastes

Among the solid wastes, polythene bags and various other non-biodegradable solids are of great concern. Among the solid wastes those are found in riverbed comprises of polythene, glass bottles, cans etc. These types of wastes are mostly thrown from the river vessels and coming from solid wastes dumped on the riverbanks. During the dry periods, decomposition of biodegradable and organic wastes ultimately produces sludge, which settles down on the riverbed. The solid deposition in these ways causes continuous rising of the bed of Buriganga River.

2.6 Grabbing Land of Buriganga River

Bangladesh is a land hungry country. Dhaka may be called as a land hungry city. The price of land in Dhaka is towering in an unprecedented increasing rate. River section may be characterized by having relatively flat and long stretched flood plane on two sides of main deep channel in the middle. The cross section of Buriganga River is supposed to have the same feature but the present reality is that the flood plane portion of the river has already been occupied and various infrastructures have been developed. The situation is so alarming that in some locations main channel portion of the river is being occupied illegally. Major encroachment in the river occurred in the downstream areas. In this way, bottlenecks are being created in the already shrunk section of the river Buriganga and causing further restriction in its flow.

2.7 Erecting structures on rivers

Various structures predominantly temporary structures are found built on the river, near bank sides. During monsoon, these are found on river water. These are mostly unauthorized but remain standing for long in connivance with the concerned regulatory agencies. Due to these structures, the river flow is hindered and comparatively more deposition of silts occurs under and around these structures. Various liquid and solid wastes are disposed of from those establishments, which contribute in adding pollution into the river.

2.8 Lack of silt management

Buriganga River is also subjected to accumulation of heavy silts on riverbed at various locations. Near Savar the river has nearly dried up due to accumulated silt. Because of the sedimentation on the upstream, the flow of Buriganga-Turag system gets cut off, especially during the dry season. In the past the river, which was flowing in one channel in 18th century, got its second channel in the west side after a huge shoal (char) rose near Basila in the middle of 19th century. The new channel was linked with Dhaleswai and Bongshi rivers, but, in course of time, the link was lost due to reducing water flow, making the stream narrower. Taking the opportunity to the continuous shrinking of the channel, illegal encroachments at some locations have already been taken place. Bangladesh Water Development Board (BWDB) has constructed eight to ten sluice gates up to Gabtali to regulate water flow in the channel. However, those sluice gates are now out of order because of deposition of silt and earth filling in the mouth.

2.9 Lifting sand from river bed

Sometimes illegal and unauthorized extraction of sand from Buriganga riverbed happens. Permission for sand extraction is sometimes given by Bangladesh Inland Water Transport Authority (BIWTA). Generally, permission is given for extracting definite volume of sand from the middle of the river within a particular span of time. However, it is seen that the sand extraction is done in huge quantity more than the permitted and from near the bank unbalancing the section of the river rigorously. As a result, riverbanks, nearby land mass and river course might be threatened.

3. PRESENT CONDITION OF THE RIVER

The Buriganga is now a lifeless water body having no water flow from upstream. The tide of the Buriganga river has become almost “dead slow”. It is one of the main reasons of being severely contaminated. The confluence has been nearly blocked. Because of high tide a little water comes from the downstream. Between early September and May, the water turns deep purple and black and emits foul odour. Several laboratory tests have proven that the water in most parts of the Buriganga river contains no dissolved Oxygen and is therefore totally devoid of aquatic lives. Department of Environment has classified the river as "biologically dead." and declared Buriganga as Ecologically Critical Area in 2009.

The cross section of the river is continuously shrinking. This is due to rise of riverbed caused by deposition of sludge, accumulation of solid wastes and illegal encroachment of the river sides. The river now possesses a maximum of about10 meter deep sludge in some areas.

4. GOVERNMENT INITIATIVES

The Government constituted a number of task forces and committees to suggest measures to tackle the pollution and encroachment of the rivers. In 2008, task force, formed by the government, made 50 recommendations regarding preventive and curative measures and integrated action plan for the rivers around Dhaka city. In short, term measure it suggested to form a River Cleaning Authority empowering with magistracy along with execution of various activities by respective authorities concern to refrain from polluting and encroaching the river. In midterm measures, it suggested to establish sewage treatment plant at various locations, to develop road and footpath along the riverbanks. In long-term measures, it suggested to develop complete sewer system and to relocate the polluting industries.

The Bangladesh Environment Conservation (Amendment) Bill-2010 was passed in the Jatiya Sangsad in 2010, bringing changes in a number of key areas of the Bangladesh Environment Conservation Act 1995. The bill proposed for stringent action against the violators of the law. The government is going to enact a new law to control extraction of soil and sand across the country for protecting the water environment.

5. JUDICIARY INVOLVEMENT

The High Court has issued several orders, suo moto rule and given verdicts time to time in the interest of protecting the river Buriganga. The government was served a legal notice to constitute the National River Protection Commission in accordance with the High Court verdict in 2009. The High Court bench delivered the verdict detailing a set of guidelines for the government to protect and preserve the rivers including Buriganga river. The court also guided the government to make long, mid and short-term plans based on the recommendations to be worked out by the commission. The High Court ordered to remove the dirty materials deposited in the riverbeds. It also ordered for removal of all structures from the government lands on the banks of the rivers around the city including Buriganga River.

The High Court in 2009 also ordered for on-site demarcation of the river boundary by May 31, 2010, by setting up pillars according to CS map. The court ordered the government to declare the four rivers around Dhaka as Ecologically Critical Area.

The High Court ordered for relocating all tanneries in Hazaribagh by February 2010. The verdict also made it mandatory for installation of Effluent Treatment Plants for all industries, including the tanneries by June 2010. The High Court has directed BIWTA to demolish and remove its recreation centre built on the Buriganga River.

6. CONCERNED INSTITUTIONAL GOVERNANCE

Various initiatives were taken by various government agencies to prevent pollution and illegal encroachment but the outcome is yet to be proved sustainable. The reasons behind are that the scale of the problems was cumulative for long periods, lack of timely, integrated and coordinated initiatives, inexperienced initiatives were mostly unsustainable, lack of proper manpower, experience, fund etc. Unsuccessful initiatives and incapacitation reflects poor governance of the concern agencies. The initiatives reflecting poor governance towards administration and management of the Buriganga river, taken by the concerned departments have been discussed below.

6.1 Bangladesh Inland Water Transport Authority (BIWTA)

BIWTA established mainly for the management of water transport system. To maintain navigability it has to dredge the riverbeds. Handlings of dredged materials were more or less identical in most cases. BIWTA in 2004 prepared a project for the protection of the river from encroachment by erecting walls and construction walkway along the riverbank. BIWTA made another plan to remove about 3 lakh cubic metres of sludge and solid wastes from the Buriganga riverbed in three months. The rubbish extracted from the riverbed was planned to be loaded in barges and taken to Dhaka City Corporation's dumping ground at Amin Bazar. The DCC refused to take the garbage for its huge volume that might fill the ground in no time and anticipating negative impact on environment by the garbage of unknown characteristics. BIWTA ultimately dumped untreated garbage on low lands without getting no objection certificate from the DoE. They also managed to sell garbage, extracted from the Buriganga River, to four individuals. The drive of cleaning the Buriganga riverbed is doomed to fail due to inability of the bidders for lack of adequate and proper equipment and experience. BIWTA could not realize that management of sediment, full of garbage mixed with sludge having presence of toxic elements, would be a problematic. In fact, cleaning and management of such dredged materials needed in depth study and consultation with concerned authorities prior to formulating the project, which BIWTA overlooked. Furthermore, due to not taking all out measures for preventing pollution; this project would not be sustainable. Besides their normal jobs BIWTA had to pull down part of total illegal structures built on river bank sites.

6.2 Dhaka District Commissioner’s (DC) office

Dhaka DC office does various innumerable activities including land administration according to the Cadastral Survey (CS) 1912 or Revenue Survey (RS) 1966. DC office time to time issued notice to the owners of the illegal structures to remove their structures from the bank of the Buriganga river but it got virtually no response. Dhaka DC office could not properly demarcate and maintain the original boundary of the Buriganaga river free from illegal encroachment. Taking the opportunity of such institutional deficiency the original flood plains of the river were occupied and are continuing.

6.3 Bangladesh Water Development Board (BWDB)

BWDB has taken up an initiative to give a new lease of life to the Buriganga river by conveying water from the Jamuna river. The objective was to augment the Buriganga river flow in dry season to maintain minimum flow, improve water quality significantly and to improve overall environment, economic and social condition. Four routs had been studied for the purpose. Finally, the New Dhaleswari-Pungli-Bangshi-Turag-Buriganga river rout has been chosen. The project was supposed to be completed by 2013. Afterward it was apprehended that the project would not be completed before 2015 owing to delay in disbursement of funds, lack of adequate dredgers and non-participation of bidders in the tender process.

6.4 Dhaka Water Supply Authority (WASA)

Dhaka WASA was supposed to maintain sanitation through developing comprehensive sewage management system along with ensuring potable water supply to the city dwellers. Dhaka WASA so long could develop sanitary sewer for 30 per cent of DCC area and yet to develop the complete sewer system-covering remaining parts of Dhaka. The rest 70 per cent non-sewer area shall have to incorporate septic tank system, which is seldom monitored or regulated by WASA. Dhaka WASA planned for setting five new sewage treatment plants at five places around Dhaka. These plants are yet to be materialized. Regarding monitoring of the quality of supplied water WASA has limitation in testing all the parameters required excepting color, acidity, turbidity, chlorine and dissolved solids.

6.5 The Dhaka City Corporation (DCC)

DCC is responsible for solid waste management in Dhaka city along with many other various diversified activities. About 20 percent of municipal wastes remain unmanaged every day. Major portion of unmanaged solid wastes near riverbanks ultimately goes in some ways or other into the river. DCC is also running with limited manpower and finds it difficult to check dumping both industrial and domestic wastes into the river. For years, the DCC men have been dumping rubbish along the embankment every day and could not remove those timely due to shortage of transporting vehicles and various limitations of dumping ground.

6.6 Rajdhani Unnayan Kartipakkha (RAJUK)

RAJUK is the authority who gives permission for building construction and takes preventive and punitive measures in case of unauthorized developments. It also monitors development of buildings. This authority could not identify the river land and thus control the unauthorized development along the Buriganga riversides.

6.7 Bangladesh Small and Cotage Industries Corporation (BSCIC)

BSCIC is responsible mainly for development of small and cottage industries. It was decided to relocate the tanneries from Hazaribagh, Dhaka to Hemayetpur, Savar. The project was inaugurated in 2005 and was supposed to be completed in 2006. The project is yet to be implemented by BSCIC. Establishing the common effluent treatment plant and paying compensation to the evicted people were found to be the major unresolved issues behind the delay in implementing the project.

6.8 Department of Environment (DoE)

DoE has mandate for conservation of environmental quality in all respect. This department formulated Environmental Impact Assessment guidelines for addressing and managing the interventions through the development projects to ensure the sustainability of the environment. DoE is running short of manpower and modern testing laboratory. Due to lack of necessary equipment, they could not verify the presence of toxic materials in the garbage of the Buriganga riverbed. DoE failed to ensure the prevention of discharging untreated and partially treated wastewaters from the polluting industries. Virtually DoE mostly run with limited cadre strength under different cadre leadership. It can hardly maintain routine and regular task of monitoring and regulating environmental quality throughout the country.

7. RECOMMENDATIONS

In the country, the rivers falling in particular district are supposed to be under the administration of respective office of the District Commissioner's. Rivers flowing inter districts are then subject to several administrations which cannot realize and address the physical and environmental needs throughout that river. Therefore, the rivers should be under one administrative system that will look into the interest of three resources water, sediments and aquatic lives of all the rivers. The authority may be designated as National Water Authority, which will administer and look into the interest of all surface waters and underground waters within the geographical including sea boundary of the country. An existing department working mostly on water resource development can be upgraded to such authority through shouldering various other jobs, which are now being done by different agencies.

There is an instruction from the High Court to establish a River Commission. It may be recommended to form the commission in a bigger perspective to make it responsible for development, management and sharing the water use amongst the stakeholders from all kinds of water resources and the commission may be named as Water Commission.

8. CONCLUDING REMARKS

The river Buriganga is said to be lifeline for Dhaka city, which is declared biologically dead. This death was caused not naturally but it is a homicide caused by some unauthorized or illegal acts of individuals or agencies being not monitored or regulated by the concerned authorities supposed to. The authorities concerns were almost non-responsive regarding the consequences of those unethical acts upon the life of the Buriganga river. The responsible authorities hardly endorse their incapability and deficiency in this respect and so did not feel to take any protective or curative measures in time. The measures so far taken are not proved to be sustainable because of their inexperienced and new initiatives of ad hoc and provisional natures. Those initiatives rather surfaced about the poor governance of the initiators. When all the concern authorities will function properly satisfying various needs of an authority owning the rivers, might save many rivers, including the river Buriganga, from dying. National Water Authority might be that authority for saving and protecting all the rivers and above all there should be a Water Commission as a supervisory apex body.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SHIFTING OF RAINFALL INTENSITY DURATION FREEQUENCY (IDF) CURVES OF DHAKA CITY

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ABSTRACT

Short duration rainfall is representative of the intensity of storm because the most intense storm last for very short time. The rainfall Intensity Duration Frequency (IDF) curves represent the essential characteristics of point rainfall for shorter durations. The present study was under taken with a broader scope and was aimed to develop IDF curves for Dhaka City for different time series, and to observe shifting of these curves. In this regard, shorter duration rainfall data for a long time scale (from 1953 to 2008) for Dhaka City was analyzed. Daily and 3-hr rainfall data for Dhaka City were collected from Bangladesh Meteorological Department (BMD) but shorter duration data like 5 min, 10 min, 15 min and 30 min were not available. The ratio method was used to obtain shorter duration data series. The generated short duration data were analyzed using Gumbel extreme value distribution method and the rainfall intensities for different return periods were obtained. Two IDF curves were plotted using these values, IDF curve-1, for the duration of 1953 to 1980 and IDF curve-2, for the duration of 1981 to 2008. Shifting of recent curves from previous curves was observed. IDF curve for the recent years (IDF -2) lies below than that of previous years (IDF-1) which indicates the decrease in intensity of rainfall for the same duration and return period. Climate change may be the factor for this shifting of IDF curves. In practical terms the developed IDF curves can be used to better manage Dhaka City storm sewer network that are vulnerable to water logging problems almost every year in rainy season.

Keywords: *Intensity, frequency, duration, climate change, shifting*

1. INTRODUCTION

Bangladesh has a subtropical monsoon climate characterized by wide seasonal variations in rainfall, moderately warm temperatures, and high humidity. Three seasons are generally recognized: summer from March to June which is hot and humid; a rainy monsoon season from June to October; and a cool, dry winter from October to March. Most places of the country receive more than 1,525 mm of rain in a year, and areas near the hills receive about 5,080 mm. Rainfall of the rainy season accounts for 70 to 85 percent of the annual total. The general observation was that the most intense storm last for very short durations. As the duration of storm increases, the maximum average intensity of storm decreases. Intensity duration frequency (IDF) analysis is used to capture the essential characteristics of point rainfall for shorter durations.

Variables used in extreme event analyses such as annual maximum rainfall intensities for short durations form the basis of important decisions in engineering practice (NERC, 1975) and urban planning. The objective being to develop Intensity duration frequency (IDF) curve using Extreme Value type 1 (EV 1) probability distribution known as Gumbel distribution, It is commonly assumed in all such analyses that the sequence of observed annual maxima (minima) is stationary, with each observation in the sequence drawn from an underlying EV distribution that remains constant over time. But if changes in climate (Vinnikov et al., 1990; Guttman et al., 1992; Groisman and Easterling, 1994; Changnon and Kunkel, 1995; Olsen et al., 1999; Douglas et al., 2000), or in land use (Bruijnzeel, 1990, 1996; Sahin and Hall, 1996), result in gradual changes in the observed sequence, the assumption that the underlying EV distribution remains constant becomes invalid. Furthermore, the report "Climate Change 2001: Impacts, Adaptation and Vulnerability" by the Intergovernmental Panel for Climate Change (IPCC, 2001) warns of climate change over the next century, envisaging "changes in the variability of climate, and changes in the frequency and intensity of some climate phenomena "

The spatial and temporal availability of water resources must be expected to change when regional climate changes. The present study was under taken with a broader scope and was aimed to develop IDF curves for Dhaka city for different time series, and to observe shifting of these curves.

2. METHODOLOGY

The spatial and temporal availability of water resources must be expected to change when regional climate changes. The present study was under taken with a broader scope and was aimed to develop IDF curves for Dhaka City for different time series, and to observe shifting of these curves. To plot IDF curves rainfall data recorded by rain gauges are needed. In many countries it is not easy to obtain a long enough series of data for short duration rainfall and, like in Bangladesh, this kind of information may be not available at all. Daily and 3-hr rainfall data for Dhaka City were collected from Bangladesh Meteorological Department (BMD) but shorter duration data like 5 min, 10 min, 15 min and 30 min were not available for Dhaka City. From daily rainfall (24-hr rainfall data from BMD records), 1-hr rainfall data was generated using the ratio 0.34 by Murry (1970) and to obtained the required 5 min, 10 min, 15 min and 30 min short durations rainfall data, the 1-hr rainfall data were multiplied by the factors of 0.32, 0.45, 0.55, and 0.71 respectively by Bell (1969).

2.1 Rainfall Intensity Calculation:

The Gumbel or Extreme Value Type1 distribution has been used extensively in hydrology to model flood flows and extreme rainfall depths by Chow et al., and Stedinger et al (1993)., Maxima were calculated for rainfall durations of 5, 10, 15, 30, and 60 minutes. The maxima have been fitted to the Extreme Value Type1 or Gumbel distributions. The Gumbel distribution is defined by the following equation;

$$P(y) = \exp(-\exp(y)) \tag{1}$$

$$y = \alpha(x-u) \tag{2}$$

With x the variable considered, P its distribution, y the Gumbel's reduced variable (same unit as x), α (-) the scale parameter and u (same unit as x) the location parameter. The parameters u and α were estimated using the method of moments by Stedinger et al (1993).

$$\alpha = \frac{\pi}{\sigma\sqrt{6}} \tag{3}$$

$$u = \mu - \frac{\gamma}{\alpha} \tag{4}$$

Where μ (mm) is the average of the series, σ (mm) its standard deviation, and γ the Euler constant (0.577215...).

The rainfall depth d (T) (mm/h) for each return period T (year) are calculated by the following formula:

$$d(T) = u + \frac{y}{a} \tag{5}$$

$$y = -\ln(-\ln(1-\frac{1}{T})) \tag{6}$$

Where *u*, a and *y* were defined previously. Then intensity of rainfall r (T) for each return period T (year) are calculated in mm/hr.

$$r(T) = d(T) \times \frac{60}{t} \tag{7}$$

Here t is the duration of rainfall in minute.

2.2 Standard Error Calculation:

Standard error for Extreme Value Type 1 distribution can be written as

$$S_e = \frac{1}{n}(1+1.1396K_t+1.1K_t^2)]^{0.5}\sigma \tag{8}$$

Where, K_t is called frequency factor which is a function of return period and the type of probability distribution to be used in analysis. Expression for K_t for Gumbel distribution, Chow (1953).

$$K_t = -\frac{\sqrt{6}}{\pi}\left\{0.577215 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\} \quad (9)$$

2.3 Plotting of IDF Curves:

Two basic IDF curves were plotted from obtained data series (table 1 and table 2). Figure 1 for IDF curve-1 and figure 2 for IDF curve-2. IDF curve-1 was obtained by plotting the rainfall duration (min) in X-axis and rainfall intensity (mm/hr) in Y-axis for different return period for the duration of 1953 to 1980 in a log-log paper. Similarly IDF curve-2 was plotted for the duration of 1981 to 2008 in a log-log paper. Table 3 and table 4 shows the standered error calculation for this data series.

Table 1: Rainfall intensity for different duration and return period (for 1953 to 1980)

Duration (min)	μ	σ	Rainfall intensity (mm/hr)					
			r(T2)	r(T5)	r(T10)	r(T25)	r(T50)	r(T100)
5	16.4678	6.2941	185	252	296	352	393	434
10	23.153	8.842	130	177	208	247	277	305
15	28.2964	10.8135	106	144	170	202	225	249
30	36.532	13.9545	68	93	109	130	145	161

Table 2: Rainfall intensity for different duration and return period (for 1981 to 2008)

Duration (min)	μ	σ	Rainfall intensity (mm/hr)					
			r(T2)	r(T5)	r(T10)	r(T25)	r(T50)	r(T100)
5	14.532	5.8326	163	225	266	317	356	394
10	20.4428	8.1991	115	158	187	223	250	277
15	24.9821	10.0271	93	129	152	182	204	226
30	32.25	12.94216	60	83	98	117	132	146

Table 3: Standard error for 1953-1980

Duration (min)	Standard error					
	2 year	5 year	10 year	25 year	50 year	100 year
5	1.09	1.84	2.48	3.35	4.01	4.67
10	1.53	2.58	3.49	4.70	5.63	6.56
15	1.88	3.16	4.27	5.75	6.88	8.02
30	2.42	4.08	5.51	7.42	8.88	10.35

Table 4: Standard error for 1981-2008

Duration (min)	Standard error					
	2 year	5 year	10 year	25 year	50 year	100 year
5	1.01	1.70	2.30	3.10	3.71	4.33
10	1.42	2.40	3.24	4.36	5.22	6.08
15	1.74	2.93	3.96	5.33	6.38	7.44
30	2.25	3.78	5.11	6.89	8.24	9.60

3. RESULTS AND DISSCUSSIONS

Two basic IDF curves were plotted from obtained data series, Fig. 1 for IDF curve-1 and Fig. 2 for IDF curve-2. IDF curve-1 was obtained by plotting the rainfall duration (min) in X-axis and rainfall intensity (mm/hr) in Y-axis for different return period for the duration of 1953 to 1980 in a log-log paper. Similarly IDF curve-2 was plotted for the duration of 1981 to 2008 in a log-log paper. From these two IDF curves, it was observed that for the same return period and duration the rainfall intensity decreased than that of previous one for Dhaka City. Say from the IDF -1 for 5 minute duration and 100 year return period rainfall intensity is 434 mm/hr but from IDF-2 it is 394 mm/hr. Similarly all the values of IDF-2 were smaller than that of IDF-1. Climate change may be the factor for this shifting of IDF curves but analysis of data from different parts of Bangladesh required to prove this argument.

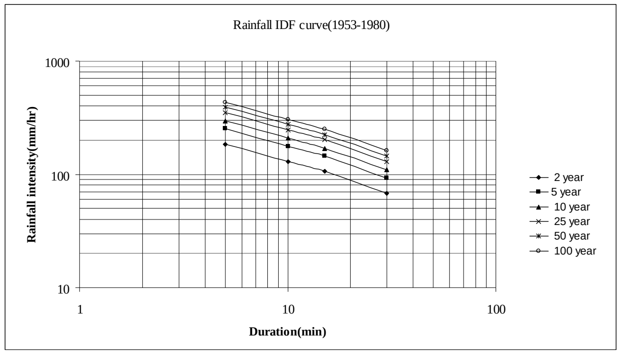

Figure1: IDF curve-1 (for duration 1953 to 1980)

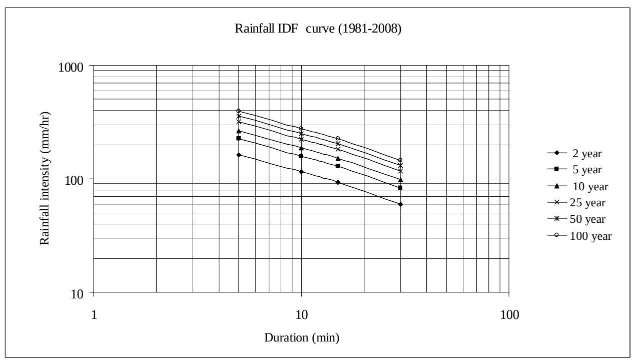

Figure2: IDF curve-2 (for duration 1981 to 2008)

4. CONCLUSIONS

Two IDF curves were developed here for different return period for the duration of 1953 to 1980 and 1981 to 2008. If both of the curves are superimposed it is observed that the IDF curve for the recent years (IDF -2) lies below than that of previous years (IDF-1) which indicates the decrease in intensity of rainfall for the same duration and return period. Results can be used as input to hydrological modeling but also to better manage the runoff characteristic of Dhaka City. This may facilitate the objective of monitoring the climate change in terms of rainfall.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ESTIMATING SHALLOW GROUNDWATER RECHARGE FROM PRECIPITATION AND WATER LEVEL OBSERVATIONS

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ABSTRACT

The quantity of precipitation that reaches the zone of saturation depends upon several factors. Because of these factors, determining the recharge to aquifers is a complicated task in all groundwater resources studies. In addition, most of the available models applied for estimating recharge requires data that are not readily available in developing countries and the cost of using such models may sometimes be prohibitively high. This study presents a data conservative approach, in which details of quantitative groundwater recharge estimation in the shallow unconfined aquifer is interpreted by the analysis of observed precipitation and water level fluctuations. Kushtia district of Bangladesh has been taken as a case study area based on observed data and information. In this study, well-known water table fluctuation technique has been modified to estimate shallow groundwater recharge so that groundwater recharge can be estimated using the shortest data available in hand. Observed monthly precipitation and weekly groundwater levels at a few observation wells in the study area are the only data required to carry out the study. The approach illustrated in this paper can be useful in any initial level of assessment in groundwater studies. In addition, results can be applied as input data for developing numerical groundwater model for any groundwater resource investigation in the study area or similar area in Bangladesh.

Keywords: *Aquifer, Groundwater recharge, Kushtia, Precipitation, Groundwater level, Water table fluctuation*

1. INTRODUCTION

Groundwater (GW) recharge is a critical hydrological parameter, which may need to be estimated at a variety of spatial and temporal scales depending on the application. It is important to quantify the water fluxes from the atmosphere to underlying aquifers for estimating the global water budgets. Aquifer-scale recharge estimates are often essential to water resource assessment and management, whereas local scale recharge is critical to assessment of GW contamination from point sources. Estimation of recharge may be required on temporal scales ranging from days to thousands of years. As aquifers are depleted, recharge estimates have become more essential in determining appropriate levels of GW withdrawal. In addition, estimation of recharge is becoming more important for contaminant transport, as aquifer management expands from cleanup of existing contamination to aquifer protection by delineation of areas of high recharge (Scanlon and Cook, 2002). Thus, understanding GW recharge and its accurate estimation is essential for the successful management of water resources and modelling fluid flow and transport of contaminants within the subsurface (Healy, 2010; Healy and Cook, 2002). The quantity of precipitation that reaches the zone of saturation depends upon several factors. Because of these factors, determining the GW recharge to aquifers is a great challenge in all GW studies (Korkmaz, 1988). In addition, most of the available models applied for estimating GW recharge requires data and information, which are not readily available in developing countries and the cost of using such models may sometimes be prohibitively very high (Das Gupta and Paudyal, 1988). Increasing demand for recharge estimates is forcing the researchers to develop approaches for building a more thorough understanding of recharge process and quantifying recharge rates that reduce uncertainties and increase confidence in recharge estimates (Scanlon and Cook, 2002). Many different methods exist for estimating GW recharge (Scanlon et al., 2002). However, methodologies related to the applications of well-bore hydrograph analysis for inferring GW recharge are reviewed by Healy and Cook (2002). Most often, these approaches requires much time and skill. In addition, necessary data and information are not available in hand to carry out the study. Therefore, simple and straightforward method for estimating GW recharge is often required in any GW study, where fewer amount of data is available in hand. The water-table fluctuation method may be the most widely used technique for estimating GW recharge, which requires knowledge of specific yield and changes in water levels over time. Advantages of this approach include its simplicity and insensitivity to the mechanism by which water moves through the unsaturated zone (Healy and Cook, 2002). In this paper, a data conservative modified water-table fluctuation approach is presented, in which quantitative estimation of GW recharge in the shallow unconfined aquifer in Kushtia district of Bangladesh is interpreted by analyzing the precipitation and water level data. It is expected that the methodology described in this paper can be applied as preliminary input parameters for developing and simulating a GW model in the numerical environment for any GW resource investigation scheme in the study area or similar area of Bangladesh.

2. METHODOLOGY

2.1 System Conceptualization

Records of water level fluctuations in monitoring wells are worth the cost and trouble of collecting only if they are used as a basis for hydrologic interpretations. Although water level records have been important to the reaching of conclusions regarding the occurrence and development of GW in specific areas, many such records still await interpretations. Similarly, a wealth of climatologic and other hydrologic data is in need of analysis (Korkmaz, 1988). In the wet period, when precipitation occurs, the first rain wets vegetation and other surfaces and then begins to infiltrate into the ground. The first infiltration replaces soil moisture, and thereafter, the excess percolates slowly across the intermediate zone of saturation moves downward towards aquifer storage and consequently, the water table goes up. In the dry period, when evapotranspiration rates and GW withdrawal exceed the available moisture from precipitation, recharge to the water table is negligible and thus, GW levels decline (Figure 1).

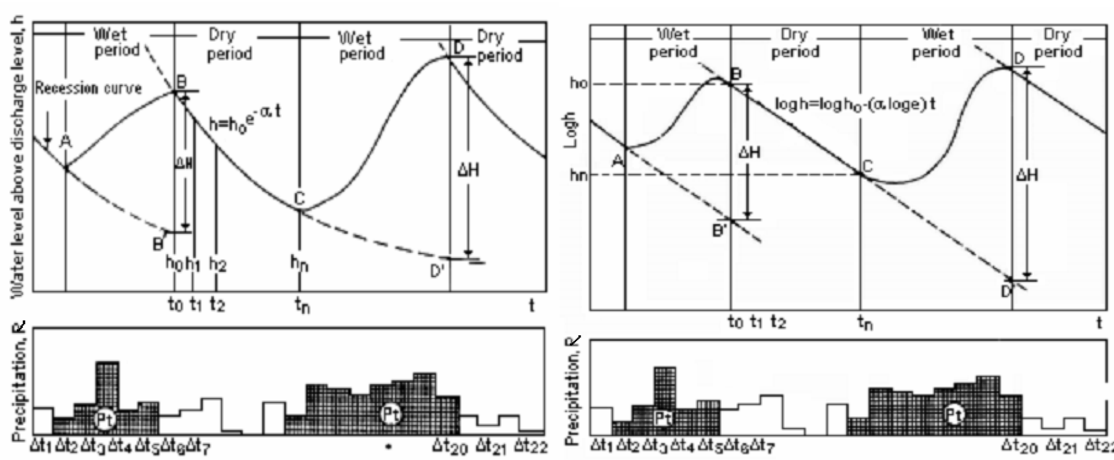

Figure 1**:** Fluctuations in water levels caused by recharge from precipitation (Adapted from Korkmaz, 1988)

In general case, it is possible to calculate the total level oscillation amplitude due to infiltration if the overall recession regime (i.e. the behavior of the aquifer without external recharge) is known. It is generally found that the component forming a GW hydrograph having a recession can be approximated by simple exponential relationship of the form (Figure 1) as:

$$h = h_0 e^{-at} \quad (1)$$

Where, *h* and h_0 are the water level above discharge level at the beginning of the measurement period and after a certain time (*t*) respectively and *a* is known as coefficient of recession or discharge coefficient.

Like any other exponential formula, on a semilogarithmic paper when water level above discharge level is plotted to the log scale and time to the arithmatic scale, the recession curves plot as nearly as straight lines (Figure 1). In the log system with base 10, the formula is converted into the following form:

$$\log h = \log h_0 - 0.43429at \quad (2)$$

The shape of the recession curve depend on the water yielding properties of the aquifer material, the transmissivity and the geometry (Johansson, 1987).

The recovery of the water level, ΔH under natural hydrological conditions is a mirror image of the recession curve. The recovery of the water level varies from year to year, depending on the amount of total precipitation (R_t) in wet period (Figure 1). GW levels are influenced by seasonal cycles in such factors as recharge from precipitation, evapotranspiration and discharge from wells and show a seasonal pattern of fluctuations (Healy, 2010). The degree of correlation between fluctuations of GW level and fluctuation of total precipitation (R_t) in wet period provides a clue as to the freeness of the connection between recharge and total precipitation (R_t) in wet period (Korkmaz, 1988). In this study, the direct estimation of GW recharge in shallow aquifer is considered using recovery of the GW level (ΔH) and total precipitation (R_t) during wet period (Figure 2). The line of regression equation can be expressed as:

$$\Delta H = a + bR_t \tag{3}$$

Where, ΔH is recovery of GW level, and R_t is total precipitation during the wet period, a and b are the regression coefficients.

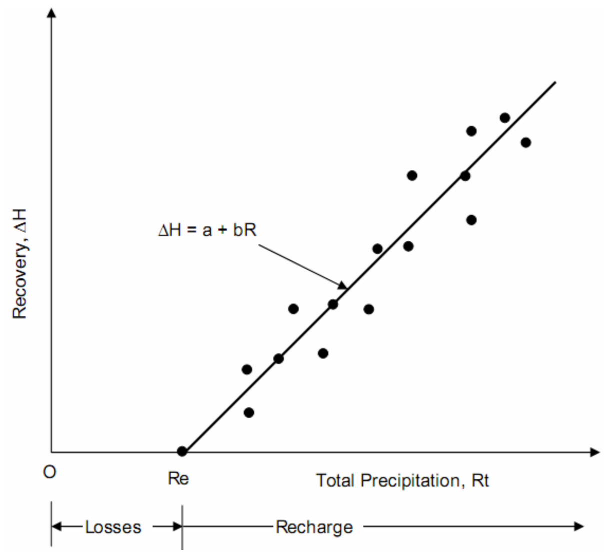

Figure 2**:** Relationship of total precipitation in wet period and recovery of the aquifer water level

Precipitation intercept, R_e is the intersection of the total precipitation recovery straight line with zero-total precipitation axis (Figure 2). It represents the amount of surface runoff and evapotranspiration for the same period. Recovery or recharge from rainfall is a function of the amount of total precipitation (R_t). If the intercept is R_e, the recharge (R_s)is estimated by the following formula, which is expressed as

$$R_s = R_{tc} - R_e \tag{4}$$

Where, R_{tc} is the result of the computation of the total precipitation by means of equation (3) during the wet period in a year.

2.2 Estimation of Groundwater Recharge

In this paper, a modified GW fluctuation technique is presented for estimating GW recharge in shallow depth based on a statistical relation between the precipitation and water levels. The analysis is based on the original conditions, which are unaffected by heavy pumping. Observed monthly precipitation and weekly GW levels are the only data required to carry out the study. For this study, daily precipitation at CL41 monitoring station and GW level data at GT5015004 monitoring well are collected from the Bangladesh Water Development Board for the period from 1992 to 2007. Kushtia district of Bangladesh has been selected as a case study area. By analyzing the monthly precipitation and water level records belonging to this period, the amount of rainfall causing a rise in water level is determined. These rainfall amounts are called "total precipitation (R_t)" causing the rise in water level. The water level rise caused by this total precipitation is termed as "water level rise due to GW recharge (ΔH)" and can be found from the graph directly. This procedure is repeated year after year for the period of record from 1992 to 2007 and the relation between these two variables R_t and ΔH is established by using simple statistical technique. ΔH is a function of the precipitation amount above a threshold rainfall value, which represents the precipitation causing surface runoff, evapotranspiration, and subsurface flow or only one of them depending one the local hydrogeological conditions in the study area. It is obtained by setting ΔH = 0 in the regression equation, which is presented in Figure 2. The recharge amount for each year is then calculated by subtracting the threshold precipitation from the total precipitation causing water level rise in that year.

3. RESULTS AND DISCUSSIONS

Kushtia district of Bangladesh has been selected as a case study area for this study based on the data availability. It has an area of 1621 square kilometres and is bounded by Rajshahi, Natore and Pabna districts to the North, by Chuadanga and Jhenaidah districts to the South, by Rajbari district to the east and by West Bengal and Meherpur district to the west. The upazillas (sub-district) are Kushtia Sadar, Kumarkhali, Daulatpur, Mirpur, Bheramara and Khoksa. Ganges, Gorai, Mathabhanga, Kaligonga and Kumar are the main rivers flowing through the district. The average maximum and minimum temperature is 37.8 ^{0}C and 11.2 ^{0}C, respectively with an annual average rainfall of 1,898 mm. The GW levels naturally fluctuate in response to a sequence of climatic events and to constraints imposed by the hydrogeologic and topographic characteristics (Figure 3). The figure demonstrates that the aquifer is recharged in the wet period due to the excess rainfall and discharged in the dry period as there is not much rainfall available in that time. In addition, evapotranspiration and water withdrawal takes place, which cause the depletion of the water table. Groundwater recharge is largest in the wet period, specially in the monsoon, when plants are dormant and evapotranspiration rates are comparatively less. In the dry period, when evapotranspiration rates exceed the available moisture from precipitation, recharge to the water table is negligible and thus, GW levels decline (Figure 3).

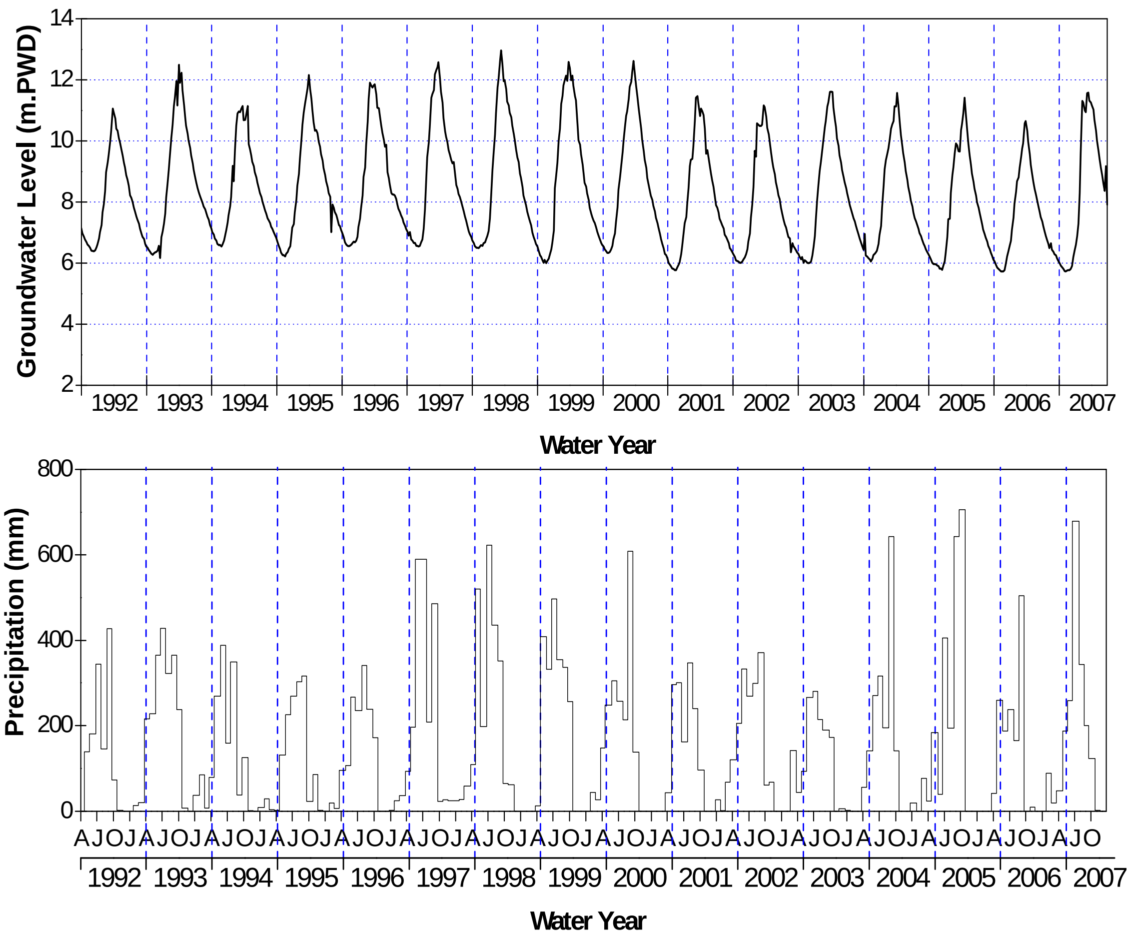

Figure 3: Weekly GWL hydrograph and monthly precipitation hyetograph for 1992 to 2007 in the study area

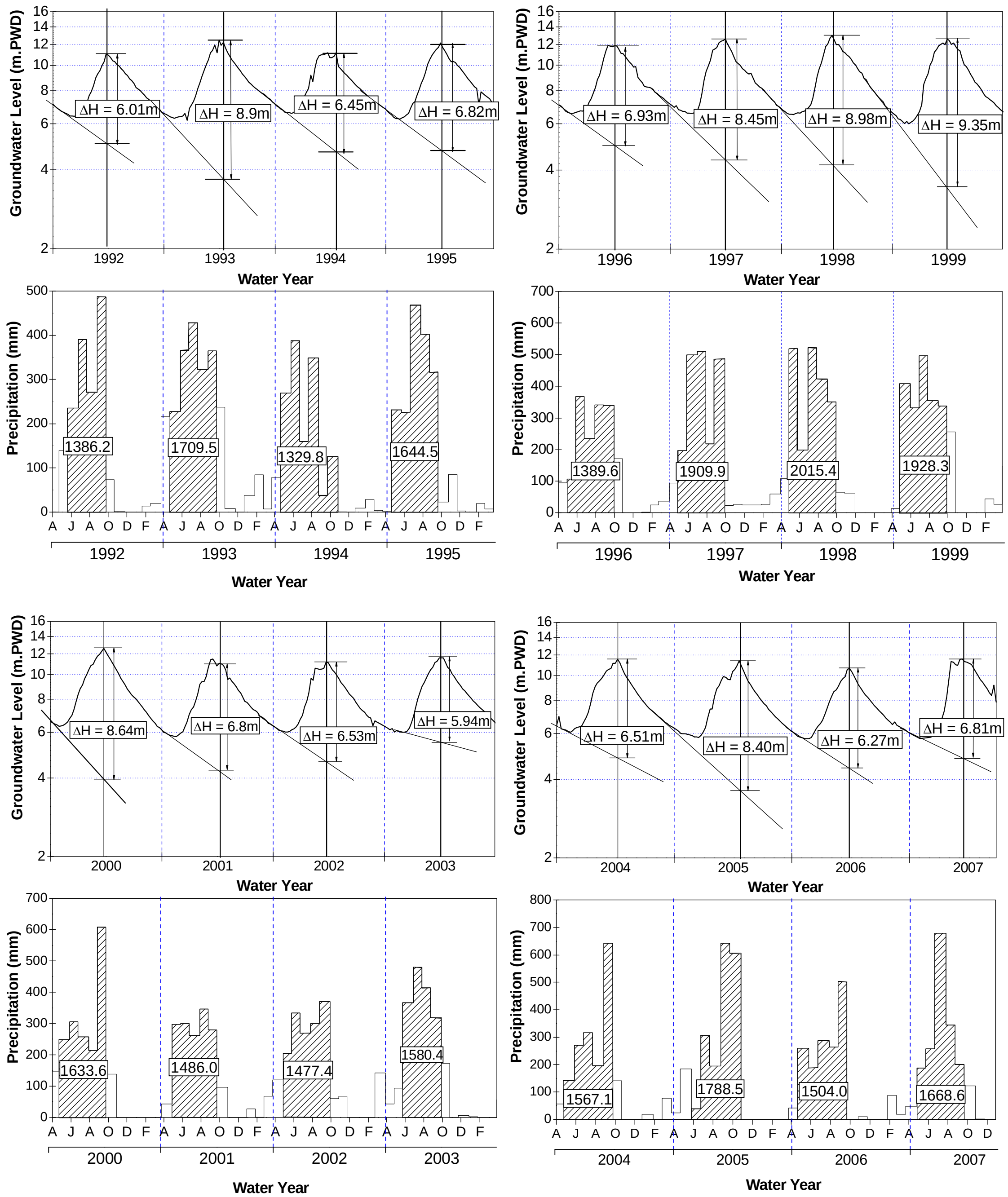

Figure 4**:** Fluctuations in water levels caused by recharge from precipitation for GT5015004 monitoring well

The recovery of the water level (ΔH) and the total precipitation (R_t) during the wet period in a year for water years from 1992 to 2007 are determined (Figure 4) and a linear regression equation during the period is established (Figure 5a). The results of linear regression analysis are presented in Table 1.

Table 1: Results of linear regression analysis

Type of relation	No. of observations	Correlation coefficient, r	Standard error of estimate	Regression Equation
Total precipitation-GW level recovery	16	0.838	1.486	$\Delta H = -1.06585 + 0.00521 R_t$

Table 2: Summary of recharge computation

Water Year	Annual Precipitation, R_y	Recovery of GW level, ΔH	Computed Total Precipitation, R_{tc}	Recharge, R_s
1992	1633.60	6.01	1356.21	1151.63
1993	2301.20	8.90	1912.83	1708.25
1994	1450.60	6.45	1442.58	1238.00
1995	1782.50	6.82	1513.60	1309.02
1996	1719.70	6.93	1534.71	1330.13
1997	2190.50	8.45	1826.46	1621.88
1998	2250.60	8.98	1928.19	1723.61
1999	2267.60	9.35	1999.20	1794.62
2000	1919.10	8.64	1862.93	1658.35
2001	1720.80	6.80	1509.76	1305.18
2002	1868.70	6.53	1457.94	1253.36
2003	1897.60	5.94	1344.69	1140.11
2004	1861.80	6.51	1454.10	1249.52
2005	1996.50	8.40	1816.86	1612.28
2006	1664.00	6.27	1408.03	1203.45
2007	1841.80	6.81	1511.68	1307.10

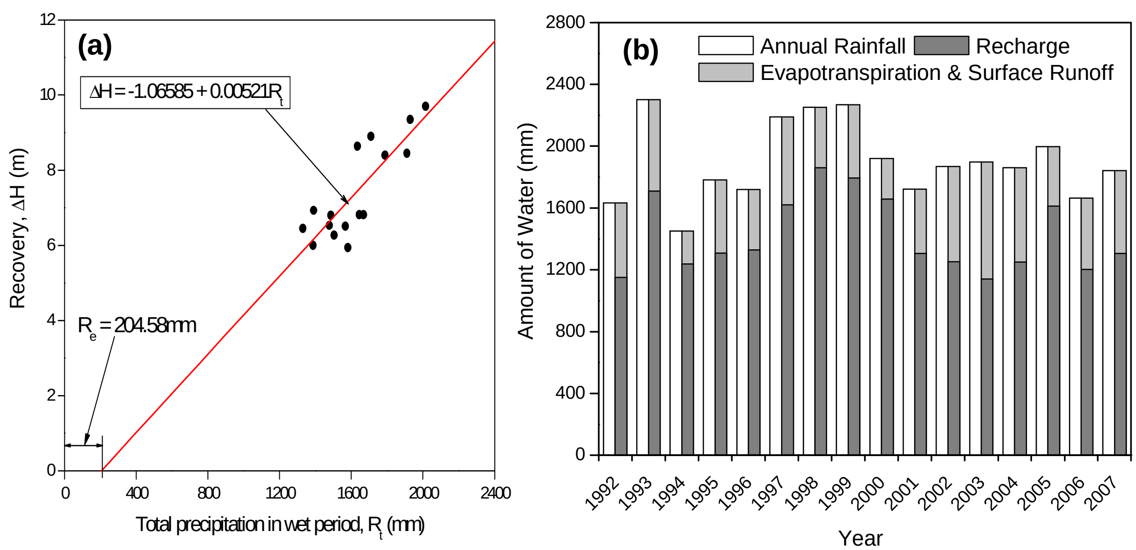

Figure 5**:** (a) Relationship of total precipitation in wet period and groundwater recovery and (b) Estimated GW recharge for 16 years in the study area

The regression equation is $\Delta H = 1.06585 + 0.00521 R_t$. The average precipitation intercept (R_e) of the aquifer is found as 204.58 mm (Figure 5a), which represents the amount of surface runoff and evapotranspiration for the same period. The computed values (R_{tc}) of total precipitation by means of the regression equation for water years from 1992 to 2007 are summarized in Table 2. The results of the computation of the recharge (R_s) from precipitation of this aquifer by means of equation (4) for the whole 16 years period and are presented in Figure 5b. The annual GW recharge in the study area for the 16 years period (1992-2007) ranges from 1794 mm in 1999 to 1140 mm in 2003 (Figure 5b). The annual average recharge during the period from 1992 to 2007 is 1413 mm (Table 2). This is about 74 percent of the average annual precipitation (1898 mm) in the study area.

4. CONCLUSIONS

The objective of this study has been a synthesis of the results, which can be obtained by analysis of water level fluctuations. The accuracy of the results depends exclusively on the base of the water level observation data. This study concludes that the methodology described in this paper is expected to open a window of opportunity for any initial level assessment of GW resources in a shallow aquifer under consideration, where shortest possible data and information are available in hand. This study conclusively proves that shallow GW in the study area is recharged mainly from precipitation and the annual average GW recharge is 1413 mm for 16 years period from 1992 to 2007. This is about 74 percent of the average annual precipitation.

ACKNOWLEDGEMENTS

The authors wish to express thanks to Bangladesh Water Development Board (BWDB) for the data supports.

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IMPACT OF CLIMATE CHANGE ON FLOODS OF BANGLADESH AND INTRODUCING FLOOD INTENSITY INDEX TO CHARACTERIZE THE FLOODING SCENARIO

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ABSTRACT

Bangladesh is a disaster prone country, and the effect of climate change magnifies the governing factors of the disasters. In this paper, firstly the temporal changes of pattern of different disasters in Bangladesh are discussed based on their occurrence rate, number of people killed, and exposure to human and economic loss. The paper is mainly concentrated on temporal variation of floods in Bangladesh. The decadal change in return period and probability of low, moderate and high flood events are presented. The term 'Flood Intensity Index (FII)' is introduced to characterize the flood, which is considered as a function of inundation depth and duration. Flood Intensity Index is calculated for two mega floods of 1988 and 1998. It is observed that although the average depth of inundation among two mega floods does not differ too much, the Flood Intensity Index in 1998 was much higher (about double) than 1988 flood, and that is why the economic loss in 1998 flood was much higher (about 2.33 times) than that of 1988 flood. Previous data and various future predictions showed that both the parameters in Food Intensity Index are in increasing trend due to climate change impact. The climate change induced increament in river discharge during monsoon will increase the inundation depth, and the increasing trend of Sea Level Rise will increase the duration of flood due to back water effect of sea. Therefore, the Flood Intensity Index will significantly be increased due to climate change. Since, the 'Flood Intensity Index (FII)' is a measurable index to characterize a flood accounting all the impacts of climate change, this index can be used in expressing the change in flooding scenario due to climate change.

Keywords: *Climate Change, Sea Level Rise, Floods, Inundation*

1. INTRODUCTION

Bangladesh is one of the biggest deltas in the world and have an area of 1,44,000 sq. km. Geographically, Bangladesh is situated at the tip of a funnel, through which huge amount of rain water discharged in monsoon to the ocean flushing over the country. Depression on the ocean, cyclones, tides and storm surges can easily affect the country through the unprotected shore. Therefore, Bangladesh is a disaster prone country. Effect of Climate Change magnifies the governing factors of disasters. The coastal belt of Bay of Bengal is in the south of Bangladesh. Among its 230 rivers, the Ganges, the Brahmaputra and the Meghna are the major rivers. They have vast catchment area (about 1.72 million km^2), whose only 7.5% lies within Bangladesh. Bangladesh experienced flood about every year. Generally 20% of the country is inundated in a normal flood, the highest flood inundates close to 100,000 km^2 in 1998. Since 1970, about 41 million people became homeless and it is estimated that it will be 68 million till 2020. In this paper, firstly the temporal changes of pattern of different disasters in Bangladesh are discussed based on their occurrence rate, number of people killed, and exposure to human and economic loss. The paper is mainly concentrated on temporal variation of floods in Bangladesh. The decadal change in return period and probability of low, moderate and high flood events are presented. The term 'Flood Intensity Index (FII)' is introduced to characterize the flood, which is considered as a function of inundation depth and duration. A heuristic approach is described to explain how the climate change will magnify the flood intensity index of this country.

2. DATA

Three types of data is used in this study: i) the rainfall data, ii) the river discharge and flood hydrograph data and iii) time series of disasters in Bangladesh. For the first one monthly rainfall records of 17 stations (out of which, complete set of data was available in 12 stations) of Bangladesh for fifty years (1958–2007) are collected from Bangladesh Meteorological Department. For this data, homogeneity of rainfall records are analyzed by Shahid (2009), and reported that the data of all the stations are homogeneous. The data on time series of historical floods in Bangladesh with inundation area are collected from Bangladesh flood forcasting and warning center (FFWC). The disaster related data were collected from OFDA/CRED International Disaster Database (http://www.em-dat.net), Université Catholique de Louvain, Brussels, Belgium.

3. ANALYSIS AND DISCUSSION

3.1 Disaster Profile of Bangladesh

The main three disasters of Bangladesh are flood, cyclone and drought. Based on the Global Assessment Report on Disaster Risk Reduction of United Nations in 2009, the world ranking of Bangladesh according to the total affected people and economic loss in different disasters is shown in Table 1. It is observed that the Flood is the mostly devastating disaster in Bangladesh compared to other disasters as it ranked 1st for the case of population exposed and 3rd in the case of economic exposure.

Table 1: Different types of disasters and the world ranking of Bangladesh (UN, 2009)

(a) Human exposure

Disaster Type	Population exposed	World ranking
Flood	19,279,660	1st out of 162 countries
Cyclone	4,641,060	6th out of 89 countries
Drought	642227	63rd out of 184 countries

(b) Economic exposure

Disaster Type	GDP exposed	World ranking
Flood	9.74 bill. USD	3rd out of 162 countries
Cyclone	2.36 bill. USD	12th out of 89 countries

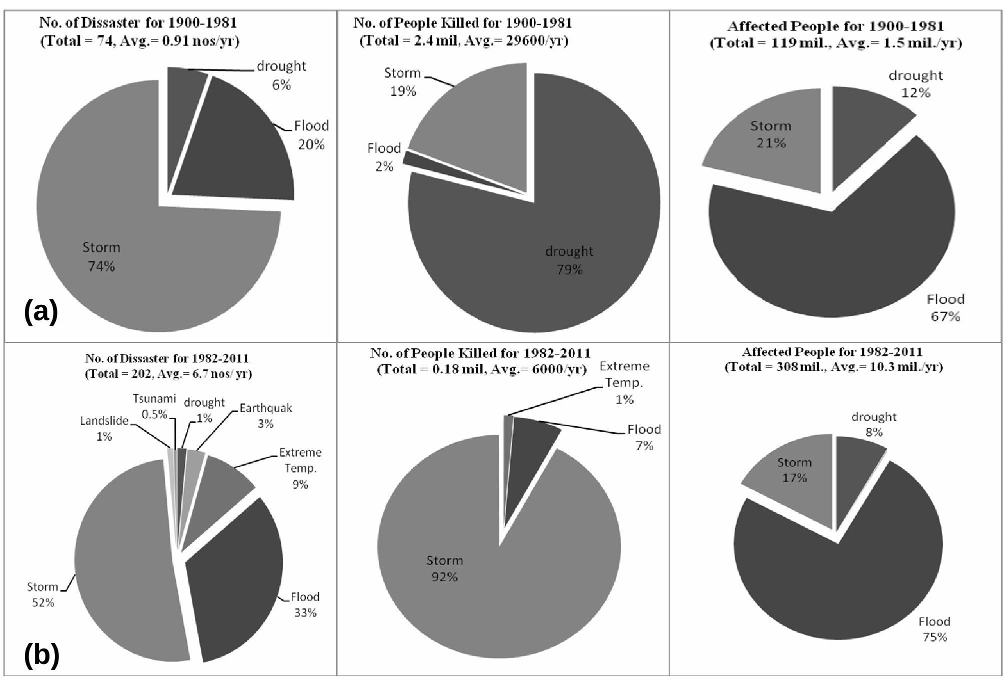
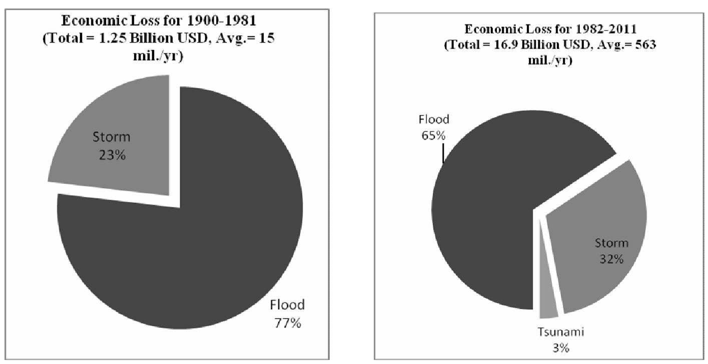


Figure 1: Affected people in different disasters between disaster year (a) 1900-1981 and (b) 1982-2011

Available disaster data from 1900 to 2011 was devided into two time spans 1900-1981 (1st span) and 1982-2011 (2nd span), and their comparative nature is compared in Fig. 1. It is observed that the number and pattern of disasters in the two time spans are varied significantly. In earlier time span, the number of disaster was about 1 number per year, and that number increased to about 6.7 in recent time span. In addition to that, new variety of disasters have been occured in recent past decades compared to that of before 1981. Although cyclone is the dominating disaster according to its number of yearly occurances with respect to others, its percent in 2nd span of time is reduced compared to 1st span of time. However, the percent of flood increased in recent years (1982-2011) compared to past decades. The yearly no of people killed by natural disasters is reduced by about 5 times in recent decades compared to 1st time span. That may be due to the improvement of management and communication facility of the country. In time span 1900-1981, the main cause of people killed was drought whereas killed due to cyclone and flood was very little compared to drought. But in the time span 1982-2011, the dominating disaster for people killed was storm (about 92%). Affected people per year was about 7 times higher in 1982-2011 than 1900-1981 span, and people mostly affected by flood in both spans, though the percentage is increasing day by day. Yearly affected people due to storm and drought is slightly decreasing during both cases.

The pie charts of Fig. 2 shows that in the recent time span 1982-2011, total economic loss was 16.9 billion where average economic loss was 563 mil./yr which was 38% higher than the past decades, though the liable disasters for economic loss was almost same. As the number of disasters has been increased in recent decades, the affected people and economic loss have also increased. In this case also, the dominating disaster is mostly flood and next is storm.



(a) Disaster Year 1900-1981 (b) Disaster Year 1982-2011

Figure 2: Comparison of economic loss in disasters between year 1900-1981 and 1982-2011

Table 2: Summary of the comparison of disaster parameters

Disaster Parameters	**1900-1981**	**1982-2011**
Events/Year	0.91	6.7
No. of people killed/Year	29,600	6,000
Affected people/Year	1.5 Million	10.3 Million
Economic loss/year	15 Mil. USD	563 Mil. USD

The summary of disaster parameters for two time spans is shown in Table 2. Disaster nos., number of affected people and economic loss increases with time in an alarming rate.

3.2 Contribution of Territorial Rainfall and River Discharge on Floods of Bangladesh

Bangladesh has a tropical climate dominated primarily by monsoon and partly by pre-monsoon and post-monsoon circulations. The annual average precipitation is about 2,300 mm, varying from as little as 1,200 mm in the west to over 5,000 mm in the east. It enjoys four different seasons namely: pre-monsoon (March to May), monsoon (June to September), post-monsoon (October to November) and winter (December to February). The seasonal variation of rainfall is very high. The scenario in the variation of river discharge is much more severe.

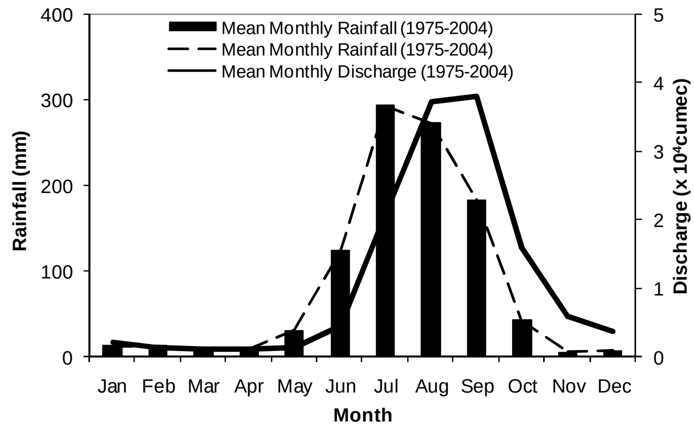


Figure 3: Ganges basin mean monthly rainfall and mean monthly discharge at Hardinge bridge point of Ganges river (Source: Jian et al., 2009)

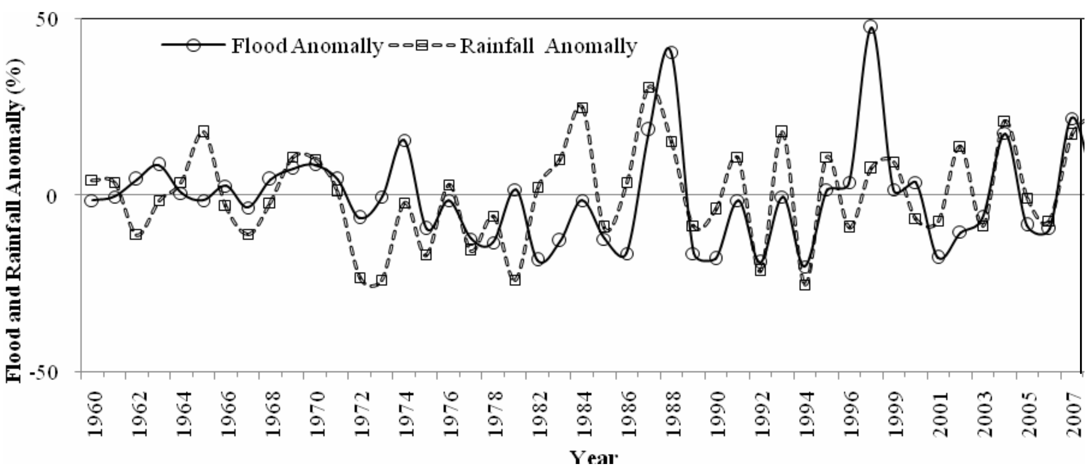


Figure 4: Corellation between Rainfall and Flood in the country

Figure 4 shows the comparison of Monsoon rainfall anomaly with that of flooding area. The anomaly is defined as the percent deviation of a parameter from its temporal mean value. It is observed that for more than 85% cases, the flood anomaly is in phase with that of territorial rainfall. Although it is expected that the huge upstream flow has a high impact on floods of Bangladesh, such scenario is observed only for 15% cases. In the figure the 1998 flood shows such a case.

Figure 5 compares the mean monthly discharge at Hardinge Bridge Point of Ganges river and that of Bahadurabad point of Brahmaputra river. It shows that it has very high discharge in monsoon, mainly July to September. The discharge curve is found to be lagged by months to the rainfall curve. Since the discharge in the river is highly influenced by the huge upstream flow outside the country, it is not expected to show a definite relationship between rainfall and river discharge pattern. But the figure that prepared averaging 20 years data, show similar pattern in curve between rainfall and discharge for Ganges Basin. It is important to mention that the annual cycles of discharge in Brahmaputra river indicates significant phase differences with Ganges. The Brahmaputra flow increases rapidly in late spring and peak at mid of July, ahead of the Ganges by about two months, probably for two reasons: firstly, due to extreme human activity in Ganges basin, plus the different basin time-scales; Second, rains generally occur in Assam (to the northeast of Bangladesh) some weeks earlier than over the Ganges catchment (Lawrence and Webster, 2002). The phase difference in the rising limb of their hydrograph is 1.5 months. Probably, this is the reason for which most of the annual flood hyetographs show double peaks: in July and end of Aug. If the Brahmaputra does not discharge out its peak flow rapidly, its peak will coincide with the peak of Ganges, it will cause an extreme high flood.

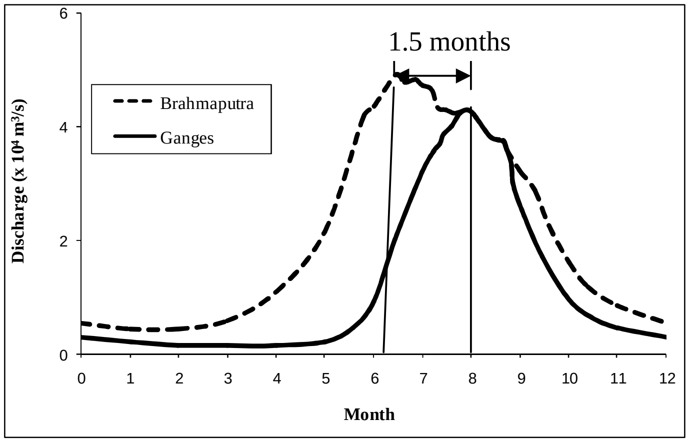


Figure 5: Time lag between Peak Discharges in Ganges and Brahmaputra River Hydrograph

3.3 Visible Impact of Climate Change on Floods

3.3.1 Increase of Brahmaputra River Discharge

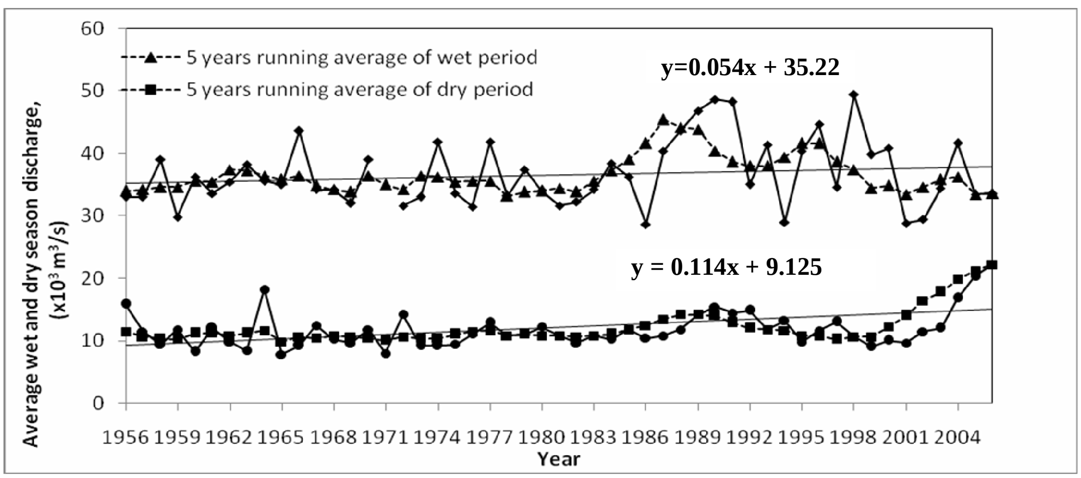


Figure 6: Temporal change of discharge in Brahmaputra River for wet and dry seasons

Figure 6 shows the temporal variation of mean seasonal discharge at Bahadurabad point of Brahmaputra river for wet and dry seasons seperately. It is found that in both the seasons, the discharge is increasing with time. In wet season, the yearly increasing rate of Brahmaputra river discharge is about 54 cumec and that of dry season is about 114 cumec. The discharge has been increased about 7.5% in last 50 years. The fluctuation in the variation of monsoon discharge is also increased. The variation of averaged monthly discharge for two averaging time spans 1956-1980 and 1981-2005 is compared in Fig. 7. The Averaged monthly Brahmaputra discharge in Bangladesh is increased about 8% in 50 years.

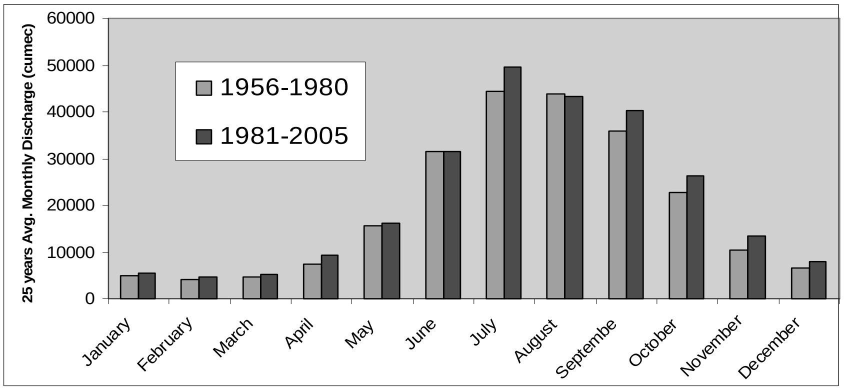


Figure 7: Monthly variation of Brahmaputra River Discharge in two time span

3.3.2 Change in the trend of historical floods and rainfalls in Bangladesh

The historical time series of floods in Bangladesh shows that the top 5 floods in terms of inundated area are occurred in last 20 years in 60 years of history (Fig. 8). 1998 Flood is the most devastating flood, inundation was close to 67% that displaced more than 30 million people with 20 million homeless; estimated damage was 2.8 billion USD. On the other hand, the second ranked flood was in1988, where 61% of the area was inundated; estimated damage was 1.2 billion USD. Although the inundation area in 1998 flood is about 6% higher than 1988, the estimated damage is 2.33 times higher. The cause behind this is explained under next subheadings in detail.

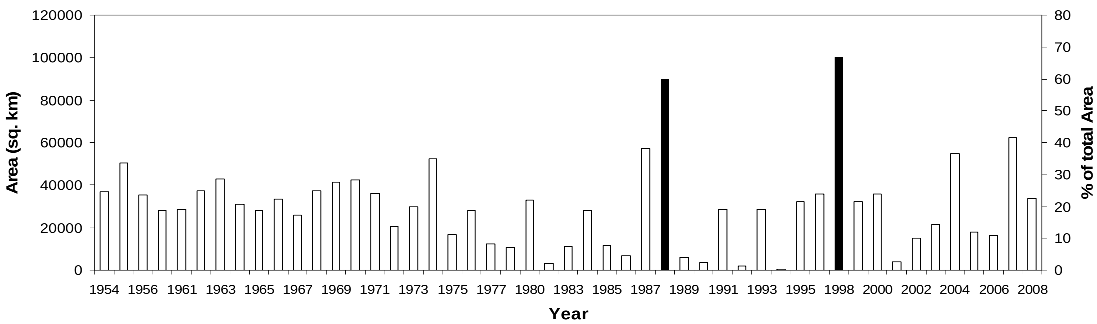


Figure 8: Time series of historical floods in Bangladesh with inundation area

Inundation to the extent of 20% area of the country is beneficial for crops and ecological balance (Annual Flood Report, 2008). But the flood more than 20% cause direct and indirect damages and considerable inconveniences to the people. From Fig. 8, it is also evident that the extreme flood events in terms of inundation area are also increased in recent years. During 1954~1972, the yearly flooding area was quite uniform, after that year to year fluctuation in flooding area is very high. It is observed that the number of moderate floods are decreased highly and they are converted to either extreme high floods or the extreme low floods. Evidence of such extreme events and rapid change of flood intensity indicates the effect of climate change.

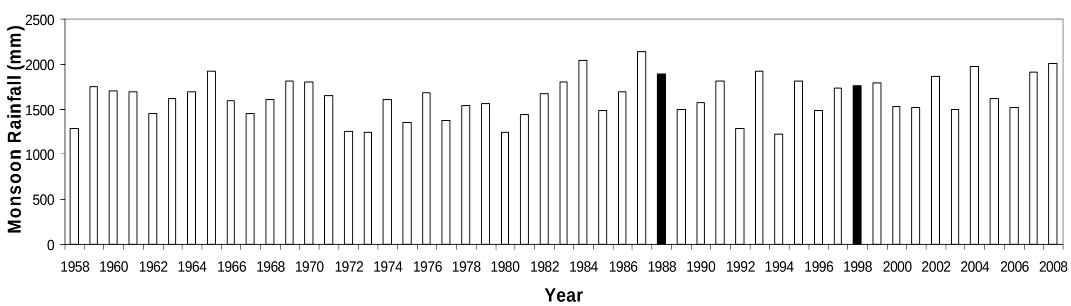


Figure 9: Time series of Monsoon rainfall in Bangladesh

Figure 9 shows the time series of monsoon rainfall over Bangladesh for a period of 1958 to 2008. If we draw a trend line for these 50 years of rainfall data, it can be found that the monsoon rainfall increases very gently as 2.65 mm/year. Many researchers explain it as the probable impact of climate change, which is also match with the prediction of 3rd IPCC report. From the figure it is also observed that the rainfall over Bangladesh in 1998 is about 6% lower than that of 1988, although in 1998 the flooding area was 6% higher than 1988. Reasonably, the rainfall at upstream also plays a role in inundating the country.

3.3.3 Impact of Climate change on the Return period and frequency of floods

To analyze the temporal change of return period and probability of floods, the floods were classified into three catagories: low, moderate and high flood events. In this study the moderate flood events were defined as the mean flooding area of a period ± 5%. Considering time series of flooding area from 1954 to 2008, the moderate flood events were found as 19% to 29% area flooding of the country. Therefore, the low flood means <19% and high flood means >29% area flooding events. To show the temporal change, the data set was divided into two time spans: 1954-1981 and 1982-2008. From the two sets of data as shown in Fig. 10, it is observed that, in high flood events (Inundation > 29% area), the return period decreased and the probability increased by about 3 times in recent decades. In low flood events (Inundation <19% area), the return period decreased in recent decades and the probability increased by about 1.4 times. In Moderate Flood events (Inundation =19 - 29% area), the return period increased and the probability is reduced to about half (1/2).

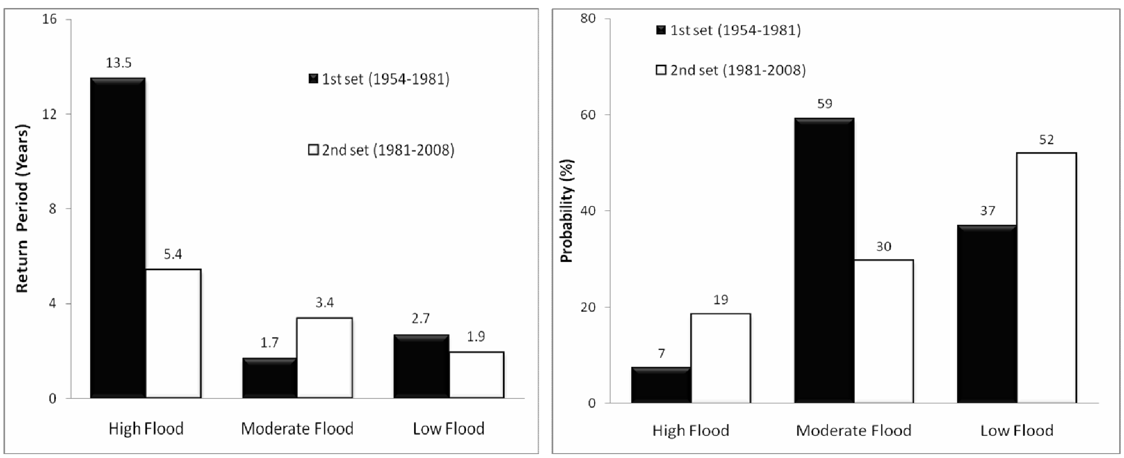


Figure 10: Return period and probability of flood

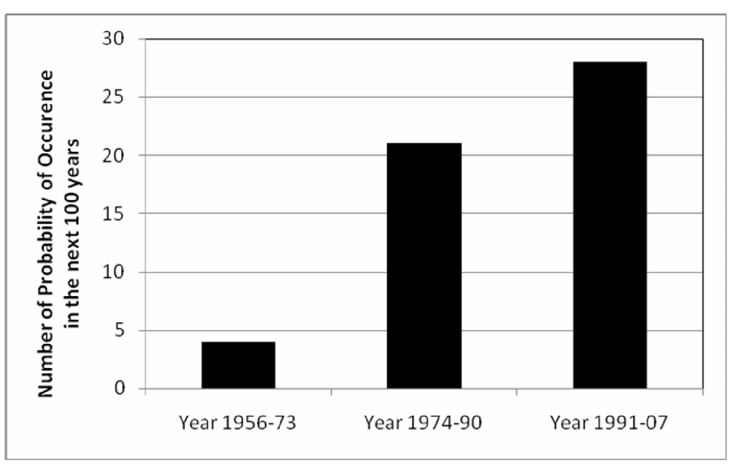


Figure 11: Increase of number of occurrence of characteristic flood over the historical years at Bahadurabad

Considering a particular example of flood at Bahadurabad point of Brahmaputra river with discharge = 76,137 m^3/s, for data set of 1956 to 2007, it is found that the probability of occurrence of the characteristic flood is increased from 4% to 28% over the last 50 years. Figure 11 shows the increase of number of occurrence of the characteristic flood over the three equal durations.

3.4 Impact of Climate Change on the Intensity of Flood

3.4.1 Conceptual Model to Explain the Climate Change Impact on Flood

We know the main causes of flood in Bangladesh are excessive precipitation, low topography and flat slope of the country. Among others, the frequent development of low pressure areas and storm surges in the Bay of Bengal can impede drainage, and the severity of flooding is greatest when the peak floods of the major rivers coincide with these effects.

IWM (2006) predicted that, by the year 2100, the temperature will be increased about 2.4^0C, the monsoon precipitation will be increased about 11.8% and the SLR will be 30cm ~ 1 m (9 cm ~ 88 cm by IPCC). They also predicted that, for 2^0C temperature rise and 10% increase in precipitation, the discharge in Ganges river will be increased about 19%, that of Brahmaputra will be increased about 13%, and about 11% discharge will be increased in Meghna river. That means, on an average, 10 to 20% discharge will be increased in all the three mighty rivers in Bangladesh by the year 2100 with a significant amount of SLR.

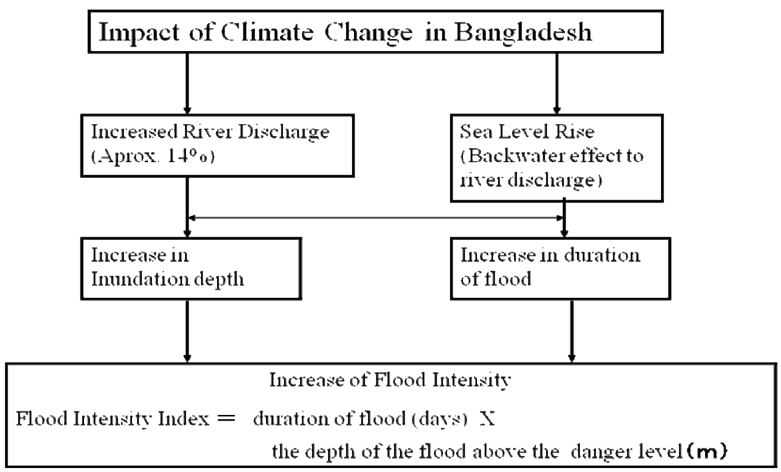


Figure 12: Impact of climate change over flood

To explain the long term effect of climate change on flooding scenario of Bangladesh, an index named *Flood Intensity Index* is introduced here to characterize the intensity of flood. Flood Intensity Index is defined as the product of the depth of flood above the danger level (m) and the duration of flood (days). A conceptual model to explain the impact of climate change on the flooding scenario of Bangladesh is presented in Fig. 12. It shows that due to increase of river discharge, the inundation depth will be increased; on the other hand, due to Sea Level Rise there will be drainage congestion to drain out the river water to the sea due to back water effect of higher elevated sea level. This will cause the slow down of rate of discharging river water to the Sea, which will actually influence to prolong the duration of flood. Since both the parameters of flood intensity index will be increased by the impact of climate change, the intensity of flood will also be increased significantly. An example of calculating Flood Intensity Index is given below.

3.4.2 Calculation of Flood Intensity Index

In this section, the flood intensity index is calculated for 1998 and 1988 floods and compared with a normal flooding year 2008. The water level above the danger level during flood is termed here as inundation depth. Figure 13 shows the comparison of inundation depths in 1998 and 1988 floods with normal flooding year 2008 for some locations in Brahmaputra basin. Figure 15 shows the same for some locations in Ganges basin. In a same flood, the depth of inundation shows different trend in different Basins. Comparing the trend of inundation depth between 1988 and 1998 flood in Ganges basin for some locations, it is found that the average flooding depth in 1998 flood is little bit higher than 1988 flood. On the other hand, in most of the locations of Brahmaputra basin, the inundation depth of 1998 flood is less than 1988 flood. On an average, it can be concluded that, the flooding depth between 1988 and 1998 flood did not differ much.

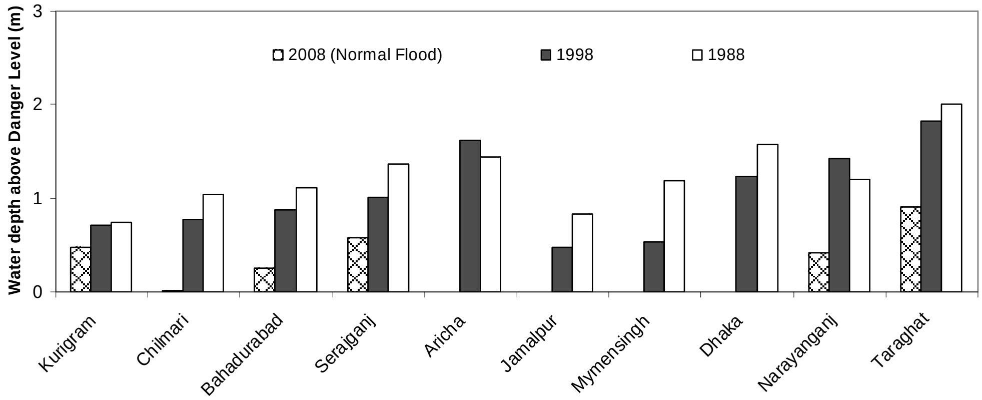


Figure 13: Comparison of inundation depth in 1998 and 1988 floods with normal flooding year 2008 for some locations in Brahmaputra basin

Figures 14 and 16 show the comparison of durations of inundation in 1998 and 1988 floods for several locations in Brahmaputra basin and Ganges basin, respectively. From the figures it is found that the durations of flood at most of the locations in both Brahmaputra and Ganges rivers are much higher in 1998 than 1988 flood. The durations of flood in 1998 were 3.88 times higher in Jamalpur (Old Brahmaputra), 3.3 times higher in Mymensingh (Old Brahmaputra), 2.5 times higher in Dhaka (Buriganga), 2.4 times higher in Bahadurabad (Brahmaputra) and 1.7 times higher in Goalanda (Ganges) than that of 1988 flood. If we compare Figs. 13 and 14, most of the places the inundation depth in 1998 is seen less than 1988 but the duration of flood in 1998 is found much higher than 1988. Therefore it can be concluded that, although the depth of inundation among two floods does not differ too much, the duration of flood in 1998 was much (up to 4 times) higher than 1988 flood.

Figure 17 shows the calculated Flood Intensity Index for different basins and their averages for years 1988, 1998, 2007, 2008 and 2010. It shows that although the depth of inundation among 1988 and 1998 floods does not differ too much, the Flood Intensity Index in 1998 was much higher than 1988 flood, that is due to the prolong duration of flood in 1998 than 1988 flood. In 1998, the peak discharges of Ganges, Brahmaputra and Meghna were high and they were almost same, so the flood duration was prolonged. But in 1988 flood, peak discharge of Meghna was much higher than the Brahmaputra and Ganges Rivers and so the overall Flood Intensity Index was lower than 1998 flood. The figure also reveals information that the 2007 flood was third largest flood accrding to Flood Intensity Index. The figure also shows that although the 1998 flood is dominated by the floods in all rivers, the floods in 1988, 2007 and 2010 are dominated by the floods in Meghna basin. On the other hand, 2008 flood is dominated by the floods in Ganges basin.

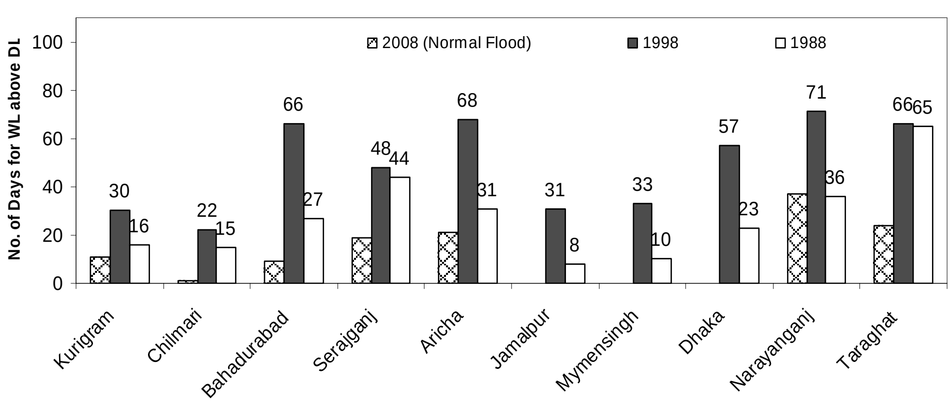


Figure 14: Comparison of durations of inundation in 1998 and 1988 floods with normal flood year 2008 for some locations in Brahmaputra basin

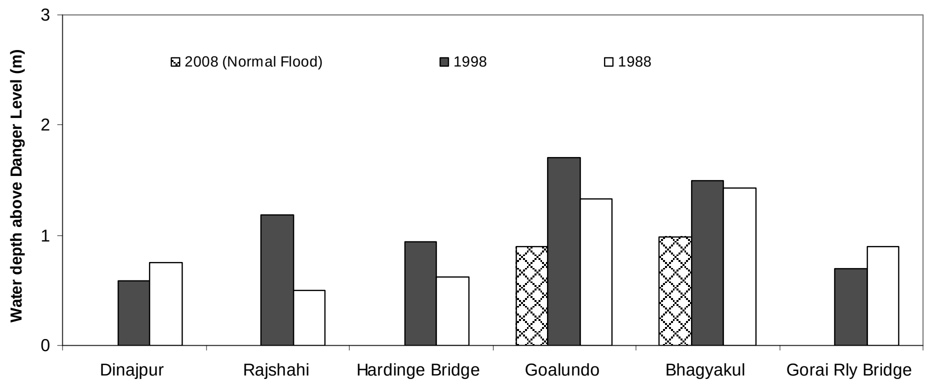


Figure 15: Comparison of inundation depth in 1998 and 1988 floods with normal flooding year 2008 for some locations in Ganges basin

4. CONCLUSIONS

Although the paper is mainly concentrated on temporal variation of floods in Bangladesh, the temporal changes of pattern of other disasters in Bangladesh are also discussed and compared with flood based on their occurrence rate, number of people killed, exposure to human and economic loss. Based on the analysis, following conclusions are made:

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- Comparing the disaster parameters for two time spans, it is found that the disaster nos., number of affected people and economic loss increases with time in an alarming rate. It is observed that the flood is the mostly devastating disaster in Bangladesh compared to other disasters as it ranked 1st in the world for the case of population exposed and 3rd in the case of economic exposure.
- Since the discharge in the river is highly influenced by the huge upstream flow outside the country, it is not expected to show a definite relationship between rainfall and river discharge pattern. But the 30 years average discharge curve of Ganges river is found to be similar pattern of rainfall (in Ganges Basin within Bangladesh) lagged by a months. Comparing 50 years data of rainfall anomaly with flood, it is observed that for more than 85% cases, the flood anomaly of Bangladesh is in phase with that of territorial rainfall anomaly.

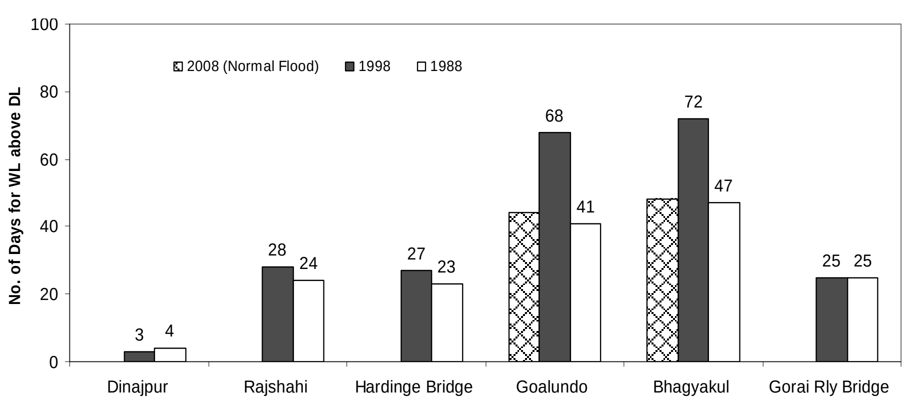

Figure 16: Comparison of durations of inundation in 1998 and 1988 floods with normal flooding year 2008 for some locations in Ganges basin

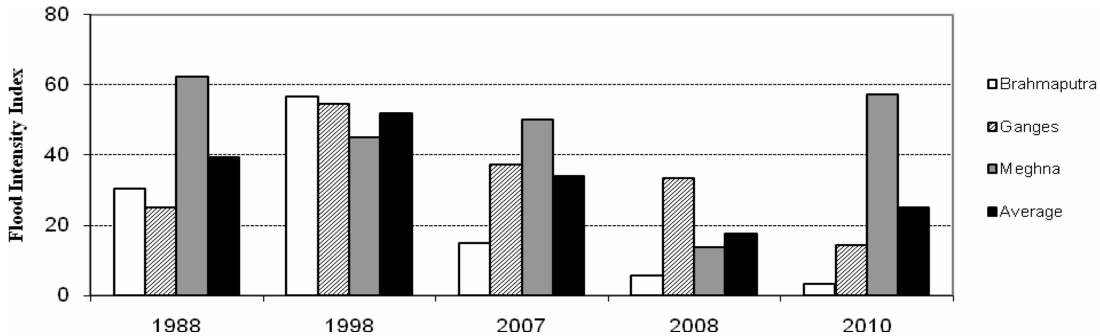

Figure 17: Comparison of Flood Intensity Index of 1998 and 1988 floods with normal flooding years for Ganges, Brahmaputra and Meghna basin

- In wet season, the yearly increasing rate of Brahmaputra river discharge is about 54 cumec and that of dry season is about 114 cumec. The Brahmaputra discharge at Bahaduarabad point has been increased about 7.5% in last 50 years.
- The historical time series of floods in Bangladesh shows that the top 5 floods in terms of inundation area are occurred in last 20 years in 60 years of history. It is observed that the number of moderate floods are decreased highly and they are converted to either extreme high floods or the extreme low floods. From the trend line for 50 years of rainfall data, it is found that the monsoon rainfall increases very gently as 2.65 mm/year. These changing phenomena in rainfall and flooding scenario in Bangladesh can be explain as the probable impact of climate change.
- The term ‘Flood Intensity Index (FII)’ is introduced to characterize the flood, which is considered as a function of inundation depth and duration. Flood Intensity Index is calculated for two mega floods of 1988 and 1998. It is observed that although the average depth of inundation among two mega floods does not differ too much, the Flood Intensity Index in 1998 was much higher (about double) than 1988 flood, and that is why the economic loss in 1998 flood was much higher (about 2.33 times) than that of 1988 flood.

Previous data and various future predictions showed that both the parameters in Food Intensity Index are in increasing trend due to climate change impact. The climate change induced increment in river discharge during monsoon will increase the inundation depth, and the increasing trend of Sea Level Rise will increase the duration of flood due to back water effect of sea. Therefore, the Flood Intensity Index will significantly be increased due to climate change. Since, the ‘Flood Intensity Index (FII)’ is a measurable index to characterize a flood accounting all the impacts of climate change, this index can be used in expressing the change in flooding scenario due to climate change.

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RIVER SITE AGRICULTURAL LAND PROTECTION MEASURES TAKEN BY THE BAMBOO BANDALLING STRUCTURES

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ABSTRACT

The river Jamuna in Bangladesh carries huge amount of sediment from the upper riparian country India during flood. These sediments are deposited in bed of the river and so the river water depth is reduced result the river bank erosion which needs to be control for the proper land use. Without the control of land use from the river bank erosion which creates the flood risk for river site dwellers by eroding the home stead and agricultural lands. As this river is hydraulically dynamic in nature and causes extensive riverbank erosion, the near bank flow velocity and scour depths in all the major rivers in Bangladesh are very high. It is mention here that the planform of the Jamuna is braided and that of the Ganges and the Meghna are meandering. Due to the different morphology of the river, different bank protection structures such as the (1) structural and (ii) non-structural measures are taken into consideration for the land use control measures. The structural measure are basically four methods such as hard point, revetment, groyne, spur and non-structural methods are such as the bamboo bandalling, purcupines, vegetation and public awareness creation etc. Among these structures, in this paper only the bamboo bandalling structure is used as a non-structural measures for the land use control.The bank protection works well by constructing bandals which has been found enough to control land use measures during the monsoon flood are addressed in this paper. It is noted that the bank materials of this river consist mainly of non-cohesive sediment such as sand and sand mixed silt which are highly susceptible to erosion. For understanding the prevailing hydro-morphological characteristics of the Jamuna river branch channel, the performance the bamboo bandalling structures are constructed as a non-structural measure is being evaluated by considering different aspects, such as its stability, cost and safety.

Keywords*: sediment, flood, structural, non-structural, bandalling*

1. INTRODUCTION

River bank erosion is the single most common problem at river which are flow on alluvial plain although the plain usually at lowland area. The erosion causes a lot of damage. That can be seen from existence of meandering symptom at the river. The erosion will result the loss of land, or property and endanger people who are live near the river. The existing bank erosion control structures are usually expensive, massive and not compatible with environment and aesthetic. This structure is made from cement concrete block etc. Those will bad influence to the vegetation growth and habitat for species that are living around water; it does not match with the spirit of eco-hydraulic. Because those are costly, the erosion control is only conducted to protect important place or populous and costly urban area, which enough worth to be protected with above construction. While, in rural area which is value of land or property is low, cannot be protect by above structure, because that is not feasible if compare the cost of the structure.

Within the South-East Asia, there is a country for the land rivers Bangladesh by name in which the Jamuna is a braided river whose braiding index varies spatially as well as with time. In general, the braiding index and the overall width are large at the upstream part than further downstream, probably due to the effects of higher slope and grain sizes. The overall width of the river exhibits an increasing trend and there is tendency of shifting westwards, especially at the upstream part of the river within Bangladesh. The widening can be attributed to an advancing alluvial fan or to the not yet completed adaptation process after the shift to its new course according to FAP 24 (1994). The shifting rate of the first-order channel of the Jamuna River is 75 m to 150 m per year. The second-order channels change continuously, large channels being abandoned and new ones developing in a few years only (Klassen,G.J.and Masselink,G.,1992). A bank erosion rate of the second-order channels of 250 m to 300 m/year is common and in extreme cases, it can be more than 800 m/year (Klassen,G.J. et al, 1993). There are some attempts taken to address the river bank erosion protection issues (Rahman, M.L.et al, 2009). The river systems of Bangladesh are shown in Figure 2 in which so many agricultural land engulfed by the river systems.

Figure 1: River System of Bangladesh (Source: downnloaded from the webpage/ Internet)

Table1. Comparison among implementation cost of different river bank protection structures

Type of structure	Name of the River	Agency	Cost US$/m	Effectiveness
Guide Bank	Jamuna Bridge	Foreign	33,000	Effective
Hard point	Sirajganj (Jamuna)	Foreign	21,000	Effective
Solid spur	Kalitola (Jamuna)	Foreign	12,500	Effective
Revetment (Geobags)	Jamuna	Foreign	2000-3000	On going
Revetment	Jamuna	BWDB	3800-4000	70-80 %
RCC spur	Jamuna/ Ganges	BWDB	950	60-70 %
RCC spur	Teesta	BWDB	350	100 %
Bandalling structures	Jamuna Branch Channel	RRI	70	On going

It is mentioned here that the Bamboo Bandalling Structure is one of the low cost structures are shown in Table 1 in which the comparative statements of implementation costs for the different river erosion protection structures (Rahman, M.L. et al, 2009) are discussed. Although the land is usually owned by impecunious farmer and become the single place its life. It is to disturb sense of justice. Because above conditions, there have developed bamboo bandalling structure which is low-cost river bank protection, easy to construct, using natural and local materials, so that will harmonies with environment. Bank erosion and channel shifting of the untrained alluvial rivers of Bangladesh are big problems to the socio-economic and environmental sector of the country. During 1960's, a number of earthen embankments were constructed along the major rivers for the protection of rural people and agricultural lands from flooding. Since then the embankments were retired several times due to river bank erosion and bank protection are often required during the monsoon and post-monsoon season. Conventionally, groynes and revetments are applied as a method of bank protection. Very recently the concept of hard points (strong revetment type structure) at the most vulnerable locations along the Jamuna river are considered, while in between hard points spurs or permeable groynes are recommended (Klaassen, 2002). By applying the spurs or groyne type conventional structures, the river bank erosion at the short term basis can be obtained, whereas, the long term stable channel or regime channel can never be developed. Alternative solutions that can be locally adaptive and friendly to environment need to be developed for the whereas, the long-term stable channel or regime channel can never be developed. Alternative solutions that can be locally adaptive and friendly to environment need to be developed for the long-term stabilization of river channels. The possibility of using bandals for long-term channel stabilization is examined using field data and laboratory investigation (Rahman et al., 2003). The responses of large-scale alluvial rivers against sudden changes created by conventional structures are not suitable for the overall stabilization of river courses. Therefore, it is important to have alternative long-term solution for river stabilization that will create minimum disturbance to river courses.

2. METHODOLOGY

During dry season huge sediment deposited over the river bed & so the river conveyance capacity is reduced that accelerated the river near the bank which accelerated erosion during monsoon in every year. So it is important to protect the rivcr bank erosion to withstand agricultural land, homestead, hat-bazar market etc in Bangladesh. A series of bamboo bandals are constructed in the left bank of the Jamuna river near the upstream of the Bangabandhu Bridge. It was found that water flow diverted towards the main river due to bandal structures resulting low velocity near the river bank. Near the river bank, the water velocity is low which results sedimentation due to effect of bandal structures.

There was huge river bank sedimentation for the construction of bamboo bandalling structures by protecting the river bank erosion at the upstream of the Bangabandhu Bridge East Guide Bund near Shaheed Salahuddin Cantonment of the Bhuapur upazila under Tangail District, Bangaladesh. The selected problem area is about the 1.5 km river bank reach in which there are 138 cross-sections considered for the bathymetry data analysis as in the index map shown in Figure1. For its pictorial evidences, three reaches such as the 0.50 km downstream reach, 0.70 km middle and 0.3 km upstream reach within 1.5 km whole reach are considered for the analysis of effectiveness of bamboo bandalling structures constructed to protect river bank erosion in the problem area. The bed level, water level data are collected in this problem area by the bayhemetry survery in the form of sounding method with reference to a Temporary Bench Mark (TBM) near the Bangabandhu Bridge East Guide Bund.

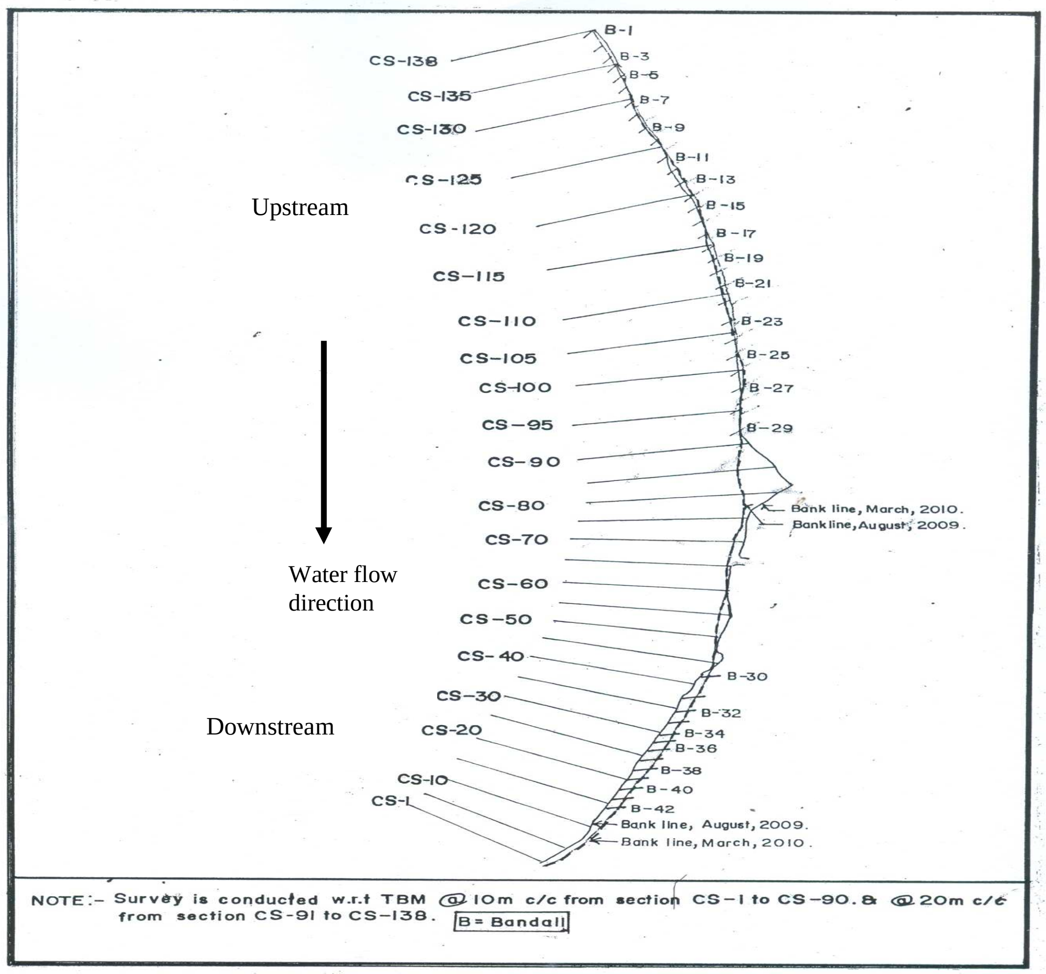

Figure 2: Location of river cross-sections in the left river bank towards main river

The working principles of bamboo bandals for the control of water and sediment flow where sediments are transported as bed load and suspended load. Within the lower half of the flow depth, major portion of the sediment flow is concentrated, whereas, within the upper half water discharges are more. Bandals are commonly applied to improve or maintain the flow depths for navigation during low water periods in alluvial rivers of Indian sub-continent. The essential characteristics of bandals are that they are positioned at an angle with main current and there is an opening below it while the upper portion is blocked. As an empirical rule the blockage of the flow section should be about 50% in order to maintain the flow acceleration. The surface current is being forced to the upstream face creating significant pressure difference between the upstream and downstream side of bandal. The flow near the bed is directed perpendicular to the bandal resulting near bed sediment transport along the same direction. Therefore, much sediment is supplied to the one side of channel and relatively much water is transported to the other side. The reduced flow passing through the opening of bandals is not sufficient to transport all the sediment coming towards this direction, resulting sedimentation over there.

Figure 3: Bamboo Bandalling is going on in the Jamuna River of Bangladesh

Figure 4: Crop Plantation within the recovered agricultural land

Figure 5: Recovery of agricultural land through river bank erosion protection

Figure 6: Paddy cropping in the agricultural land by river bank erosion protection

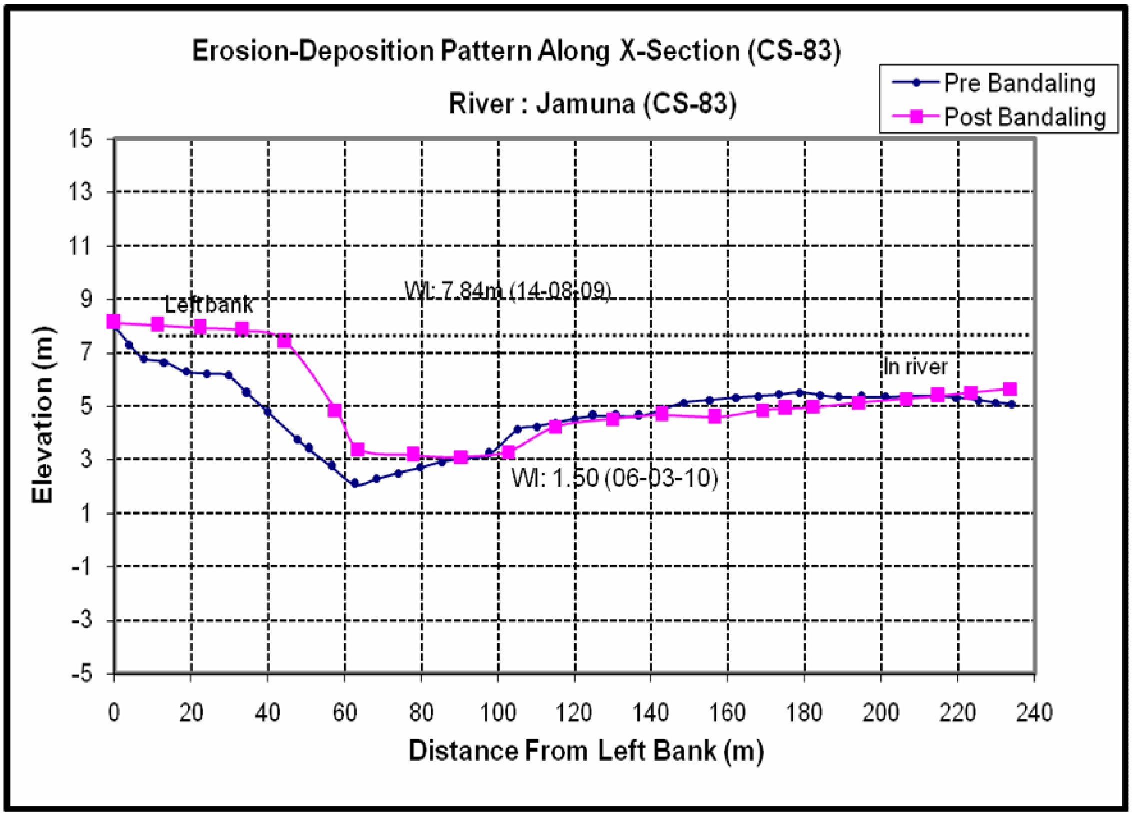

Figure 7. Agricultural land recovery by river bank erosion protection with bandalling

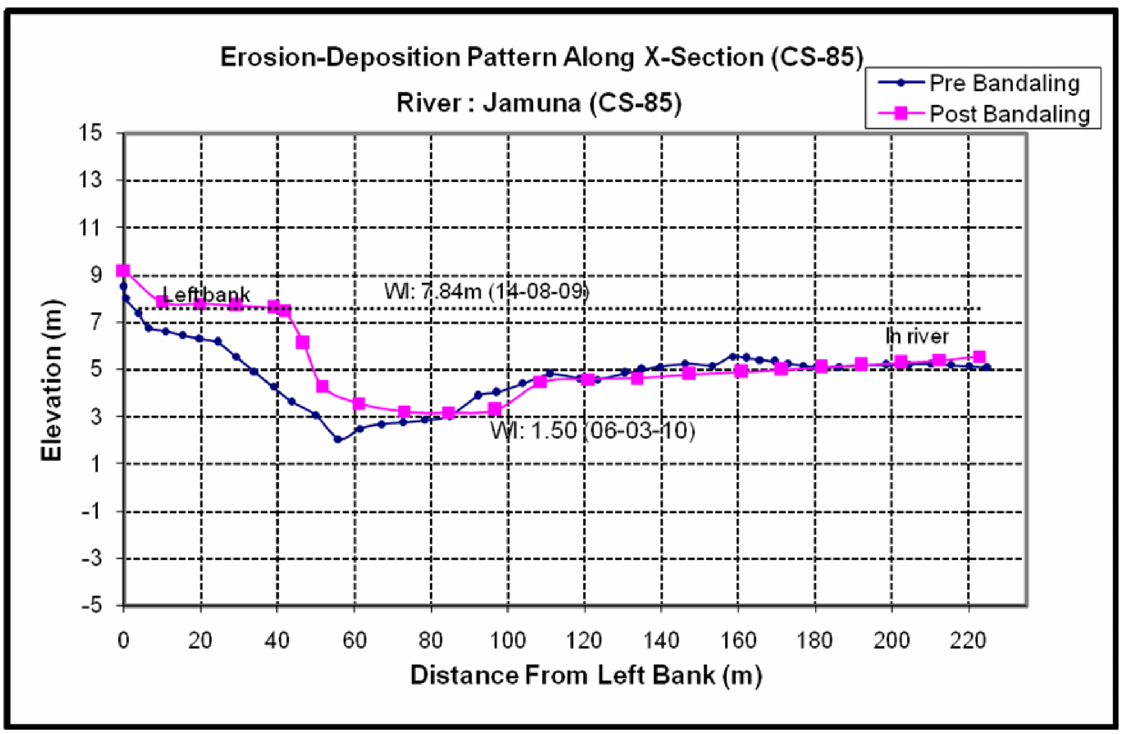

Figure 8. Agricultural land recovery by river bank erosion protection with bandalling

3. RESULTS AND DISCUSSIONS

Primarily collected river bathymerty data are analysed as well as the photographs are taken from the study area of the river. It is seen from the figure 3 so that the construction of bamboo bandalling construction is going on. It is found in figure 4 that the top of the surface water for the constructed bamboo bandalling is blocked by the bamboo fencing for which the velocity near the river bank is low than that of less velocity away from the river channel. It is appeared from figure 5 & figure 6 so that huge amount of sedimentation occurred in which the agricultural land is recovered during flood period due to effect of bamboo bandalling structures. In this recovered agricultural land, the crops are panted whose yield is higher than the normal agricultural land. Primarily collected river bathymetry data is plotted as in figure 7 & figure 8 that has also given us the erosion and siltation pattern which is the indication of the good performance of bamboo bandalling structures to protect river bank erosion as well as recovered agricultural land.

4. CONCLUSIONS

It is concluded that, due to construction of bamboo bandals, there is a siltation near the river bank where as there is deep pool away from the river bank. So it is obvious that the bamboo bandals are working as a river bank erosion protection & recovery of agricultural land with the aid of sedimentation near the river bank. So the bamboo bandalling structures are capable for protecting river banks by flow diversion towards the main channel leading to deep navigational channel formation in the main river. On the other hand, flow velocities are higher at the main channel increased the depth of the navigational channel that ensure the navigational channel development. If the bandal structure functions optimistically, the river can get sufficient time for its adjustment and new main channel and bank line development.

ACKNOWLEDGEMENT

The authors are grateful to the River Research Institute, Faridpur, and Government of Bangladesh for the financial assistance to conduct this study for such type of research.

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EXISTING FACILITY OF DRINKING WATER FOR A TYPICAL ARSENIC CONTAMINATED COASTAL AREA IN THE SOUTH-WESTERN REGION OF BANGLADESH

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ABSTRACT

In south-western part of the Bangladesh, deep grounder water aquifer contains high salinity and shallow depth is contaminated with arsenic pollution. The people in these areas are mainly depending on natural pond water (use a Pond Sand Filter (PSF)) and rainwater harvesting to meet their fresh water demand throughout the year. Therefore, this project was undertaken to study the present situation of fresh water in Paikgacha upazilla, Khulna, Bangladesh. The raw and treated water samples from six different PSFs are collected from upazilla Paikgacha (namely Village Paikgacha, Sheikhpara, Maloth, Gopalpur) and tested for water quality parameters (p^H, Color, Turbidity, Cl^-, DS, SS, TS, TC, E.coli and BOD_5). The analysis shows that the water quality parameters vary with the WHO and BDS allowable limits and the quality of the raw water is extremely poor. For treated water values of p^H, turbidity, DS and TS are within the WHO and BDS allowable limit. In the most cases values of chloride ion concentration, SS and colour are exceeding the WHO and BDS allowable limit. The total coliform, E.coli and BOD_5 values represents that the treated water of the study area is unsuitable for drinking purpose as almost all the samples exceeds WHO and BDS allowable limit. The performance of existing pond sand filters are not satisfactory as a result, the local peoples are frequently suffering by several water born diseases.

Keywords: *Arsenic, Pond Sand Filter (PSF), Salinity, Water quality parameters, World Health Organization (WHO.)*

1. INTRODUCTION

Water is one of the vital components of the physical environment. There is an almost unfathomable amount of water on earth: about 1.4 billion km^3 (330 million cubic miles) (Barlow and Clark, 2002). About 97% of the earth's water is salt water in the oceans and a tiny 3% (about 36 million km^3) is fresh water contained in the poles in the form of ice, ground water, lakes and rivers, which supply most of human and animal needs (Ahmed and Rahman, 2000). Nearly, 70% from this tiny 3% of the world's fresh water is frozen in glaciers, permanent snow cover, ice and permafrost. Thirty per cent of all fresh water is in underground, most of it is in deep, hard-to-reach aquifers. Lakes and rivers together contain just a little more than 0.25% of all fresh water; lakes contain most of it (Kalogirou, 2005). The data relevant to water requirements shows that around 25% of the total world populations do not have an adequate fresh water supply, both for quality and quantity (Agha et. al., 2005). The quality of drinking water is closely associated with human health, and providing safe drinking water is one of important public health priorities. Estimated 80 per cent of all diseases and over one third of deaths in developing countries are caused by the consumption of contaminated water, and on an average as much as one tenth of each person's productive time is sacrificed to water-related diseases (UNCED, 1992).

It is estimated that approximately one-third of the world's population use groundwater for drinking (Nickson et al., 2005). However, in the coastal regions in any country, groundwater quality patterns are complex because of the input from many different water sources. These include precipitation, seawater, ascending deep groundwater, and anthropogenic sources such as wastewater or irrigation return flow (Ramkumar et. al., 2010). In recent years, peoples living in coastal, remote or arsenic contaminated areas of Bangladesh have been received global attention due to scarcity of fresh water for their living purpose. Contaminated wells exceeding the Bangladesh standard of 0.05 mg/L have been identified in 41 of the country's 64 districts. It has been estimated that a population of 25 to 36 million are exposed to arsenic contamination and related health risks. The main sources of water in Bangladesh are surface waters in rivers, reservoirs, lakes, canals and ponds, and ground water in deep and shallow aquifers. There are certain areas in the coastal belt of Bangladesh where tube wells are not successful, because ground water is mostly saline upto depths of 700-1000 ft. (DPHE-UNICEF, 1989) and suitable freshwater aquifers are not available. In addition to this, arsenic contamination of ground water in Bangladesh has been recognized as major problem since 1993. The concentration of arsenic in excess of allowable limits has been found in shallow tubewells in many parts of Bangladesh. Thousands of people have already been identified to be affected by arsenic poisoning, in addition to the millions potentially under threat from drinking contaminated water (Ahmed and Rahman, 2000). Provision of arsenic free water is urgently needed for immediate protection of health and well being of the people living in arsenic affected areas. However, to ensure the safe water for drinking and all other household usage, the concerned persons and organizations (Department of Public Health Engineering (DPHE)-Danida, UNICEF, WHO, NGO Forum etc.) are searching for appropriate alternative technology for the coastal belt. Pond sand filter (PSF)s were first introduced by DPHE-UNICEF jointly to overcome the problem since 1984 on a pilot basis. The NGO Forum has also been implementing PSF and rainwater harvesting system since 1997 in the coastal area of Bangladesh. The people in many places of southern fringes of Khulna, Noakhali, Bagerhat, Satkhira, Barguna, Perojpur, and Patuakhali districts are mainly depending on PSF as community base system and rainwater harvesting as household base system to meet their fresh water demand throughout the year. In this study, Paikgacha a typical arsenic contaminated Upazilla in the coastal belt of the south-western region of bangladesh has been investigated to study the present situation of fresh water condition for their living purpose.

2. METHODOLOGY

For better understanding the present scenario of existing drinking water facilities in a typical arsenic contaminated coastal region, Upazilla Paikgacha has been selected for the present study. To identify the technical and social problems of existing facilities a detail field investigation has been carried out. Raw water and filtered water samples have been collected from selected five PSF's and water quality parameters such as total coliform, fecal coliform, turbidity, salinity, pH, TS, TDS, SS, alkalinity and color have been tested.

The steps of the methodology are shown in the flow chart given in Figure 1.

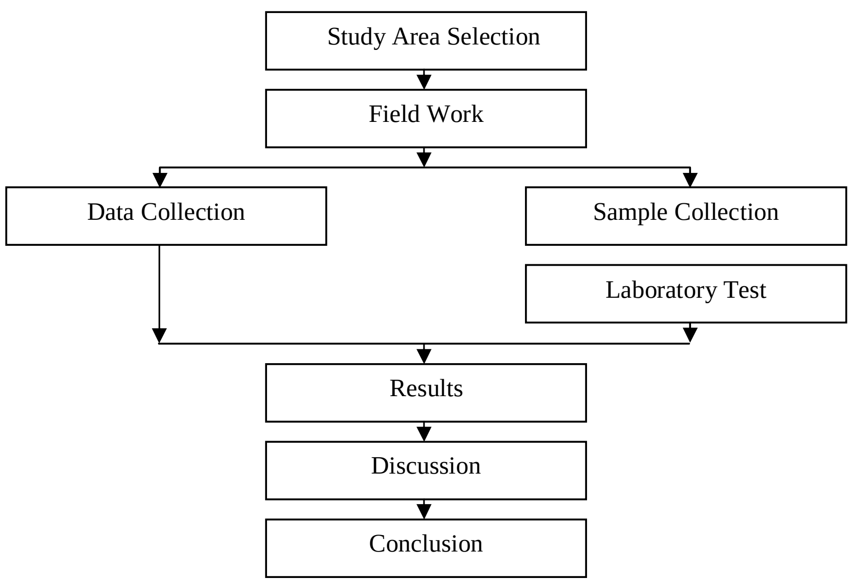

Figure 1: Flow chart showing the steps of the methodology

3. STUDY AREA AND DATA COLLECTION

For better understanding of the present scenario of existing drinking water facilities upazilla Paikgacha under Khulna district has been selected for the present study. To identify the technical and social problems of existing facilities a detail field investigation has been carried out. Paikgacha is one of the typical arsenic contaminated Upazilla in the south-west coastal belt of Bangladesh. It is located about 62 kilometers west of the district headquarters of Khulna and about 350 kilometers south from the capital Dhaka. Paikgacha upazilla is divided by the river, Shipsa and the household of this area are mainly living on both side of the river. The land of this area is defined as medium high land, and the soil quality is alluvial, loamy and sandy. The demographic characteristics of the area are similar to any other coastal region of Bangladesh. The available sources of water are ground water and surface water. The ground water contains excessive arsenic and salinity which exceeds the allowable limits. So the ultimate source is the surface water namely pond water. The pond water is being used by filtration called pond sand filter. Many pond sand filters is being chocked up during rainy season and polluted through the natural pollutants. The upazilla is a flood affected zone. Almost every year flood is occurred and flowing through the alluvial plain. In recent two natural storms namely Sidor and Aila had occurred and inundate the whole area. The resulting effect of Aila over stress fresh water supply of that area. Also this flood water mixed with various waste and increase coliform and salinity. In this situation the pond water is totally unhygienic for drinking. Use of such contaminated water is the cause of various diseases like diarrhea, dysentery, cholera among rural people, which results a chronic impact over rural health and economy of Bangladesh.Recently rain water harvesting is alternative source of drinking water. The area of Paikgacha is 35.76 km^2, the no of household is 55,664 and overall population is 2,95,732. Details of Paikgacha upazilla are given in Table 1.

Table 1: Details of Paikgacha

Union	Area (km²)	No. of Family	Population	Details of PSF (as on March 2011)		
				Running	Chocked up	Total
Horidali	18.75	5917	30886	12	1	13
Kopilmoni	37.85	6995	38192	36	4	40
Lata	43.47	2739	14379	11	5	16
Deluty	43.58	3896	19805	11	3	14
Soladana	44.47	5179	26468	40	13	53
Laskar	41.92	3961	25050	12	4	16
Gadaipur	18.79	5511	28215	13	7	20
Raruli	21.56	6444	35452	9	5	14
Chandkhali	40.37	10147	51954	34	14	48
Goraikhali	47.00	4875	25341	21	9	30
Total	357.76	55664	295732	189	65	264
In addition 380 household rain water harvesting system also exists.						

Six PSF's located at village Sheikhpara (Latitude 22°38'59"N, Longitude 89°18'32"E), Maloth (Latitude 22°39'08"N, Longitude89°18'40"E), Gopalpur (Latitude22°36'11"N, Longitude 89°18'32"E), Gopalpur (Latitude22°36'12"N, Longitude 89°18'12"E), and Paikgacha (Latitude22°35'00"N, Longitude 89°19'46"E), under Paikgacha upazilla have been selected on the basis of preliminary investigation. It is to be mentioned here that all the PSFs were constructed by DPHE-Unicef. A map shown the study area and PSF location is given in Figure 2. Raw and treaed water for the selected PSF's were collected from the month of July 2010 to February 2011 to cover both monsoon and non-monsoon period in a year. Plastic bottles of 500 ml capacity were used for this purpose. They were thoroughly cleaned by rinsing 3 to 4 times with sampling water. Before sampling from well, sufficient amount of water was pumped out so that the sample represents the GW from which the well is fed (Raghunath, 1990). Then the containers were filled up to the overflow and were immediately sealed to avoid contact with air. In pond sand filter at Sheikhpara, Gopalpur-1 and Gopalpur-2 tube well water were used as raw water and at Maloth, Paikgacha-1, and Paikgacha-1 pond water were used as raw water. Then the collected samples were tested in Environmental Engineering laboratory of KUET, Khulna, Bangladesh for water quality parameters.

Table 2: GPS locations of samples collected area

Location Name	Latitude	Longitude
Sheikhpara	22°38'59" N	89°18'32"E
Maloth	22°39'08"N	89°18'40"E
Gopalpur1	22°36'11"N	89°18'32"E
Gopalpur2	22°36'12"N	89°18'12"E
Paikgacha Madrasa (old)	22°35'00"N	89°19'46"E
Paikgacha Madrasa (new)	22°35'00"N	89°19'46"E

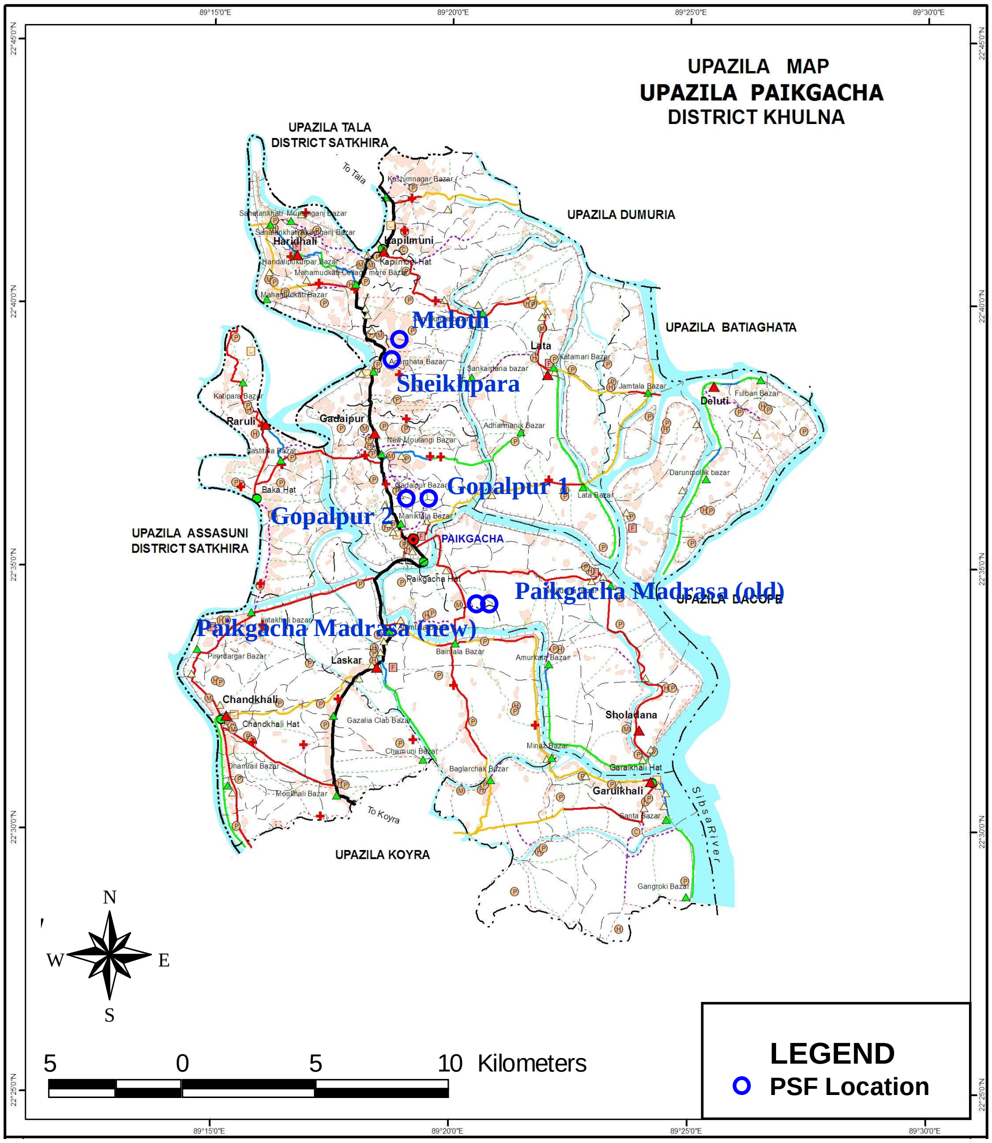

Figure 2: Existing locations of pond sand filter in Paikgacha upazilla

2.2 Collection and testing of water samples

Pond sand filter is a small scale-filtering device having manually operated treatment units used to treat the raw water (from tube well and pond) based on the principle of slow sand filtration. The filter consists of a 4'x 4'x4.5' brick chamber filled with a 3 A. layer of sand and a I ft. layer of graded gravel. The raw and treated water samples from six different pond sand filters are collected from Upazilla Paikgacha (namely village Sheikhpara (Latitude 22°38’59”N, Longitude 89°18’32”E), Maloth (Latitude 22°39’08”N, Longitude 89°18’40”E), Gopalpur-1 (Latitude 22°36’11”N, Longitude 89°18’32”E), Gopalpur-2 (Latitude 22°36’12” N, Longitude 89°18’12”E), Paikgacha-1 (Latitude 22°35’00”N, Longitude 89°19’46”E) and Paikgacha-2 (Latitude 22°35’00”N, Longitude 89°19’46”E)) from the month of July 2010 to February 2011 to cover both monsoon and non-monsoon period in a year. Plastic bottles of .5-liter capacity were used for this purpose. They were thoroughly cleaned by rinsing 3 to 4 times with sampling water. Before sampling from well, sufficient amount of water was pumped out so that the sample represents the GW from which the well is fed (Raghunath, 1990). Then the containers were filled up to the overflow and were immediately sealed to avoid contact with air (Clesceri et al. 1989). The containers were labeled including its station name, source, number, date & time for identification and brought to the Environmental Engineering Laboratory of KUET. In pond sand filter at

Sheikhpara, Gopalpur-1 and Gopalpur-2 tube well water were used as raw water and at Maloth, Paikgacha-1, and Paikgacha-1 pond water were used as raw water. Then the collected samples were tested in Environmental Engineering laboratory of KUET, Khulna, Bangladesh for water quality parameters such as total coliform, faecal coliform, turbidity, salinity, pH, TS, DS, SS, BOD_5 and colour have been tested. TDS, EC and p^H were determined by TDS meter, Conductivity meter and p^H meter respectively. Color and turbidity were measured by Spectrophotometer, Hellige turbid meter respectively. Chloride was determined by titration method. For chloride test 0.0141N $AgNO_3$ with K_2Cr_2O7 indicator was used. All the measured values of water quality parameters are also compared with the WHO and BDS for evaluating the vulnerability.

4. RESULTS AND DISCUSSIONS

3.1 Test result of the collected water samples

The collected raw and treated water samples from the selected PSF's were tested in Environmental Engineering laboratory of KUET, Khulna, Bangladesh for water quality parameters such as total coliform, faecal coliform, turbidity, salinity, pH, TS, DS, SS, BOD_5 and colour have been tested. TDS, EC and p^H were determined by TDS meter, Conductivity meter and p^H meter respectively. Color and turbidity were measured by Spectrophotometer, Hellige turbid meter respectively. Chloride was determined by titration method. For chloride test 0.0141N $AgNO_3$ with K_2Cr_2O7 indicator was used. All the measured values of water quality parameters are also compared with the WHO and BDS for evaluating the vulnerability. The measured value of different physical and chemical water quality parameters of the collected water samples is summarized in Table 3, Table 4 and Table 5. PSF at Paikgacha Uttarpara Madrasa -2 was not in function from the month January 2011, therefore, values of water quality parameters are determined only for raw water (pond water) at Paikgacha -2 (Table 4 and Table 5). Findings of the study have been assessed according to WHO drinking water quality guidelines and Bangladesh Standards (BDS) as well.

Table 3: Test result of water sample collected in 30th July, 2010

Location	Water	P^H	Color (Pt-Co)	Turbidity (NTU)	Parameters (mg/L)					Coliform (No./100ml)	
					Cl-	TS	DS	SS	BOD_5	TC	EC
Sheikhpara	TW	8.16	32	2.25	160	340	320	20	0.85	151	123
	RW	8.10	45	5.08	240	390	320	70	1.36	117	96
Maloth	TW	8.07	25	0.85	50	350	310	40	0.84	130	23
	RW	8.03	114	11.9	160	320	300	20	1.33	69	51
Gopalpur-1	TW	7.79	22	1.65	30	450	410	40	0.32	118	76
	RW	7.47	105	19.5	170	510	410	100	2.21	78	44
Gopalpur-2	TW	7.58	24	0.9	150	700	590	110	0.63	103	98
	RW	7.32	8	6.11	350	850	780	70	2.2	34	28
Paikgacha Uttarpara Madrasa -1	TW	8.56	16	2.36	120	850	850	0	0.6	88	11
	RW	8.59	146	33.1	240	920	890	30	4.46	20	12
Paikgacha Uttarpara Madrasa -2	TW	8.59	0	3.74	230	840	830	10	0.9	123	3
	RW	8.56	295	45.3	480	880	800	80	1.11	52	41
Allowable limit		6.5~8.5	15	10	600	1000	1000	0	0.2	0	0

Table 4: Test result of water sample collected in 21st January, 2011

Location	Water	p^H	Color (Pt-Co)	Turbidity (NTU)	Parameters (mg/L)					Coliform (No./100ml)	
					Cl-	TS	DS	SS	BOD_5	TC	EC
Sheikhpara	TW	8.01	13	4.19	190	520	340	180	0.43	181	107
	RW	8.15	197	16.7	290	430	320	110	1.65	153	125
Maloth	TW	8.01	5	0.61	90	410	370	40	0.66	169	57
	RW	8.10	117	14.0	210	490	350	140	0.96	109	89
Gopalpur-1	TW	7.74	155	0.94	80	420	310	110	0.27	176	98
	RW	7.47	68	11.7	300	450	340	110	3.74	147	63
Gopalpur-2	TW	7.79	0	4.21	200	880	630	250	1.51	148	126
	RW	7.34	119	29.4	690	1590	1270	320	0.84	67	46
Paikgacha Uttarpara Madrasa -1	TW	8.41	102	4.80	150	740	720	20	0.40	149	54
	RW	8.45	495	10.7	380	800	710	90	2.09	77	41
Paikgacha Uttarpara Madrasa -2	TW	X	X	X	X	X	X	X	X	X	X
	RW	8.45	495	10.7	380	800	710	8.45	3.44	85	53
Allowable limit		6.5~8.5	15	10	600	1000	1000	0	0.2	0	0

Table 5: Test result of water sample collected in 21st February, 2011

Location	Water	p^H	Color (Pt-Co)	Turbidity (NTU)	Parameters (mg/L)					Coliform (No./100ml)	
					Cl-	TS	DS	SS	BOD_5	TC	EC
Sheikhpara	TW	7.67	20	2.26	210	480	310	170	0.55	223	168
	RW	7.46	93	7.84	350	520	350	170	1.79	141	89
Maloth	TW	7.45	28	5.99	120	360	290	70	0.71	211	117
	RW	7.40	98	31.7	280	440	260	180	1.12	178	124
Gopalpur-1	TW	6.81	19	3.50	160	880	840	40	0.24	226	118
	RW	7.90	92	5.11	420	860	840	20	2.87	183	93
Gopalpur-2	TW	7.30	8	0.33	270	540	450	90	1.68	175	167
	RW	6.93	212	27.5	740	480	460	20	2.91	101	73
Paikgacha Uttarpara Madrasa -1	TW	7.94	47	7.58	280	920	860	60	1.07	194	83
	RW	8.07	361	37.5	510	950	920	30	2.37	98	49
Paikgacha Uttarpara Madrasa -2	TW	X	X	X	X	X	X	X	X	X	X
	RW	8.05	412	52.3	1020	760	710	50	1.62	121	67
Allowable limit		6.5~8.5	15	10	600	1000	1000	0	0.2	0	0

3.2 Variation of pH

The p^H value is an important index of acidity or alkalinity and the concentration of hydrogen ion in ground waters (Murugesan et al., 2006). Figure 3 shows, the seasonal variation of pH value of collected water samples. p^H values of almost all the water samples were in the permissible range of 6.5-8.5 according to World Health Organization (WHO) guideline values and standard values prescribed in Bangladesh Environment Conservation Rules' 1997 (BECR'97). Figure 3 shows; the P^H value of water is maximum for the month July 2010. The survival of aquatic organisms is also greatly influenced by the p^H of water bodies because most of their metabolic activities are p^H dependent (Chen and Lin, 1995). A controlled value of p^H is desired in water supplies, sewage treatment and chemical process plants. In water supply p^H is important for coagulation, disinfection, water softening and corrosion control.

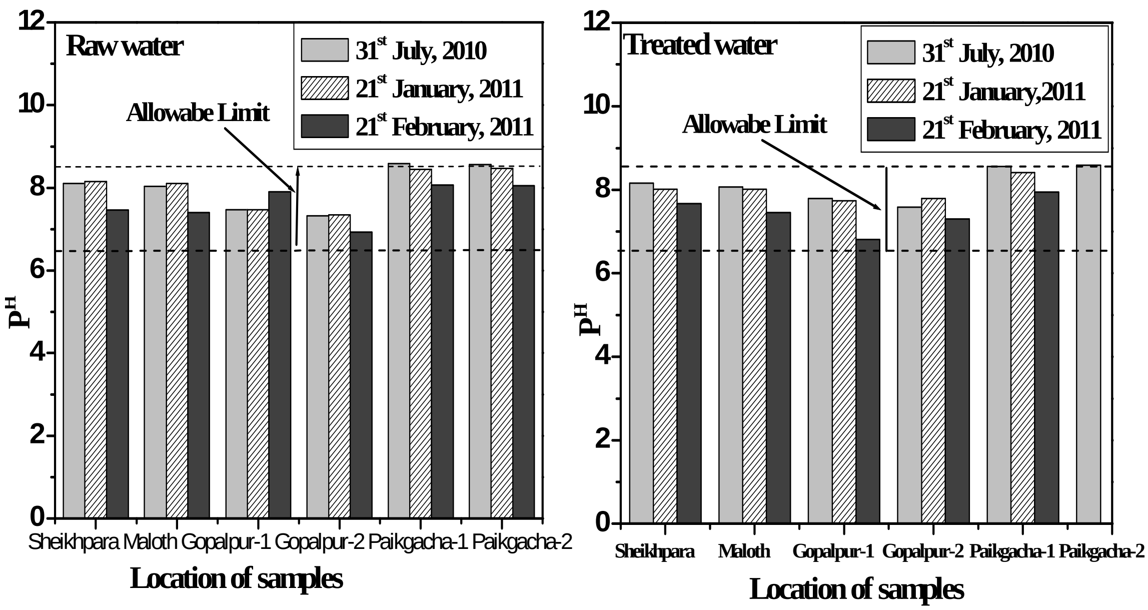

Figure 3: Comparison of p^H value of collected water samples

3.3 Assessment of color

Color in water is primarily due to the presence of colored organic substances (primarily humic substances), metals such as Fe, Mn or highly industrial wastes. The comparison of color of raw and treated water are shown in Figure 4. For raw water in Figure 4, it is shown that the values of color are exceeding the allowable limit for almost all the samples. On the other hand for treated water it is also shown that almost all water samples exceeds the permissible limit (15 Pt.co) of groundwater color in accordance with WHO allowable limit. So it is observed that, there is less improvement of color in treated water.

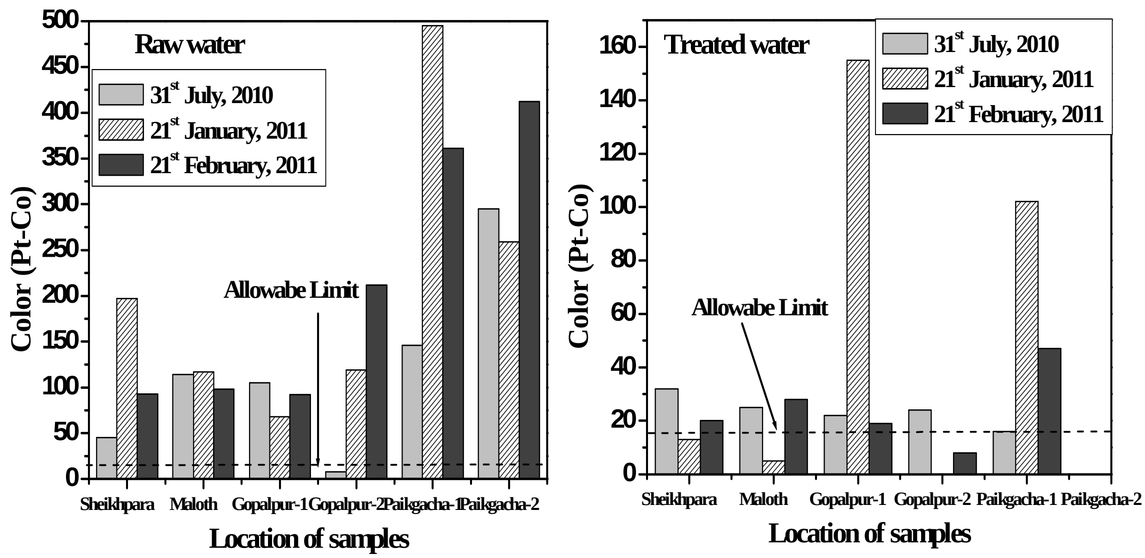

Figure 4: Comparison of color value of collected water samples

3.4 Assessment of turbidity

The comparison of turbidity of raw and treated water are shown in Figure 5. For raw water in Figure 5 it is shown that, most of the cases values of turbidity are exceeding the allowable limit. On the other hand for treated water 100% samples turbidity values are within the WHO and Bangladesh Standards (BDS) allowable limit. So it is observed that there is significant improvement of turbidity in treated water.

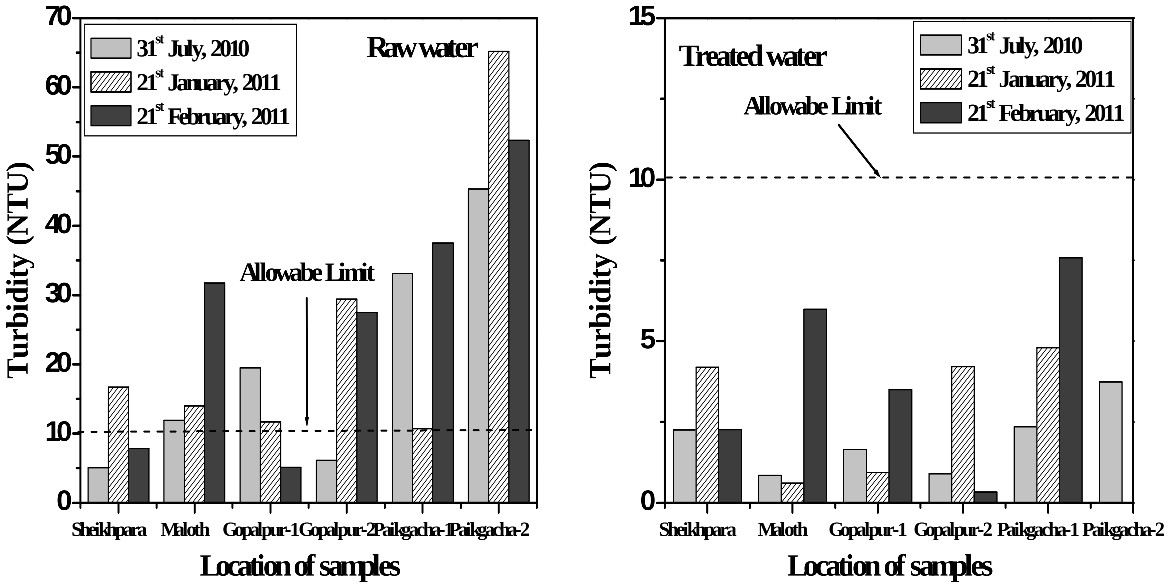

Figure 5: Comparison of turbidity value of collected water samples

3.5 Assessment of salinity

The chloride ion concentration is gradually increase from July to February are shown in Figure 6. For raw water maximum chloride ion concentration is 1020 mg/L and for treated water maximum chloride ion concentration is 280mg/L (21st February, 2011) (Table 5). The seasonal variation of salinity in the KCC area reveals that salinity has been started to increase gradually from November due to recharge of aquifers by saline water (Adhikary et al., 2011) and in the month of February, it has been increased to a high value (Figure 6). Most chloride in ground water comes from evaporation, salty connate water and seawater (Nasrin et. al., 2005). Ground water containing significant amount of chloride also tend to have high amount of Na^+ indicating the possibility of contacts with water of marine origin (Hem, 1970). At concentration above 250 mg/L chloride rich water gives a salty test, causes various diseases such as high blood pressure etc but it depends on individual adaptability (Sawyer and McCarty, 1978).

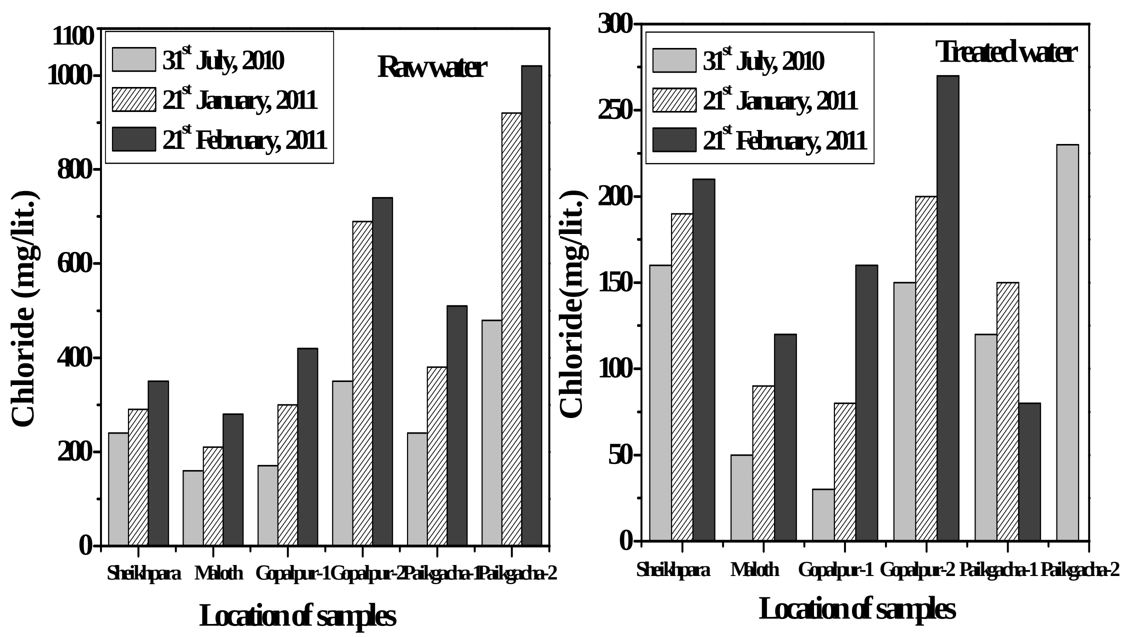

Figure 6: Comparison of chloride value of collected water samples

3.6 Total dissolved solids (TDS)

The comparison of total solids of raw and treated water are shown in Figure 7. During the study period, 11% raw water samples exceed the permissible limits (1000 mg/L) of WHO and BECR '1997 throughout the study period. Depending on the TDS values, ground water is to be grouped as excellent, good, fair, poor and unacceptable (Ahmed and Rahman, 2000). Most of the samples lie between good (300-600 mg/L) and fair (600-900 mg/L) class. Total solid of the treated water samples were well below the permissible value.

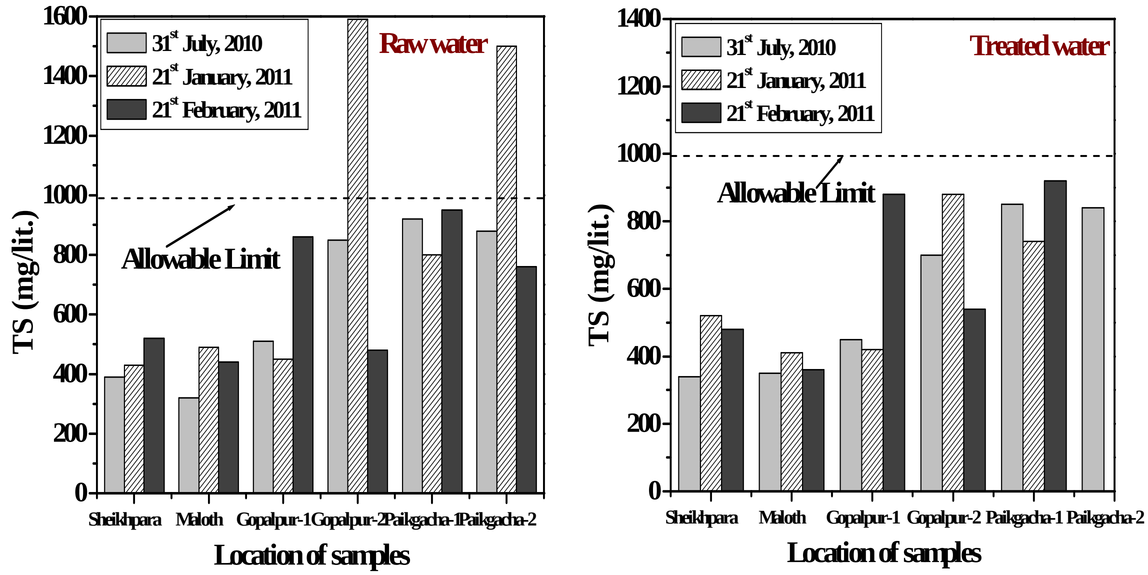

Figure 7: Comparison of total solid value of collected water samples

3.7 Dissolved solids (DS)

The comparison of dissolve solids of raw and treated water are shown in Figure 8. For raw water in Figure 8 it is shown that the values of dissolve solids are within the allowable limit except in 21st January, 2011 at Gopalpur-2. On the other hand for treated water in Figure 8 it is shown that the values of dissolve solids are within the allowable limit in all locations.

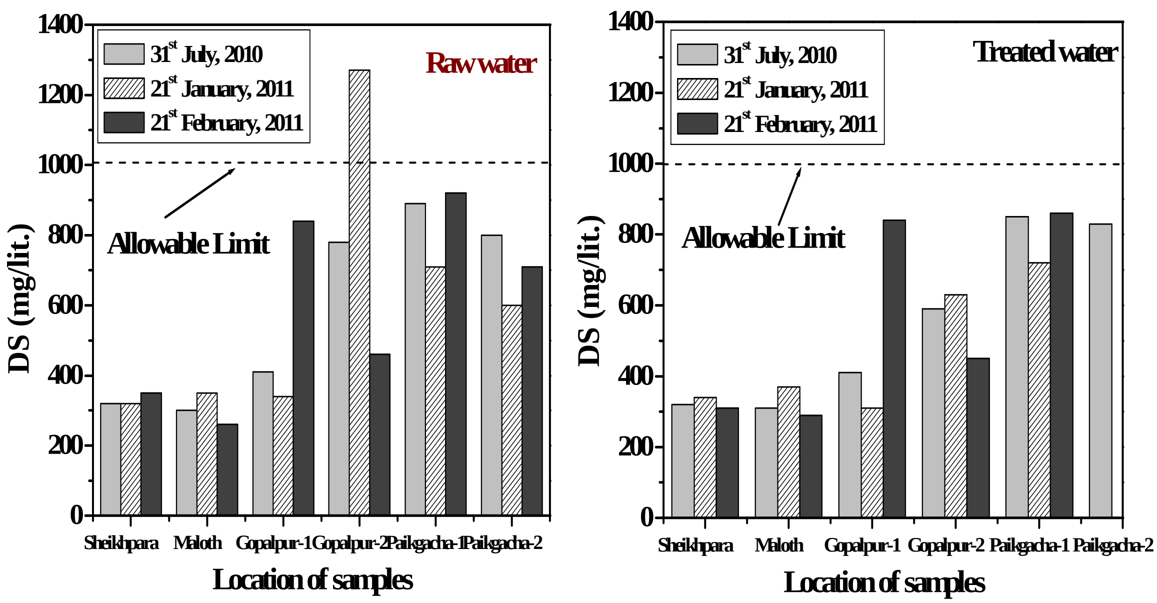

Figure 8: Comparison of dissolve solid value of collected water samples

3.8 Suspended solids (SS)

The comparison of suspended solids of raw and treated water are shown in Figure 9. For raw water the values of suspended solids are exceeding the allowable limit all the cases. On the other hand for treated water it is shown that the values of suspended solids are exceeding the allowable limit in almost all the cases.

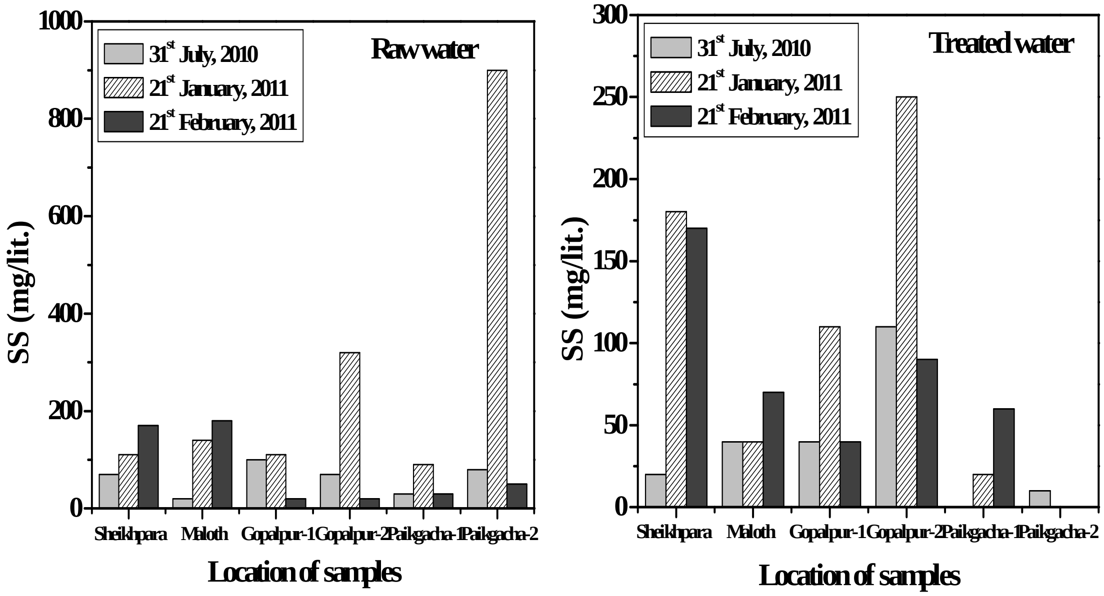

Figure 9: Comparison of suspended solid value of collected water samples

3.9 E.coli

The comparison *of E.coli* of raw and treated water are shown in Figure 10. For raw water and treated water in Figure 10 it is shown that the values of E.coli are exceeding the allowable limit (0 No/100 ml). So it is observed that there is less or no improvement of E.coli value in treated water.

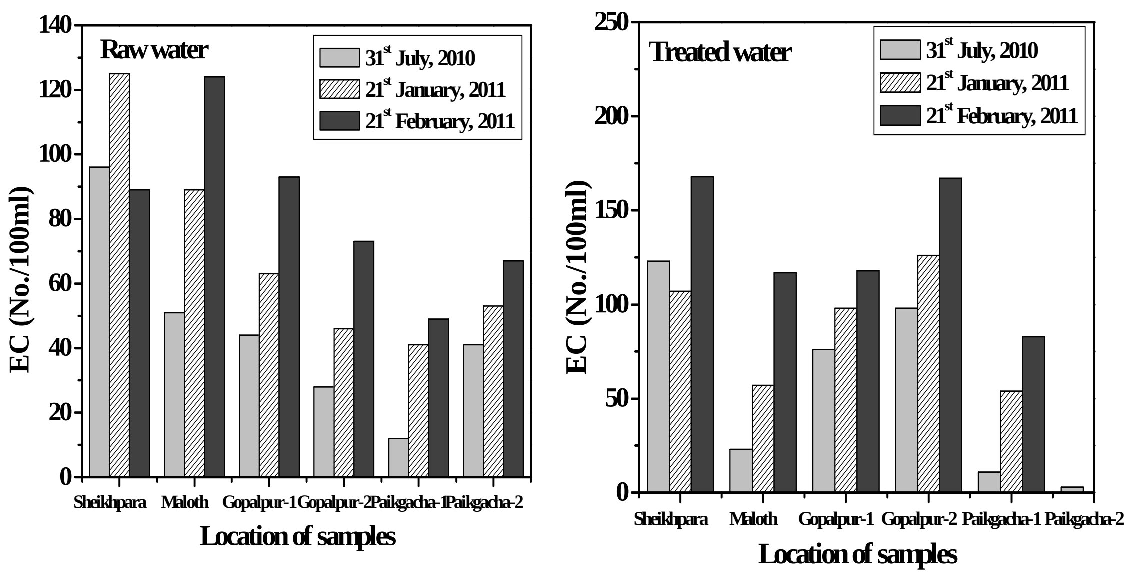

Figure 10: Comparison of E.coli value of collected water samples

3.10 Total coliform (TC)

The comparison of total coliform of raw and treated water are shown in Figure 11. For raw water and treated water the values of total coliform are exceeding the allowable limit (0 No/100 ml) all the cases. So it is observed that there is no improvement of total coliform in treated water.

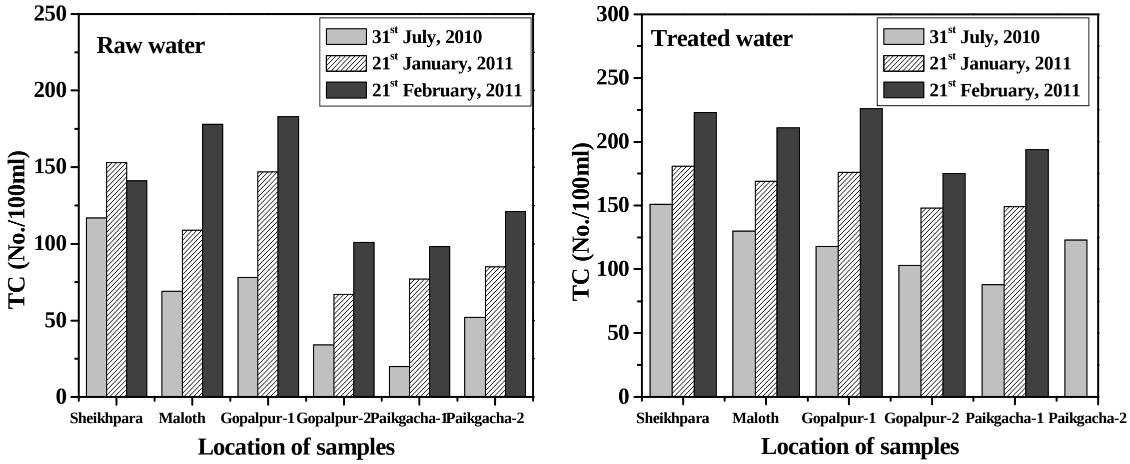

Figure 11: Comparison of total coliform value of collected water samples

3.11 Biochemical oxygen demand (BOD_5)

The comparison of BOD_5 of raw and treated water are shown in Figure 12. BOD_5 value of almost all the collected samples was significantly higher than allowable limit. So it is observed that there is no improvement of BOD_5 in treated water.

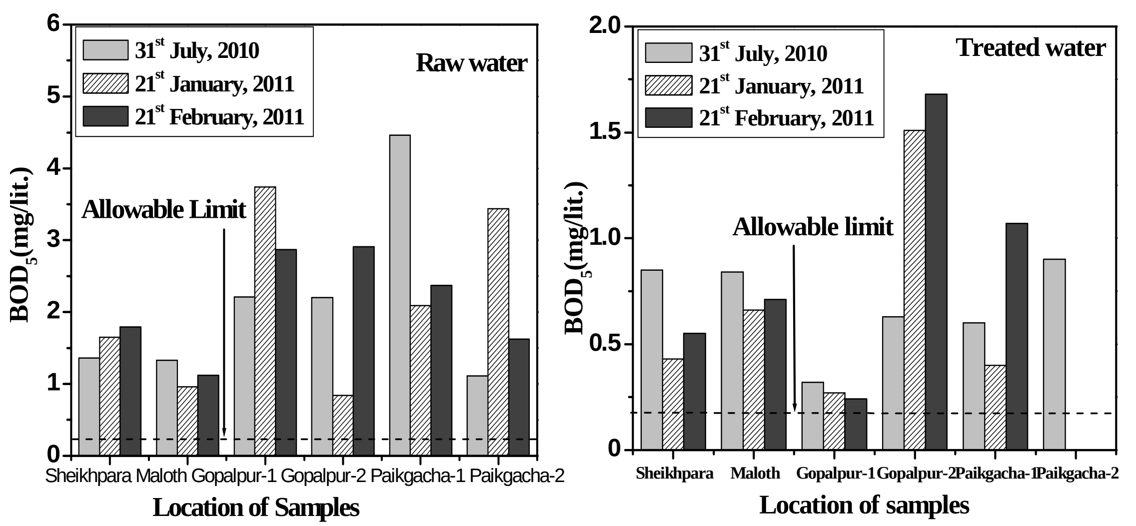

Figure 12: Comparison of BOD_5 value of collected water samples

5. CONCLUSIONS

Water samples collected from the selected pond sand filters were tested from July 2010 to February 2011. The analysis shows that the water some quality parameters vary with the World Health Organization (WHO) and Bangladesh Standards (BDS) allowable limits and the quality of the raw water is extremely poor. The study shows that, only 12% of treated water samples p^H value exceeds BECR '1997 and WHO standard. The color values of the treated water represents that almost all the samples are exceeding the WHO and BDS allowable limit. Turbidity, dissolve solids and total solids values of 100% treated water samples are within the WHO and Bangladesh Standards (BDS) allowable limit. The major finding of the study was evaluating of salinity (chloride ion concentration) of raw water and during the study period 69% samples exceeds WHO standard whereas 13% samples exceeds BECR '1997 allowable limits, which reveals that the study area have been facing tremendous salinity problem and water is unsuitable for drinking. In the most cases values of suspended solids are exceeding the WHO and BDS allowable limit although there is some improvement of suspended solid values in treated water. The total coliform, E.coli and BOD_5 values represents that the treated water of the study area is unsuitable for drinking as all the samples exceeds WHO and BDS allowable limit. It is concluded that suspended solid, total coliform, E.coli, color, Cl^- and BOD_5 is the main problem in the selected coastal areas. The performance of existing pond sand filters are not satisfactory as a result, the local peoples are frequently suffering by several water born diseases. This study also reveals that performance of PSFs are declining due to some design faults including insufficient depth of filter bed, inadequate pre-filter chamber, insufficient outlet structure to prevent the negative pressure in the filter bed, lack of awareness and reluctance among the beneficiaries. Government and NGOs should take necessary technical and social steps to improve the efficiency of PSFs.

ACKNOWLEDGEMENTS

The authors wish to express their acknowledgements to the Department of Civil Engineering, Khulna University of Engineering & Technology (KUET), Khulna, Bangladesh for extending all sorts of supports to use the laboratory facilities for this study.

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DISASTER MANAGEMENT IN BANGLADESH

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ABSTRACT

Recent events such as floods, earthquakes, tsunamis, hurricanes and other threats of pandemics have increased our vulnerability towards natural disasters. These days Bangladesh has been prone to more natural disasters than in the last three decades. Now its time we should take necessary precautions to mitigate the impact of these disasters. The consequences of natural and man made disasters and the vulnerabilities to which population are exposed can be mitigated if proper planning and management can be provided. However, it is not always completely possible to eliminate one risk, but it can be minimized by careful planning, mitigations and prompt actions. These all can be explained in one term Disaster Management. This paper discussed the definition of disaster, disaster management, different phases of disaster management (mitigations, risk reduction, prevention, preparedness, response and recovery), disaster faced by Bangladesh, consequences of these disasters, implementation of disaster management. Disaster is no more a new topic for us nowadays. But tackling disaster with full preparation is the major agenda. Though various measures are taking from different organizations (government and non-government organization), we need a full strategy to handle the situation. The main purpose of this paper is to introduce disaster management focusing on reduction of disaster risk, which will have a significant contribution to the goal of protecting vulnerable communities, their lives, assets and livelihoods as well as smoothing the path of development. It includes both pre planning and post planning.

Keywords: *Disaster, Disaster Management, Disaster Risk Management, Vulnerability Reduction, Risk Assessment.*

1. INTRODUCTION

Bangladesh is situated in the 24 00 N, 90 00 E and due to its geographical condition and as one of the most crowded countries on the planet, much of the lush, low-lying landscape is subject to yearly flooding, and the subsequent devastation of cyclones. In recent decades the global warming has aggravate the situation. Less than 25% of Bangladesh is only 1 meter above sea level. The United Nations Inter-governmental Panel on Climate Change has predicted Bangladesh could lose nearly one-fifth of its land by 2050 because of rising sea levels due to global warming. Bangladesh heads the list of the countries most at risk of flooding. Hence, disaster management has become a vital issue in our country. Here different disaster and disaster management in Bangladesh are discussed.

2. DISASTER

A disaster is a natural calamity or hazard. The root of the word *disaster* comes from the Greek word, which means ‘bad star’. It also has an astrological theme in which the ancients used to refer to the destruction or deconstruction of a star as a disaster. In a recent document published by the United Nations Development Program (UNDP) in the Americas, a disaster is defined as ‘a social crisis situation occurring when a physical phenomenon of nature, socio-nature or anthropogenic origin negatively impacts vulnerable populations…causing intense, serious and widespread disruption of the normal functioning of the affected social unit’. Another widespread definition is ‘disasters occur when hazards strike in vulnerable areas’. Therefore, it is clear that a hazard or natural calamity is not necessarily being a disaster until it affects human lives, infrastructure, production etc. For example, an undersea earthquake might not result in the loss of any life or damage to property, and a typhoon is not disaster until heavy rain and wind cause damage or disruption to inhabited areas. In addition, a specific disaster may spawn a secondary disaster that increases the impact. A classic example is an earthquake that causes a tsunami, resulting in coastal flooding.

3. DISASTERS IN BANGLADESH

Bangladesh is the most vulnerable country to climate change impact. Being the largest delta in the world located at the downstream of the second largest river system, the country is subject to a series of natural disaster. This Country is frequently hit by cyclones, floods, mudslides, tornados, river erosion, earthquake and drought. Each year our country faces any of this disaster. By 2050, 70 million people could be affected annually by floods; 8 million by drought; up to 8% of the low-lying lands may become permanently inundated (From USAID). Here common disaster faced by Bangladesh will be discussed in brief.

3.1 Cyclones

Bangladesh has approximately 710 kilometers of coastline, which leaves huge tracts of land open to the destructive effects of cyclones and storm surges. Tropical cyclones occur at the rate of 1.3 a year in the coastal districts of Bangladesh, making the area one of the world's most cyclone-prone. The most recent cyclone, Cyclone Sidr, hit Bangladesh on November 15, 2007 with an enormous intensity. Winds of 220-240 km/hr and the cyclone’s width of 600 kilometers caused over 3,000 deaths and projected costs of $2.3 billion dollars due to widespread devastation to houses, infrastructure, and livelihoods. But as Bangladesh begins a massive cleanup operation, many are thankful that it wasn't much worse. As devastating as it was, Sidr has taken far fewer lives than 1991's Cyclone Gorky, which killed at least 138,000 people, and 1970's Bhola, which left as many as 500,000 people dead and is considered the deadliest cyclone, and one of the worst natural disasters, in human history.

Table 3.1: Cyclones in Bangladesh

Date	Area	Maximum wind speed (km/h)	Casualties
5-6 Nov, 1971	Chittagong	-	-
23-30 Nov, 1971	Sundarbans	97-113	-
6-9 Dec, 1974	Sundarbans	-	-
13-15 Aug, 1974	Khulna	80.5	600
24-28 Nov, 1974	Cox's Bazar surge up 2.8 -5.2m	161	200 people, 1000 cattle 2300 house destroyed
9-12 May, 1975	Bhola, Cox's Bazar, Khulna	96.5-112.6	5 people
9-12 May, 1977	Khulna, Noakhali, Barisal Chittagong	112.63	-
14-15 Oct, 1983	Chittagong, Noakhali	122	-
5-9 Nov, 1983	Chittagong and coastal Area	136	300 fishermen, 50 boats
24-25 May,1985	Chittagong and coastal Area	154	11069
8-9 Nov, 1986	Chittagong, Noakhali,Barisal	110	14

24-30 Nov, 1988	Coastal islands of Barisal and Khulna	162	5,708 people, and numerous wild animals at The Sundarbans (deer 15,000, Royal Bengal Tiger 9), cattle 65,000. Total damage to crops reached Taka 9.41 billion.
29–30 April, 1991	1991 Bangladesh cyclone	225	150,000 people, 70,000 cattle loss of property was estimated at about Tk 60 billion.
31 May-2 June, 1991	Patuakhali, Barisal, Noakhali and Chittagong	110	-
29 April-3 May, 1994	Coastal islands near Cox's Bazar	210	400 people, 8,000 cattle
21–25 Nov, 1995	Coastal islands near Cox's Bazar	210	650 people, 17,000 cattle
16–19 May 1997	Chittagong, Cox's Bazar, Noakhali and Bhola	225	126 people
25–27 Sep, 1997	Chittagong, Cox's Bazar, Noakhali and Bhola	-	-
15 May, 2007	Cyclone Akash, Chittagong	85	50 fishermen missing, 3 confirmed killed. In all, 14 people were killed by Akash and damages amounted to US$982 million.
15 Nov, 2007	Cyclone Sidr, southern part of Bangladesh	215	2,000 deaths and severe damage
27 Oct, 2008	Cyclone Rashmi	75	15 people were killed and thousands of homes were damaged. At least 50 fishermen were reported missing when about 15 fishing trawlers capsized offshore
17 April, 2009	Cyclone Bijli	75	7 people were killed and 84 were injured. 200,000 people evacuated from Chittagong.
25 May, 2009	Cyclone Aila	110	179 fatalities from flooding, 58,950 animals were killed by the storm with up to 50,000 deer missing, 500,000 people homeless

3.2 Floods

Flooding normally occurs during the monsoon season from June to September. The convectional rainfall of the monsoon is added to by relief rainfall caused by the Himalayas. Melted water from the Himalayas is also a significant input and flood every year. Bangladesh has experienced seventeen highly damaging floods in the 20th century. Since independence in 1971, Bangladesh has seen floods of a vast magnitude and required external emergency assistance in 1974, 1984, 1987, 1988, 1998 and 2004. The largest recorded flooding in Bangladesh history occurred in 1998, when nearly 70% of the country was under water for several months. Each year in Bangladesh about 26,000 km^2, (around 18%) of the country is flooded, killing over 5,000 people and destroying 7 million homes.

Table 3.2: Floods in Bangladesh

Year	Impact
1987 (July-August)	Lasted for 63 days and affected area is estimated as 57,300 square kilometers (about 40% of the total area of the country), killed at least 880 people and estimated cost was $1,000,000,000
1988 (August-September)	Inundated about 82,000 km^2 of land, (about 60% of the area) killed at least 254 people and estimated cost $450,000,000
1998 (July-September)	Over 1000 deaths and 30 million people being made homeless and covered over 75% of the total area of the country
1999 (July-September)	The total deaths were approximately 918. 30 million people were made homeless. 100,000 square kilometers of land (mainly agricultural) was destroyed, this decreased Bangladesh's production rate by 20%
2004 July	Around 30 million Bangladeshis affected by flood, and more than 40% of the capital city, Dhaka were the underwater.
2007 July	Around 500 people died and a total of 10,211,780 people were badly affected by it. 56,967 houses were damaged

3.3 Tornadoes

Though Bangladesh had some deadliest tornadoes, people do not consider them as Oklahoma. Nevertheless, they get some bad ones. Also high population density and poor construction makes them vulnerable to high fatalities. The world’s deadliest tornado was in Bangladesh during April 26, 1989. The one and half mile wide, (F5) took place in Daultipur-saturia in Barisal district with approximately 1,300 fatalities.

Table 3.3: Tornadoes in Bangladesh

Area	Date	Fatalities
Magura-Narail Districts, Khulna	April 11, 1964	>500
-	April 14, 1969	923
Dhaka	April 17, 1973	681
Madaripur-Shibchar, Dhaka	April 1, 1971	500
Daultipur-Sulturia, Barisal	April 26, 1989	>1300
Jamalpur-Tangail Districts, Dhaka	May 13, 1996	>700
Savar and Gazipur Districts, Dhaka	September 19, 2000	2 fatalities, 5 injures

3.4 Earthquakes

Bangladesh is surrounded by the regions of high seismicity, which include the Himalayan Arc and SHILLONG PLATEAU in the north, the Burmese Arc, Arakan Yoma anticlinorium in the east, and complex Naga-Disang-Jaflong thrust zones in the northeast. It is also the site of the Dauki Fault system along with numerous subsurface active faults and a flexure zone called Hinge Zone. These weak regions are believed to provide the necessary zones for movements. The huge density and high-rise buildings in Dhaka city also aggravate the possibility of earthquake. On September 18, 2011, we faced the most recent earthquake (Sikkim earthquake with magnitude 6.9). The earthquake was felt most strongly in northern Bangladesh. The quake was also felt in Dhaka, Sylhet, Faridpur, Bogra and as far as Chittagong.

Table3.4: Earthquakes in Bangladesh

Year	Impact
1997	Occurred on 22 November in Chittagong with a magnitude of 6.0. It caused minor damagearound Chittagong town
1999	Occurred on 22 July at Maheshkhali Island with the epicentre in the same place, a magnitude of 5.2. Severely felt around Maheshkhali island and the adjoining sea. Housescracked and in some cases collapsed.
2003	Occurred on 27 July at Kolabunia union of Barkal upazila, Rangamati district withmagnitude 5.1.
2011	Sikkim Earthquake, with a magnitude of 6.9 centered within the Kanchenjunga Conservation Area, near the border of Nepal and the Indian state of Sikkim. At least 111 people were killed in the earthquake but no casuality in Bangladesha. Most of the deaths occurred in Sikkim, with reports of fatalities in and near Singtam in the East Sikkim district.Several buildings collapsed in Gangtok.

3.5 Landslides

Illegal hill cutting due to rampant building has left some 70,000 people at risk of landslides in 18 sub-districts of Khagrachhari, Rangamati and Bandarban hill districts, as well as the city of Chittagong. Mudslides are common during the rainy season in Bangladesh. The country has more than 200 rivers and is prone to severe flooding. More than 300 people have been killed in landslides in Chittagong in recent years.
In June 2007, a landslide at Mati Jharna colony of Lalkhan Bazar, right in the heart of Chittagong, killed 127 people and injured 100 more during last year's annual monsoon, when a hill collapsed on to an adjacent slum. Another landslide occurred after 3 days continuous rainfall in Lalkhan Bazar in 3rd July, 2011, with the mud burying at least seven houses and killed 17 people.

4. DISASTER MANAGEMENT

Several natural disasters that have occurred in the past few years and our vulnerability to natural threats have placed the disaster management in the front page. Disaster management has a long history, starting with mitigation plan of Noah (A) by building ark to deal with the greatest flood. Now it not only involves relief plan but also emphasize on precautions. A very fast step is risk assessment, which involves the probability of occurrence a disaster. Though its really hard because there is always uncertainty about event occurrence and after that recover planning and estimate the loss. This indicates disaster management has become a vital issue and it has different phase of working. Since there is no specific standard to differentiate phases (as every organization has their own phases according to their objectives), here four phases are taking into consideration.

4.1 Mitigation

Mitigation is measures taken to reduce both the effects of hazards and the vulnerable condition to it. In order to reduce future impact of disaster, mitigation activities can be focused on the hazard itself or the elements exposed to the threats. Not all disasters, particularly natural disasters, can be prevented, but the risk of loss of life and injury can be mitigated with good risk assessment, evacuation plans, environmental planning and design standards. Mitigation measures can be structural or non-structural. Structural measures use technological solutions like flood levees and building retrofitting for earthquakes. Non-structural measures include legislation, land-use planning (e.g. the designation of non-essential land like parks to be used as flood zones), and insurance. This phase can also be known as disaster risk assessment where pre-disaster and post-disaster plans are done. The mitigation phase differs from the other phases in that it focuses on long-term measures for reducing or eliminating risk. The implementation of mitigation strategies is a part of the recovery process if applied after a disaster occurs.

4.2 Preparedness

Preparedness involves putting in place the various steps called for by the recovery disaster plan. Preparedness is a continuous cycle includes planning, managing, organizing, training, equipping, exercising, creating, evaluating, monitoring and improving activities to ensure effective coordination and the enhancement of capabilities of concerned organizations to prevent, protect against, respond to, recover from, create resources and mitigate the effects of natural disasters, acts of terrorism, and other man-made disasters. Some common measures taken as preparedness are,

- Developing community based work.
- Proper communication plan and alarm system.
- Proper maintenance and training of emergency services.
- Development and exercise of emergency warning methods.
- Development of organizations of trained volunteers among civilian populations.

Another aspect of preparedness is casualty prediction, the study of how many deaths or injuries to expect for a given kind of event. This gives planners an idea of what resources need to be in place to respond to a particular kind of event.

4.3 Response

Response includes immediate steps taken to deal with a disaster. First the disaster should be detected which is going to be occurred and after the preliminary needs (search and rescue) as well as basic humanitarian needs of the affected population should be taken under consideration. Initial response always comes from the near by communities and organizations. There is a need for both discipline (structure, doctrine, process) and agility (creativity, improvisation, adaptability) in responding to a disaster. There is also the need to onboard and build an effective leadership team quickly to coordinate and manage efforts as they grow beyond first responders. The leader and team must formulate and implement a disciplined, iterative set of response plans, allowing initial coordinated responses that are vaguely right, adapting to new information and changes in circumstances as they arise.

4.4 Recovery

Once emergency needs have been met and the initial crisis is over, the people affected and the communities that support them are still vulnerable. Recovery activities include rebuilding infrastructure, health care and rehabilitation. These should blend with development activities, such as building human resources for health and developing policies and practices to avoid similar situations in future. An important aspect of effective recovery efforts is taking advantage of a window of opportunity for the implementation of mitigation measures that might otherwise be unpopular. Citizens of the affected area are more likely to accept more mitigation changes when a recent disaster is in fresh memory.

5. DISASTER MANAGEMENT IN BANGLADESH

In the past, disaster management just deals with relief work and reconstruction. Now the scenario has changed by introducing preparedness, mitigations plans. As one of the worst disaster prone country, the Government of Bangladesh has taken the following significant steps during the last few years for building up institutional arrangements from the national to the village level:-

Establishment of a disaster management organization named Disaster Management Bureau (DMB) in 1993.

Renaming of the Ministry of Relief and Rehabilitation as the Ministry of' Disaster Management and Relief (MDMR).

Establishment of Emergency Operations Center (EOC) at MDMR.

Establishment of Disaster Management Council and Committees from national down to field levels for overall disaster management.

Organizational Structure at the National Level. Following are the institute at national level to meet crisis situations:-

National Disaster Management Council (NDMC). It is headed by the Humble Prime Minister to formulate and review the disaster management policies and issue directives to all concerns.

Inter-Ministerial Disaster Management Co-ordination Committee (IMDMCC).It is headed by the Humble Minister in charge of the Ministry of Disaster Management & Relief (MDMR) to implement disaster management policies and decisions of NDMC / Government.

National Disaster Management Advisory Committee (NDMAC). It is to be headed by an experienced person having been nominated by the Humble Prime Minister.

Cyclone Preparedness Program Implementation Board (CPPIB).It is headed by the Secretary, Ministry of Disaster Management & Relief to review the preparedness activities in the face of initial stage of an impending cyclone.

Disaster Management Training and Public Awareness Building Task Force (DMTATF).It is headed by the Director General of Disaster Management Bureau to co-ordinate the disaster related training and public awareness activities of the Government, NGOs and other organizations.

Focal Point Operation Coordination Group of Disaster Management (FPOCG).It is headed by the Director General of DMB to review and co-ordinate the activities of various departments/agencies related to disaster management and also to review the Contingency Plan prepared by concerned departments.

NGO Coordination Committee on Disaster Management (NGOCC).It is headed by the Director General of DMB to review and co-ordinate the activities of concerned NGOs in the country.

Committee for Speedy Dissemination of Disaster Related Warning/ Signals (CSDDWS).It is headed by the Director General of DMB to examine, ensure and find out the ways and means for the speedy dissemination of warning/ signals among the people.

Organizational Structure at the Field Level. Following are the institutes/committees in the field meeting crisis situations:-

District Disaster Management Committee (DDMC). It is headed by the Deputy Commissioner (DC) to co-ordinate and review the disaster management activities at the district level.

Upazila Disaster Management Committee (UZDMC). It is headed by the Upazila Nirbahi Officer (UNO) to co-ordinate and review the disaster management activities at the Upazila level.

Union Disaster Management Committee (UDMQ). It is headed by the Chairman of the Union Parishad to co-ordinate and review and implement the disaster management activities of the concerned union.

Institutions Playing Pivotal Role in Disaster Management in Bangladesh. Following institutes are playing pivotal role in disaster management:-

- The Space Research and Remote Sensing organization.
- The Bangladesh Meteorological Department.
- The Flood Forecasting and Warning Center.
- The Bangladesh Television.
- The Radio Bangladesh.
- The Cyclone Preparedness Program (CPP).
- Other voluntary organizations.

Problems Being Faced During the Management of Disaster

There are some constraints if not addressed in time may greatly affect the disaster management operation and thereby reduce the capability of the organizations/ agencies including the Armed Forces.

Absence of Common Forum. In our country, there is less opportunity to exchange views among the agencies involved that hinders mutual understanding and cooperation.

Lack of Co-ordination. Sometimes the works of different agencies are not well co-ordinated, and may turn into an utter failure.

Confusion About Jurisdiction of Work. In all situations, the role of each agency should be clearly defined. Without it destroys the congenial atmosphere.

Lack of Mutual Respect. In most cases officers of civil and military administration suffer from ego-centric complexities. These create rigidity and hinder smooth functioning.

WAYS TO OVERCOME THE PROBLEM

To overcome the mentioned problems, few suggestions are discussed below:

Mutual Respect. All civil agencies and military administration must have respect for each other to ensure effective co-ordination and success in a given task.

Proper Communication and Co-ordination. Proper and timely communications between civil and military administrations help achieve the goal.

Specialized Training. Training helps to understand the mitigation measures and annual preparedness.

Public Awareness. All members of the District Cells should be trained in order to undertake effective mass awareness program to the public of the disaster prone areas.

Equipment Requirements. With some specialized equipment to fight the disaster, Armed Forces would play a pivotal role in complementing government effort to restore normalcy in disaster-affected areas.

6. CONCLUSIONS

Disasters are major threats to people's livelihood in Bangladesh. In a few minutes or hours a disaster can eradicate years of local development efforts. Catastrophic disasters like Sidr, Aila, and severe floods have shown how disaster risk and economy of a country are related. Still we are not recovered from the effect of Aila and the footprint of Sidr is still visible in Sundarbans. Definitely there were lots of endeavor to address disaster management but we need more effort to tackle it. Along with funding, we should encourage people to come with new innovative ideas and research work related to disaster management, early alarm system, mitigation process, flood proof houses etc.

Disasters are no more local issues its global now. So we have to improve our foreign policy and legislature to improve our condition. We have to encourage entrepreneurship to deal with these calamities. Just increase the number of organization is not enough, we also have to work out the quality. Communication between every organization is very important because mutual work will ensure progress in larger way rather than individual.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PROTECTION OF HARMFUL EFFECT OF TSUNAMI BY RETAINING SYSTEM AT COASTAL AREA

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ABSTRACT

Most of the developed countries are situated at coastal area. Natural disasters like tsunami have occurred at recent time at different places of different countries. Most recent tsunami occurred at Japan. The financial loss is approximately $70 billion of Japan. Statistics has said that the total cost to construct the retaining wall will be1 percent of that total financial loss. Bangladesh is also situated at coastal area. The Longest seashore situated at Bangladesh from Chittagong to Cox's Bazaar is approximately 155 km long. The cost required to construct Retaining wall at this beach is $7-$8 billion that is almost half of construction cost of Padma Bridge. It is also possible to protect the devastating effect of Natural disasters like Tsunami by grid system high-rise building. This building may be highly acceptable of tourists. These may be used for accommodation of people, Hotel, Market etc. The recent natural disaster that took place in Bangladesh was on November 15, 2007. It was a cyclonic storm; named Sidr that brought havoc in southern and central Bangladesh. Millions become homeless after the storm had passed. It killed almost 4000 people and injured about 30,000. Every year occurs like this Natural disaster and Millions become homeless. This building may be use for this homeless people for their rehabilitation. To save some places by Retaining wall from the devastating effect Natural disaster may be use for school, hospital, airport, stadium, university etc to beach. Topographic condition of coastal area may be of different elevation and some places are very suitable for accommodation of people. Retaining wall is usually suitable at high-elevated area, hilly region & places where accommodation is difficult. The cost required to construct the Conventional Retaining wall to Chittagong to Cox's Bazaar beach is $ 5625~$ 6250 per ft length against 50 ft height rising water. If we silly change the structure of 'Conventional Retaining wall' like 'Retaining wall against tsunami' the cost will be $1750~$ 1875 per ft length against 50 ft height rising water that reduce the 68.88 % cost. Therefore, we stable the 'Retaining wall against tsunami' instead of 'Conventional Retaining wall' against huge lateral water force. For 20 storied with 45 ft width Building is $ 18750 ~ $ 20000 per ft length. Total cost for 155 km of 'retaining wall against tsunami' is about $ 8, 90,000 & total cost for 155 km of building is about $9500000. Overall, only Retaining wall is not profitable construction. On the other hand, only grid system building requires huge cost that is expensive for a country. So Retaining wall & building combinedly can give safe & wonder for a country. Bangladesh may be most attractive country to the tourists to build high-rise building to beach and may be enrich to save the life of people & Natural resources from the devastating effect of Natural disaster.

Keyword: *Natural disaster like Tsunami, Retaining wall, Grid system building, financial loss.*

1. INTRODUCTION

Earthquake under sea releases huge amount of energy. This energy forces to the water and water moves to upward. This called tsunami. Ultimately, tsunami or tornado means huge amount energy. Tsunami or tornado fall on the locality and damage the shelter of human and animals. Beside this, damage the different type structure like low rise to high-rise building, bridge, culvert etc. Destroy the huge amount of crops that affect the economy of a country. Therefore, tsunami cause huge amount of financial loss.

The research of Retaining wall reduces or fully stops the financial loss. Most of the tsunami affected country take attempt to protect harmful effect the tsunami like Japan. But all the dam has been damaged by the tsunami. Therefore, enough rigid and low cost retaining wall stabilization is very important.

In this case, two types of force come to the wall. Huge lateral force and huge bouncy force from bottom to the basement that want to float the retaining wall. There is no natural support to protect the wall. So pile should be use to protect the retaining wall.

Lateral force of H height wall is P=$\gamma H^2/2$ for unit width.

Bouncy force of L length basement is P= γH^2 for unit width

Since huge amount of lateral force come to the wall, so to protect that huge amount of lateral force, the shear reinforcement must be use. If additional shear reinforcement does not provide to protect this force, it will be uneconomical & unacceptable.

2. METHODOLOGY

2.1 Force coming to Retaining wall

Two types of force come to the retaining wall.

1. Lateral force of water &
2. Bouncy force

There is no considerable soil support because of bouncy force of water. Bouncy force may vary from full seepage to no seepage because of sudden to timely occurring of tsunami.

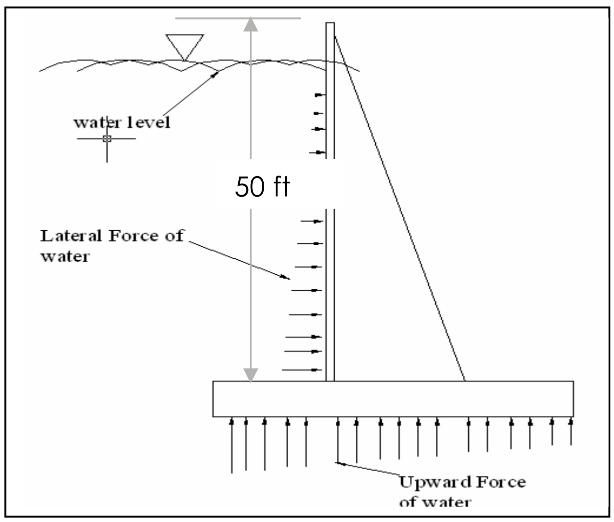

Fig: 2.1 Force coming to the retaining wall

2.2 Conventional Retaining Wall

Conventional Retaining Wall is consisting of two parts.

1. Basement &
2. Cantilever wall

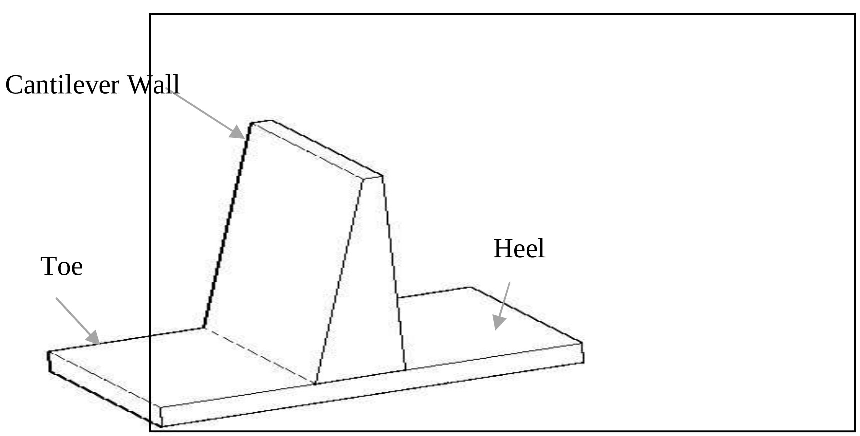

Fig: 2.2 Conventional Retaining Wall

Huge lateral force coming from water wave & static water cannot protect the only designed by flexural reinforcement conventional retaining wall .There is no shear reinforcement. That is why total lateral force coming to the wall from water force protects by concrete strength, which produce uneconomical project.

2.3 Retaining Wall against Tsunami

Parts of Retaining Wall

1. Front wall
2. Shear wall
3. Basement Beam
4. Bottom Slab
5. Pile

- **Front wall**

Front wall transfers only water pressure to shear wall.

- **Shear wall**

Shear wall protect the lateral load coming from the front wall & transfer to the basement beam. This shear wall is designed both shear & flexural reinforcement.

- **Basement Beam**

Basement Beam is very stiff due to both shear & flexural reinforcement. Only beam transfers total load to the pile without basement slab.

- **Bottom Slab**

It only transfers the bouncy force to the beam.

- **Pile**

Pile is most important portion for this retaining wall. Pile designs not only tension or compression force but also lateral force because huge amount of lateral force protect the pile.

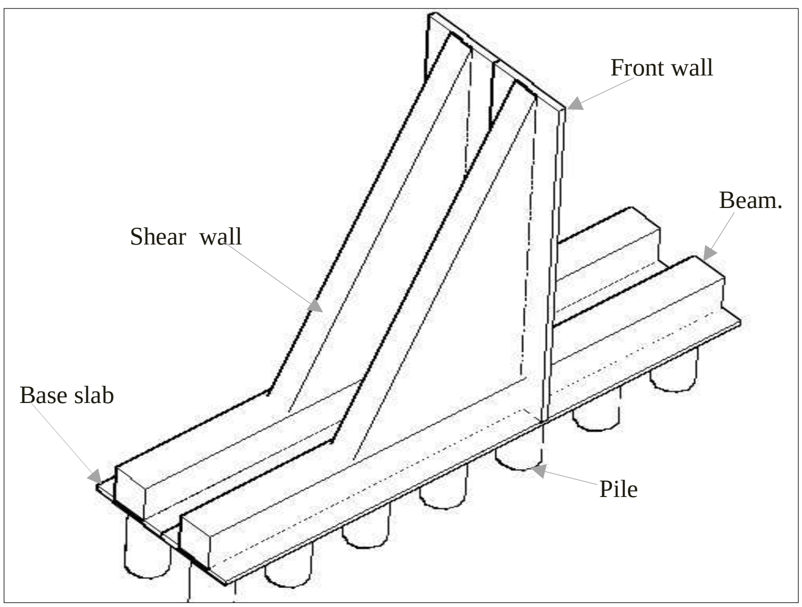

Fig: 2.3 A Portion of Retaining Wall against Tsunami

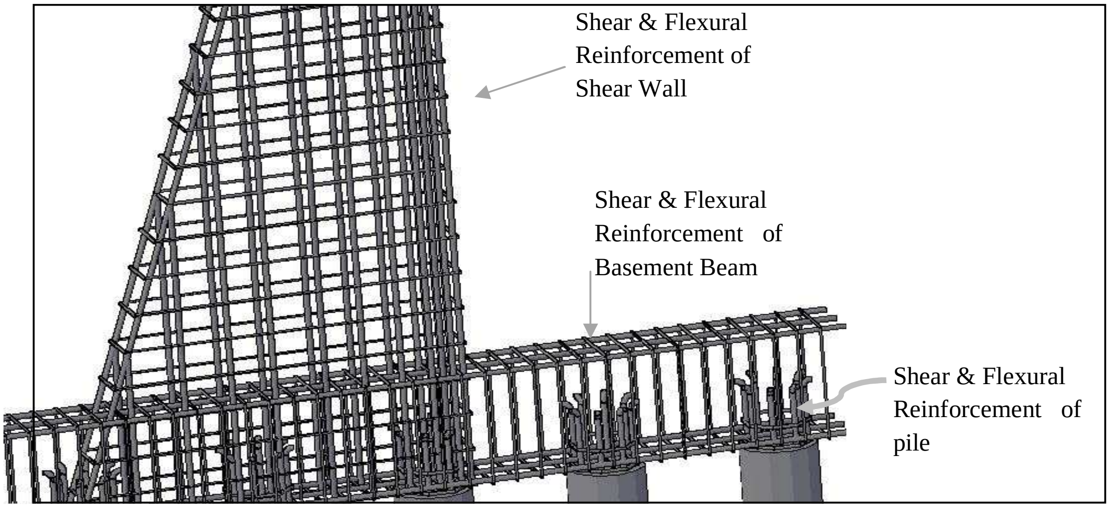

Fig: 2.4 Arrangement of Shear & Flexural Reinforcement of a portion of Retaining wall.

2.4 Complete protection of effect of tsunami by high-rise building & retaining wall

Topographic condition of coastal area may be different elevation and some places are very suitable for accommodation for people. This building is highly acceptable for tourist. Besides some places may be use for school, Hospital, Airport, Stadium etc. Retaining wall is suitable usually at highly elevated and hilly areas & at unsuitable area for accommodation.

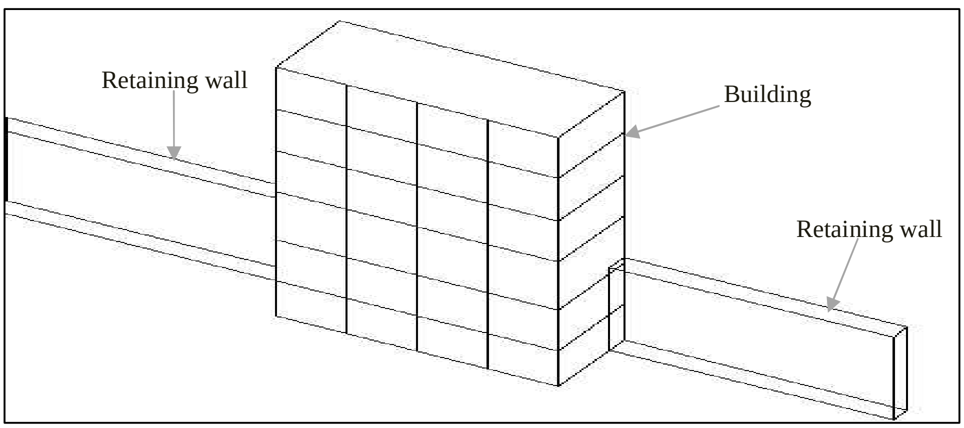

Fig: 2.5Complete protection of effect of tsunami by high-rise building & retaining wall

3. RESULTS

The cost required to stable the retaining system against 50 ft rising water.

1. Conventional Retaining wall is $ 5625~$ 6250 per ft length.
2. Retaining Wall against Tsunami is $1750~$ 1875 per ft length.
3. For 20 storied with 45 ft width Building is $ 18750 ~ $ 20000 per ft length.
4. 1% Building of Total length & other is Retaining Wall against Tsunami will be cost $101.05 crore.

To stable the retaining system with building and retaining wall combinedly to beach by 1 percent of total financial loss of one time occurring natural disaster like tsunami Bangladesh can save the Millions of people, natural resources.

4. DISCUSSIONS

The Longest beach of Bangladesh from Chittagong to Cox's Bazaar is 155 km. Cost of Retaining Wall against Tsunami $ 1750~$ 1875 per ft length. Cost of 20 storied with 45 ft width Building $ 18750 ~ $ 20000 per ft length. Total cost for 155 km of retaining wall is about $ 8, 90,000 & Total cost for 155 km of building is about $95, 00,000. So total retaining wall is unprofitable & total building is unacceptable for a country like Bangladesh. However, retaining wall against Tsunami & grid system high-rise building combinedly can make a country safe, wonder & economically strong.

5. CONCLUSIONS

When tsunami or tornado falls on the locality, it damages the shelter of human and animal. Besides these, damages the different type structure like low rise to high-rise building, bridge, culvert etc. Destroy the huge amount of crops that affect the economy of a country. Therefore, tsunami causes huge amount of financial loss. The research of Retaining wall reduces or fully stops the financial loss. Statistic has said that the total cost would be to construct the retaining wall is 1 percent of total financial loss. Overall, only retaining wall is not usable construction. On the other hand, only grid system building is required huge cost that is unacceptable for a country. So Retaining wall & building combinedly can give a safe, wonder & developed country

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE OF TUBULAR SOLAR STILL IN DESALINATION: A CASE STUDY

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ABSTRACT

This study aims to provide a low cost solar stills using locally available materials to meet small scale fresh water demand in the coastal, arid and remote regions. In this study a tubular solar still (TSS) was designed, constructed and field experiment have been carried out at Khulan University of Engineering & Technology since 2008. It is comprised of tubular frame (1m long and 20cm in diameter, made of 2.75 mm GI wire) covered with a transparent normal polythene paper and a black rectangular trough (1.0 m long, 16 cm width, 5 cm deep, made of carton paper) for storing saline water.The average daily production rate for the TSS was found as 3.03 lit/m^2-day, with highest and lowest values were 3.81 and 1.79, respectively. Highest productions were observed in April-June and lowest in December-January. The initial cost for the preparation of the TSS was only 80 Tk./TSS and the production cost of the distilled water was Tk. 0.39 Tk./lit. It is concluded that due to low initial cost and also the construction, operation and maintenance of TSS is easier than that of BSS, one can easily use a TSS for drinking and other purposes in remote, coastal and arid areas or in an emergency to meet small scale fresh water demand.

Keywords: *Ambient air temperature, Basin type Solar Still, Desalination, Salinity, Solar energy, Tubular Solar Still (TSS).*

1. INTRODUCTION

Water is one of the most abundant resources on earth, covering three-fourths of the planet's surface. In the world, demand of portable fresh water is increasing day by day because of population explosion all over the world, greater industrial development, expansion of agricultural activities and climate change. Now it is recognized that freshwater is a scare resources and more country is converted into water-stressed country due to the scarcity of freshwater resources. There is an almost unfathomable amount of water on earth: about 1.4 billion km^3 (330 million cubic miles) (Barlow and Clark, 2002). About 97% of the earth's water is salt water in the oceans and a tiny 3% (about 36 million km^3) is fresh water contained in the poles in the form of ice, ground water, lakes and rivers, which supply most of human and animal needs (Ahmed and Rahman, 2000). Nearly, 70% from this tiny 3% of the world's fresh water is frozen in glaciers, permanent snow cover, ice and permafrost. Thirty per cent of all fresh water is in underground, most of it is in deep, hard-to-reach aquifers. Lakes and rivers together contain just a little more than 0.25% of all fresh water; lakes contain most of it (Kalogirou, 2005). The data relevant to water requirements shows that around 25% of the total world populations do not have an adequate fresh water supply, both for quality and quantity (Agha et al., 2005). Water shortages affect 88 developing countries that are home to half of the world's population. In these places, 80-90% of all diseases and 30% of all deaths result from poor water quality (Leitner, 1998). Furthermore, over the next 25 years, the number of people affected by severe water shortages is expected to increase fourfold (Engelman et al., 2000). Some of this increase is related to population growth, some is related to the demands of industrialization. Currently, water consumption doubles every 20 years, about twice the rate of population growth (Barlow and Clark, 2002). The ground water source is being polluted by organisms, organic and inorganic compounds due to the ultimate disposal of man-made harmful pollutant into the underground reservoir (Malik et.al, 1982). In the coastal region of Bangladesh, salinity in water poses a serious problem for the communities in the affected area. The increase in salinity in Khulna region started after the commencement in 1975 of the Farrakka Barrage operation in India, which significantly reduced the flow in the Ganges, located at upstream of the Gorai River, a major source of freshwater to the rivers surrounding Khulna. At present, the principle cause of salinity intrusion in Khulna region is the drop of hydraulic head during the dry period (November to May) into both surface and groundwater of the area (Hassan et al., 1998). Although this communities are virtually surrounded by water but practically they suffer due to lack of drinkable water. There are deep aquifers containing water of acceptable salinity for human consumption; but the deep aquifers containing sweet water are not found at all possible locations. Thus the removal of salinity is immense for the population of those regions to obtain sufficient amount of potable water by mixing saline water with distilled water. One of the alternative sources of potable water for this area appears to be the development of the appropriate desalination technology.

Most desalination techniques consume a large amount of energy. Moreover, many remote towns and communities rely on costly and often limited supplies of diesel fuel for their energy needs. These and other forms of fossil fuels are sometimes heavily subsidized by government to meet community service obligations (Water Corporation, 2000). Therefore finding methods of using renewable energy to power the desalination process is desirable. Solar distillation is the simplest desalination technique, compared with other types, e.g., multiple-effect distillation, multi-stage flash, reverse osmosis, electro-dialysis and biological treatment due to no need of fossil fuel or electricity. The main disadvantage of solar distillation is its low productivity of distillate but although it may also be one of the viable options for providing drinking water for a single house or a small community in arid or remote regions. A basin-type solar still is the most popular method of solar distillation, but main drawbacks of the basin type are not easy of construction and the difficulty in rapid and easy removal of basin accumulated salt. Therefore, we designed a new type of low cost of solar distillation unit, Tubular Solar Still (TSS), to overcome such difficulties in the maintenance and management. It is comprised of tubular frame covered with a transparent polythene paper and a trough for storing saline water. In this study, a low cost TSS was designed, constructed and field experiments have been carried out at Khulna University of Engineering and Technology (KUET) since January 2008.

2. METHODOLOGY

A low cost Tubular Solar Still (TSS) is designed and constructed using locally available materials. It is consisted of Tubular frame covered with a transparent normal polythene paper and a black rectangular trough for storing saline water. Field experiments are conducted using the constructed TSS. Daily distilled water production and hourly production of some typical days are recorded. Collected data was analyzed and correlations are proposed for daily output. Finally, the water production cost is estimated and cnclusions are drawn. The steps of the methodology are shown in the flow chart given in Figure 1.

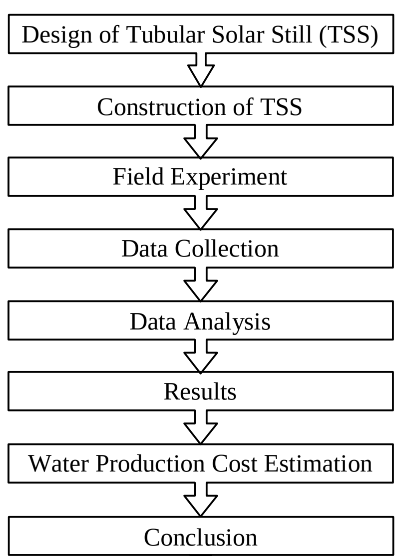

Figure 1: Flow chart showing steps of the work

3. PRODUCTION PRINCIPLE OF TSS

Production principle of TSS is illustrated in Figure 2. The solar radiation, after transmission through a transparent polythene cover, is mainly absorbed by saline water in the trough. The tubular cover and trough absorb the remaining small amount of the solar energy. Thus the water in the trough is heated and then begins to evaporate. Many types of heat transfer occur inside the tubular cover and outside, e.g., evaporative heat transfer from the saline water to the tubular cover, condensative heat transfer from the saline water and the tubular cover, the trough and the water, the trough and the tubular cover and the cover and the atmosphere, radio-active heat transfer between the water surface and the tubular cover and the tubular cover and the atmosphere. The evaporative water is transferred to the tubular cover and then finally condensed on the tubular cover inner surface, releasing its latent heat of vaporization. The condensed water trickles down the bottom of the tubular cover inner surface due to gravity and is stored in a collection bottle through a pipe provided at the middle.

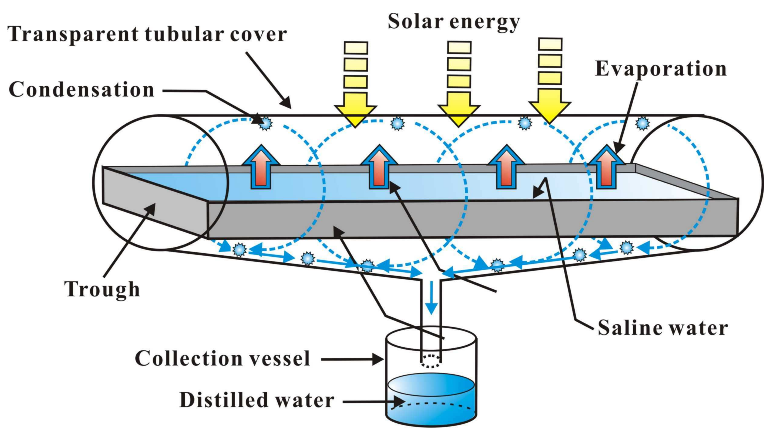

Figure 2: Production principle of TSS

4. DESIGN, CONSTRUCTION AND FIELD EXPERIMENT

4.1 Design and Construction of Tubular Solar Still

TSS is consisted of tubular frame covered with a transparent normal polythene paper and a black rectangular trough for storing saline water. The Tubular frame is 1.00 m long, 20 cm in diameter and is comprised of 2.75 mm thick helical G.I wire. The pitch of the spiral ring is about 5 cm. The trough is 1.0 m long, 16 cm width, 5 cm deep and is made of carton paper (painted black inside) covered with black polythene. The schematic diagram of the TSS is shown in Figure 3.

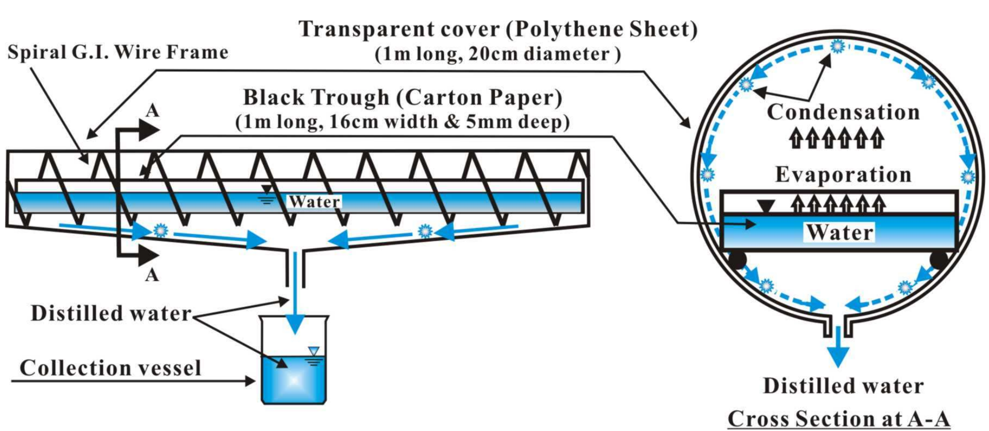

Figure 3: Schematic diagram of the TSS

4.2 FIELD EXPERIMENT

The field experiments have been carried out on the roof top of the Civil Engineering building of Khulna University of Engineering and Technology (KUET) since January 2008. Figure 4 shows the photograph of the field experiment. A wooden frame was used to support the TSS so that free circulation of air occurs beneath the still. The vessel for distilled water collection was put in a wooden box covered (inside) with styrofoam (heat insulator) in order to collect the amount of distilled water from the TSS accurately. One end of the TSS was kept fixed and other end could open to clean the trough or to remove the accumulated brine and to feed the saline water in the trough. The daily output from the still is collected approximately two hours after sunset. The hourly output was also measured for some typical days. The hourly outputs were also measured in some typical days to observe the hourly variation of the productivity of the still. Solar radiation flux and ambient air temperature were also measured at one minute interval using a data logger. A pyranometer and thermocouple were used to measure the solar radiation flux and temperature, respectively.

In the year 2010, the TSS was 1.20m long, 22cm in diameter and the trough was 1.18m long, 16cm width and and 5 cm deep and made of carton paper covered with black polythene sheet. Whereas, in the year 2008 and 2009, it was composed of a tubular copper frame covered with a transparent polythene paper and a black semicircular trough made of black polyvinyl chloride for storing saline water. In the year 2008, it was 80cm long, 15cm in diameter and the trough was 75cm long, 11cm in outside diameter and 1mm thickness. Wheres, it was 95 cm long, 15 cm in diameter and made of 5 mm diameter hollow copper pipe and the trough was 90cm long, 15cm in outside diameter and 1mm thickness. Also, the TSS was kept inclined downward at one end and a transparent bottle of 5cm long at that end was set to collect the distilled water output.

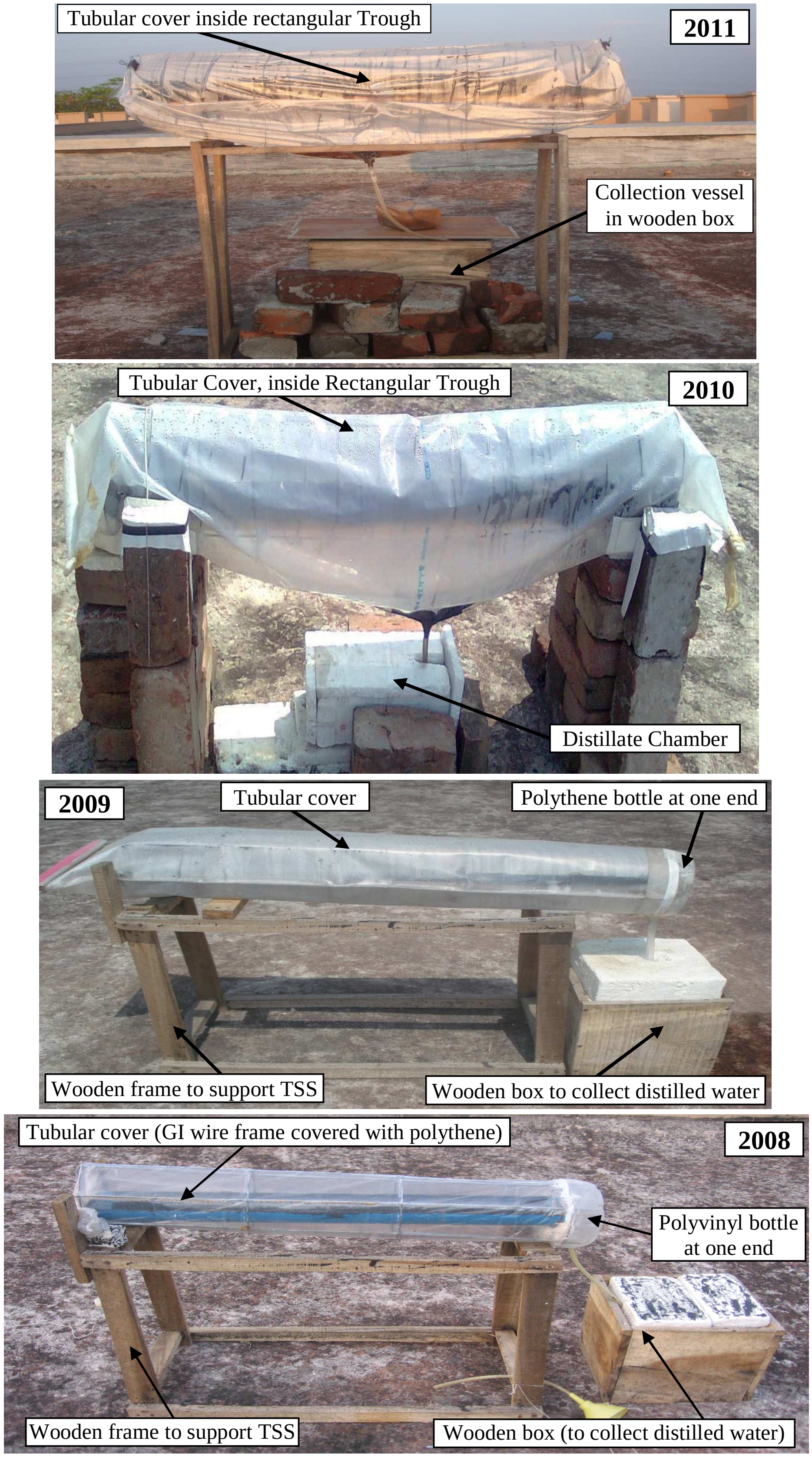

Figure 4: Photograph of field experiment on TSS

5. ANALYSIS AND RESULTS

Hourly and dailty distilled water output from the TSS are used to calculate the daily and hourly production rate per unit surface area of the saline water in the trough. Figure 5 shows the observed diurnal variations of hourly production per unit saline water surface area, solar radiation flux and ambient air temperature for the TSS at KUET, Khulna for March 18 of 2010. It is observed from the figure that the solar radiation flux rose rapidly after sunrise (approximately 6:30) and peaked approximately 12:00 after declining gradually. The air temperature also rose gradually in the morning (approximately 7:00) till 13:00, and declined gradually in the afternoon. Whereas the production was recorded from 9:00 in the morning (clearly indicating that there is a distinct time lag between evaporation and production or condensation), increased gradually up to 13:00, and then declined in the afternoon. It was also seen that the slope of the hourly production rate in the morning is steeper than that of the afternoon. The total distillate output for the day is found as 3.76 lit/m^2 (602 ml).

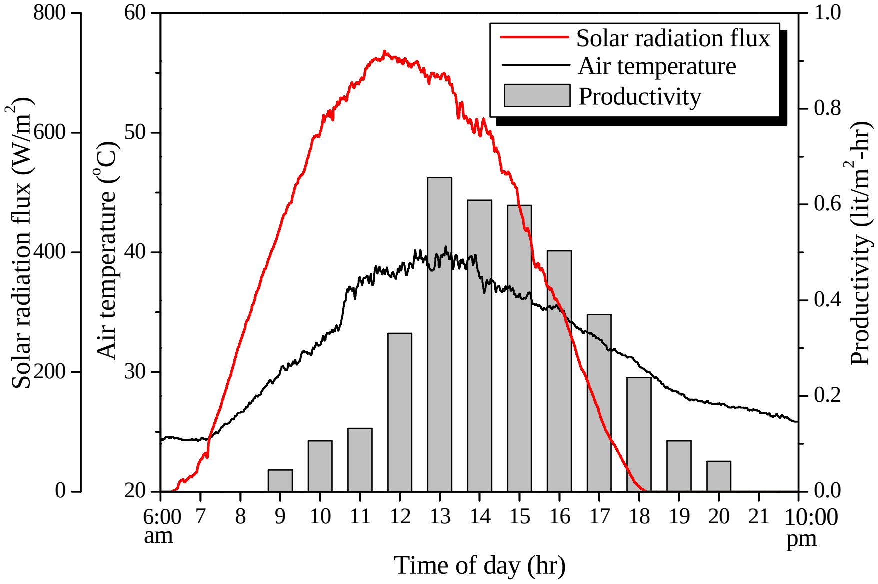

Figure 5: Observed diurnal variations of ambient air temperature, radiation flux and production fluxes for the TSS at KUET, Khulna for March 18 of 2010.

Figure 6 shows the variations of the observed daily distillate production rates from January to April of 2008, 2009, 2010 and 2011. The average production rates are estimated as 1.70, 2.60, 3.20 and 2.50 lit/m^2-day for the month January, February, March and April, respectively.

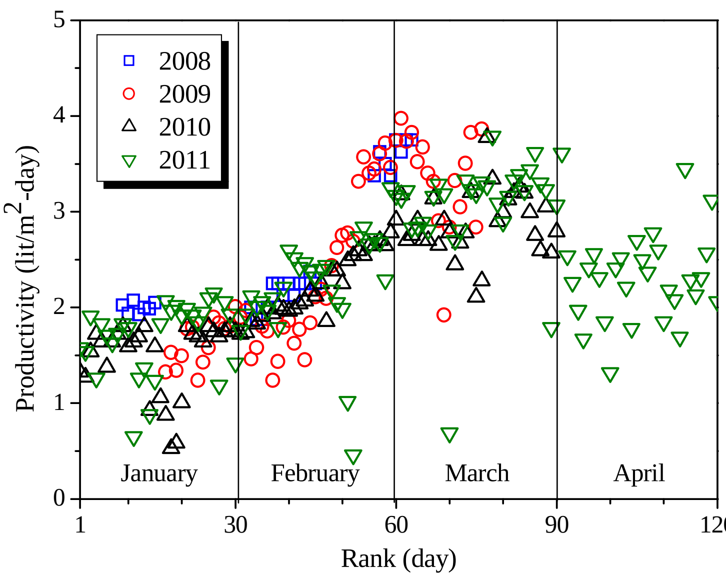

Figure 6: Variations of the observed daily distillate production rates for the TSS at KUET, Khulna from January to April of 2008, 2009, 2010 and 2011

One year basis field work on the TSS has been carried out at KUET, Khulna from October 2009 to September 2010 and average daily productions for each months are calculated and tablated in Table 1. For calculating the average daily production, non-sunshine days and remarkable very low-sunshine days due to cloud, rain etc. are not considered in the calculation. Figure 7 shows the variations of the maximum daily production per unit saline water surface in the trough for the TSS for all 12 months in the year 2010. It is seen from the figure that the production is minimum in December-January (1.8–2.0 lit/m^2-day) and then increased radpidly, peaked between April–July (3.5–3.8 lit/m^2-day) and then declined gradually. The highest maximum daily production is observed in April as 3.81 and lowest in January as 1.79 lit/m^2-day. The average maximum daily production for a month is 3.03 lit/m^2-day. The correlation for the maximum daily production rate in any month in a year can be expressed by the regression equaion given below.

$$y = 0.967 - 1.168\,x - 0.145\,x^2 + 0.005\,x^3 \qquad (r^2 = 0.83) \tag{1}$$

where, y = maximum daily production rate in any month (lit/m^2-day)
x = rank for the month (1 for January, 2 for February and so on)

Table 1: Average daily production rate for all twelve months for the TSS in the year 2010

January	February	March	April	May	June	July	August	September	October	November	December
1.79	2.77	3.75	3.81	3.61	3.13	3.70	3.32	3.22	3.17	2.19	1.94

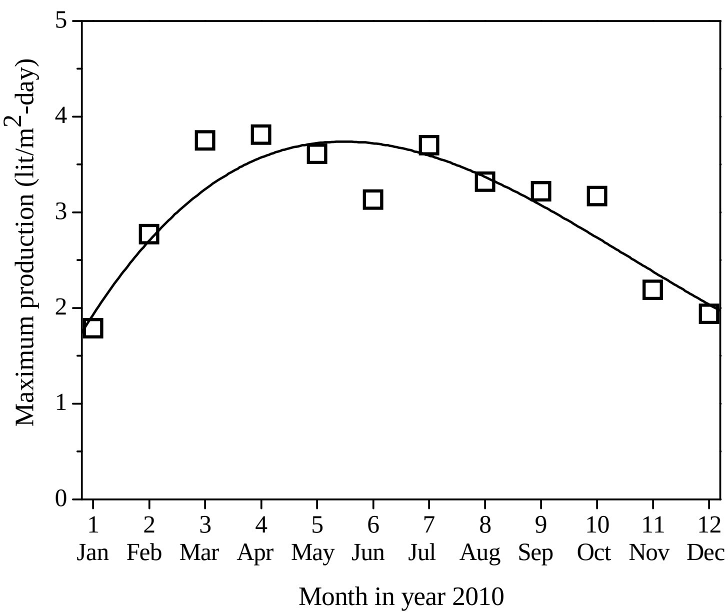

Figure 7: Maximum daily production for the TSS for all 12 months throughout the year 2010.

6. COST ESTIMATION

The initial cost of each tubular solar still was found as Tk. 80.00 and the total cost throughout the design life is estimated as Tk. 110.00. Table 2 shows the cost estimation of tubular solar still. Production cost of water from the TSS is estimated as 0.38 Tk./lit. The production cost is calculated as follows.

The average daily production for a month	= 3.03 lit/m^2-day	= 0.485 lit/day
Assume average daily production throughout a year	= 0.80 × 0.485	= 0.388 lit/day
Assume design life of a TSS	= 2 year	
The production of water in the design life	= 0.388×2×365	= 283.2 lit
Production cost of water	= 110 /283.2	= 0.39 Tk./lit

Table 2: Cost estimation of tubular solar still (Design life = 2 years)

SL. No.	Item Description	Amount in Tk.
1	G. I. wire to make the tubular frame	30
2	Trough preparation	10
3	Polythene to cover the tubular frame as condensation surface	15
4	Wooden frame to support the TSS	15
5	Miscellaneous	10
	Initial Cost =	**80**
6	Maintenance cost throughout design life (lump sum)	30
	Total Cost =	**110**

5. DISCUSSIONS AND CONCLUSIONS

The production rate of a solar stills is mainly depends on the intensity of solar radiation and the production rate will be higher in the summer season. The TSS is constructed using locally available material and the construction is very simple. Also, the operation and maintenance of a TSS is very simple and easy. It could open easily to clean the trough or to remove the accumulated brine and to feed the saline water in the trough. The average daily production rate for the TSS is found as 3.03 lit/m^2-day, with highest and lowest values are 3.81 and 1.79, respectively. Highest productions are observed in April-June and lowest are December-January. Initial cost for the construction of the TSS is found as 80 Tk./TSS and water production cost is estimated as 0.39 Tk./lit.

The result presented in this study gives clear information to understand the behavior of production rate and other related informations for the TSS. The application of this process can fulfill the demand of fresh water for drinking purpose for single household effectively. It is concluded that due to low initial cost and also the construction, operation and maintenance of TSS is easier than that of BSS, one can easily use a TSS for drinking and other purposes in remote, coastal and arid areas or in an emergency to meet small scale fresh water demand.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

IMPLEMENTATION OF RAINWATER HARVESTING - A CASE STUDY FOR GAZIPUR DISTRICT IN BANGLADESH

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ABSTRACT

Recently Bangladesh is facing severe crisis in supply of safe drinking water due to increasing trend of arsenic contamination in the underground water in different parts of the country. As a result of accelerated growth of shallow hand pump tube wells most people of the country has become heavily dependent on ground water. Rainwater harvesting (RWH) system possesses a great potential to minimize the consumption rate of underground water. This project investigates the feasibility of guiding potential use of rainwater in Bangladesh. For that purpose as a part of this study, an educational institute at Gazipur named Islamic University of Technology (IUT) was selected. This article assesses the possibility of RWH at IUT campus which can be a model of every institutional organization for minimizing the consumption of ground water. A survey was done to collect the monthly rainfall data for Gazipur district for 50 years. Measurements of the paved and unpaved area of the IUT campus were taken to determine the runoff water. Estimation on RWH is done on the basis of collected data. The conclusion of this article focuses on the reduction of ground water consumption, energy saving and arsenic free water.

Keywords: *Drinking water; rainwater harvesting; runoff; groundwater consumption; energy saving.*

1. INTRODUCTION

Once upon a time Bangladesh was an enriched country with many natural resources. But due to rapid urbanization, unchecked population growth, lack of awareness in using resources have made Bangladesh the poorest one. That’s why to save the country from further deterioration, it’s high time to put emphasis on some problems which are threading everyone. Among the problems the single but the biggest one is scarcity of water. The most convenient source of water is underground which is collected by installing water pump. The groundwater is free from pathogenic microorganisms and available in adequate quantity in shallow aquifers. So the huge demand of this source results lowering of water level. Moreover, a huge amount of energy is required to extract groundwater using pumps. Higher rate of energy consumption and cost are also considerable problems for Bangladesh. So, choice of a method of collecting water which is not only easy, convenient but also posseses the quality of environmentally sound and safe. According to these considerations Rainwater Harvesting (RWH) is the most favorable and affordable technology.

In the water cycle, while there are several methods by which the earth looses water, there is only through rainfall does the water come back to the earth. At this stage the water is relatively clean and can be collected for use with minimal capital investment. Compared to the conventional systems of water supply for domestic consumption, agriculture, industrial and other uses that emphasize abstraction from surface streams, deep wells and even the seas, rainwater is much cheaper, as it requires minimum treatment and needs little if any reticulation systems. In this context, rainwater harvesting can become a popular technique to improve the storage and recharge of water. Proper recharge of harvested water can augment the groundwater storage and increase the groundwater level (Chandra, 1979). This will also partially meet the demand of drinking water. It would also reduce the wastage of water due to surface runoff and has the potential to choke the storm drains (Abdulla and Al-Shareef, 2009).

Rainwater harvesting system, which has been widely used in many parts of the world, possesses a great potential in addressing today's real challenge of acute arsenic poisoning in different parts of the country. It is an option, which has been adopted in many areas of the world where conventional water supply systems are not available or have failed to meet the needs and expectations of the people (Alam, 2006). The rainwater is free from arsenic contamination and the physical, chemical and bacteriological characteristics of harvested rainwater represent a suitable and acceptable means of potable and non-potable water. People can construct storage reservoirs so that they can use rainwater during the entire rainy season and about 2-4 months of the dry period. Theoretically 20% of the total rainfall might satisfy almost the whole of the Dhaka City's demand, collected during the monsoon (Kabir and Faisal, 1999). A total of 228 nos. of rainfall stations and 54 nos. of automatic rainfall recorders were surrounded over Bangladesh (BWDB, 1999). Those rainfall stations were giving the rainfall records every day. But for this case study the data of Dhaka station were used as the selected area is Gazipur which is adjacent to Dhaka city.

2. OBJECTIVES OF THE STUDY

The overall objectives of this study are:
1. To assess the feasibility of rainwater harvesting technique in IUT campus.
2. To assess the present groundwater consumption rate at IUT campus and contribute the stored rainwater for future potable and non-potable use.
3. To save energy and cost for sustainability.
4. To solve the water logging problem at the backside agricultural land of IUT due to discharge of excess rainwater from IUT during the rainy season for environmental conservation and economic development of the surrounding area.

3. LITERATURE REVIEW

The world’s single biggest water problem is scarcity (Jury and Vaux, 2006). Rising population and urbanisation coupled with climate change may reduce urban water supply in developing countries (Murad et al., 2007; O’Hara and Georgakakos, 2008). According to Aditya and Sneha (2011) rapid industrial development, urbanization and increase in agricultural production have led to freshwater depletion in many parts of the country. Extensive use of advanced pumping techniques has made it possible to extract groundwater from greater depth, making the problem more acute. In this context, rainwater harvesting can become a popular technique to improve the storage and recharge of water.

Bangladesh is a tropical country situated between 20º34´and 26º38´ N latitude and 88º10´ to 92º41´ E longitude. The country experiences a heavy rainfall during monsoon, generally between 1,500 and 3,500 mm. Monsoon usually lasts from May to October and occasional rainfall in November. During this period, it gets ample rainwater, which could reduce the dependency on groundwater at least for 6 months (Islam et al., 2010). Moreover, an estimated 40 million of population of Bangladesh are drinking arsenic contaminated water (UNICEF, 2006) and about 70 million people of 59 districts out of 64 districts are at risk (Safiuddin and Karim, 2001). Rainwater harvesting is such technology which can prevent arsenic contaminated risks.

4. STUDY AREA

For this case study an educational institution at Gazipur named Islamic University of Technology (IUT) was selected, which has a residential campus. For collecting groundwater two deep tube wells are used at IUT campus, which are mechanically operated. It has almost 30 acre of land most of which is paved. So it has a huge potentiality of harvesting rainwater. Moreover rooftop rainwater harvesting can be made available here because of its large number of infrastructures like two academic buildings, two cafeteria, library, workshops, indoor stadium etc. Compared to roof catchment, ground catchment techniques provide more opportunity for collecting water from a larger surface area. IUT has vast brick soling surface area within its boundary. Because of all these reasons IUT campus was selected for this study. Detailed analysis of rainfall pattern of Gazipur area and complete measurements of land occupied by IUT campus were done. For this purpose the layout plan of this campus was used to measure by scale and some of the measurement have been done through plane table survey and field works. Metric chain and engineering tape were used for different area measurements.

A schematic diagram of IUT campus layout plan has been shown in figure 1. In the following figure there are some numbers like 7, 7A, 3, 2, 10, 20 which indicate the old academic building, new academic building, mosque, central plaza, residential hall and tennis court consecutively. So, it is obvious that this huge paved area along with the unpaved area like the tree park, sports field would help us to get required amount of rainwater for use during every monsoon season from April to October. Because these eight months have significant rainfall intensities in Bangladesh.

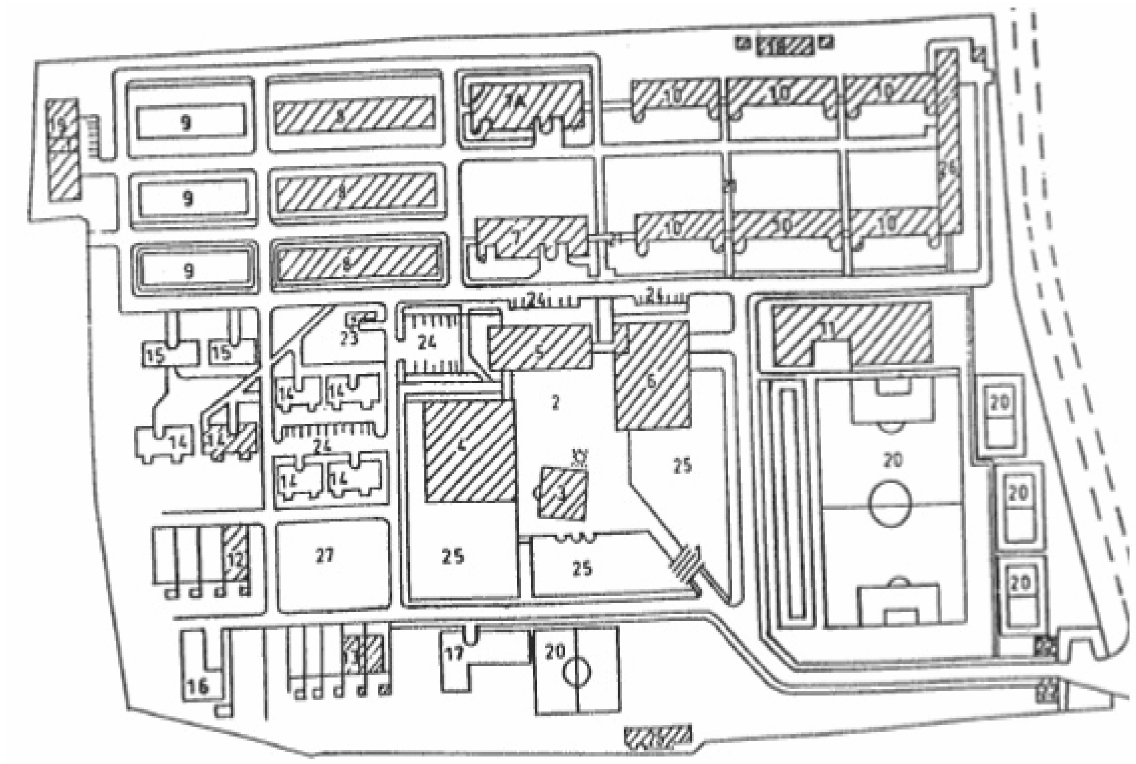


Figure 1: Layout Plan of IUT

5. STUDY METHODOLOGY

The methodology of the case study is described as follows:

1. The rainfall data for the last 50 years (from 1960 to 2010) were collected from Bangladesh Meteorological Department.
2. On the basis of this data rainfall analysis was done for the last 50 years consisting average monthly rainfall for 50 years, divergence of rainfall in a specific month for 50 years, variation of average yearly rainfall for 50 years to assess the possibility of getting required amount of rainwater.
3. The site selection was done according to the specified requirement which can fulfill the objectives.
4. Measurements of whole IUT area was done on the basis of layout plan and plane table surveying.
5. Engineering tape was used for calculating the area of some infrastructures like cafeteria, academic buildings, mosque, paved walkway, paved parking lot etc.
6. The existing level of groundwater was measured from the pump operator room.
7. The details of existing drainage system were considered to collect the expected amount of rainwater.
8. Cost and energy reduction analysis was done using the demand of water during the peak season (from April to October).

5.1 Assessment of Rainwater Availability

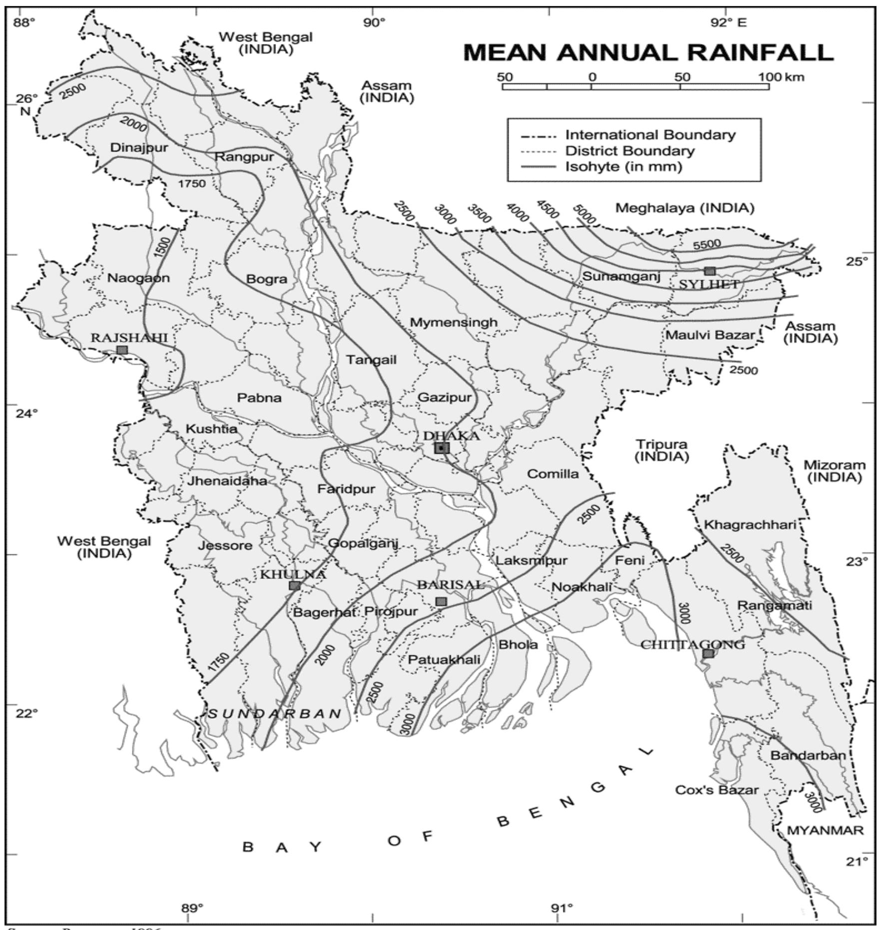
Rainwater is available in adequate quantity in Bangladesh. The spatial distribution of average yearly rainfall in Bangladesh has been shown in figure 2. The nearest rainfall record station to Board Bazar is Dhaka and thus the data of this station is used for calculating total amount of rainwater available in IUT campus.

5.2 Rainfall Catchment Area

The catchment area for rainwater collection is usually the roof, which is connected with a gutter system to lead rainwater to the storage tank. Moreover, the concrete roof of different infrastructure of IUT gives clean water. The paved road of brick soling also provides water through drainage system. The measurement of the paved area of IUT as Rainfall catchment area has shown in table1.

5.3 Rainwater Collection System

Rainwater may contain many impurities. But in comparison with other sources its quality is much better. The collection of this rainwater will be done through the existing drainage system of IUT. Here the central and the internal rainwater drainage line are connected with the peripheral drainage system. It has an outfall just behind the water reservoir of IUT. So a pipe can be easily installed from that outfall to the existing water reservoir to store and supply the rainwater on regular basis. There are many gutters and drain for collecting the rainwater from paved roads, walkway, sport fields and even the rooftop rainwater collection system is convenient for collecting rainwater.



Source: *Brammer, 1996*

Figure 2: Spatial distribution of average yearly rainfall in Bangladesh (Brammer, 1996)

The existing rainwater collection system is designed in such a way that it never creates any water logging or flooding in IUT campus. The flow through this drainage system finally discharges at the backside of IUT. There are many agricultural lands behind IUT campus which are over flooded during rainy season due to the excess discharge of water. But the implementation of this RWH can easily solve this problem and it will also save a huge amount of groundwater consumption along with electricity saving by stopping pump operation during this period of year.

6. DATA ANALYSIS

Rainfall data of 50 years (from 1960 to 2010) has been collected from Bangladesh Meteorological Department. The data has been showed in table 2. ('-' indicates missing data for the corresponding month). The nearest rainfall record station to Board Bazar is Dhaka and thus the data of this station is used for available rainwater calculation. From these data the average monthly rainfall of each year was calculated. It appears that the average monthly rainfall in Bangladesh during this period shows an enormous yearly diversity.

Bangladesh is a tropical country and receives heavy rainfall due to north-easterly winds during monsoon season. The variation of average monthly rainfall distribution of Gazipur area based on rainfall data from 1960 to 2010 is shown in figure 3. The figure shows that the significant rainfall only concentrated from April to October. The rainfall from November to March is not adequate to meet the demand during that dry period. Therefore the rainwater should be stored during the monsoon period rather than the dry season. Rainwater should be collected in a storage reservoir and then supply this water for various patable or non-potable uses.

Table 1: Measurement of IUT paved area

Segment	**Area (m^2)**
Mosque	472.5
Auditorium	1600
Parking Lot (Old academic building)	418.5
Administrative building	734.25
3 workshops	930 x 3 = 2790
D-type building for faculty members	325
E-type building for faculty members	462.5
F-type building for faculty members	246.75
Cafeteria & Library	1560.25
Student Centre	771.75
South Hall	1326
North Hall	1603
Old Academic Building	816.75
New Academic Building	1023
Ansar Camp	126
Gate	28
Security house	26.25
Substation of power	60
Water Storage & pump house	186
Medical Centre	250
Laundry	250
Parking Lot (New academic building)	945
Tennis Court	975.75
Basketball Court	800.5
3-storey building for M.Sc. students	1040
Central Plaza	3072.75
All paved roads	12140.57

Bangladesh is basically criss-crossed by many rivers. Moreover Bangladesh has a moderate climate. So, rainfall pattern is very much effective here for RWH. It is a kind of water source which will lead a sustainable development as well as environmental safety. From table 2 and figure 4, it is clear that during rainy season, July has the highest total monthly rainfall. Variation of rainfall in July for 50 years is also illustrated in figure 4. which shows that the rainfall pattern was almost constant from 1960 to 1981. But from 1982 to 2010 it shows a great alteration. For example, from 1960 to 1981, the rainfall was almost above 300 mm. But from 1982 to 2010 it differs a lot from 200 to 600 mm. That’s why 50 years of rainfall data were taken for predicting the rainfall pattern, because more safe and accurate amount of rainfall can be predicted by analyzing more rainfall data. Rainfall pattern in Bangladesh changes due to climate change which may includes global warming, temperature rise, green house effect, mean sea level rise etc. But whatever the reason is, it will directly affect the future consumption of water. So, it is better to implement rainwater harvesting to keep something usable for future generation.

Analysis of the variation of average yearly rainfall for 50 years in figure 5. From this figure, it is evident that from 1960 to 1988 the average yearly rainfall was between 150 to 200 mm. During this period, only 1967 shows a little difference from this pattern having 148.83 mm rainfall. But from 1989 to 2010 it shows a huge difference in yearly rainfall pattern like in 1989 it was 135.583 mm, in 1991 it was 237.5 mm, and again in 1992 it was 97.41667 mm. So these values clearly indicate that there was no consistency in rainfall pattern for the last 22 years. Because of this variation in rainfall pattern average rainfall data for whole 50 years were taken.

Table 2: Rainfall data of 50 years for Gazipur district

Yr	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	mm/month
1960	0	0	16	19	358	304	655	189	231	38	24	0	152.8333
1961	-	12	20	205	219	856	296	288	221	52	0	0	197.1818
1962	0	15	6	166	205	191	355	273	395	180	0	0	148.8333
1963	0	0	51	98	219	621	404	186	200	182	7	3	164.25
1964	9	42	18	296	236	354	629	155	269	283	-	0	208.2727
1965	0	-	22	55	305	442	304	480	300	50	131	-	208.9
1966	-	-	-	34	127	270	291	306	496	261	14	15	201.5556
1967	23	12	168	185	216	241	363	504	266	74	1	0	171.0833
1968	0	5	121	-	194	590	480	212	128	69	74	0	170.2727
1969	0	1	65	86	95	249	198	540	201	103	2	0	128.3333
1970	16	8	23	45	192	276	496	280	200	427	32	0	166.25
1971	3	28	-	-	344	339	550	540	-	118	-	-	274.5714
1972	0	11	12	248	340	353	249	380	-	105	0	0	154.3636
1973	-	21	32	131	621	414	-	238	348	128	64	86	208.3
1975	1	29	13	98	317	235	559	307	329	232	25	0	178.75
1976	0	7	117	34	459	627	346	361	165	114	8	0	186.5
1977	0	66	71	255	381	252	306	92	131	273	10	24	155.0833
1978	0	20	18	194	454	529	320	426	192	98	0	0	187.5833
1979	3	13	6	17	114	258	267	525	382	146	55	51	153.0833
1980	3	32	54	147	414	323	380	269	296	300	0	0	184.8333
1981	10	42	109	274	272	168	356	188	320	82	9	35	155.4167
1982	0	15	81	104	154	514	136	346	258	146	51	0	150.4167
1983	-	61	138	318	348	300	179	437	322	253	-	18	237.4
1984	13	1	5	124	707	637	694	311	478	58	0	0	252.3333
1985	8	1	195	176	300	399	262	317	306	79	0	10	171.0833
1986	22	0	23	247	191	304	443	171	687	237	172	3	208.3333
1987	4	0	33	230	109	316	526	462	363	104	7	33	182.25
1988	0	44	74	282	513	580	255	169	196	213	153	3	206.8333
1989	0	32	0	85	228	319	347	59	305	240	0	12	135.5833
1990	0	36	151	154	202	229	567	227	247	181	103	6	175.25
1991	27	8	46	53	529	320	318	345	692	392	14	106	237.5
1992	1	47	0	25	153	132	386	182	158	83	2	0	97.41667
1993	0	52	88	113	556	504	421	432	417	217	19	0	234.9167
1994	13	54	115	201	254	266	153	246	169	55	14	0	128.3333
1995	8	31	0	88	264	237	354	360	205	91	112	1	145.9167
1996	0	21	54	199	208	343	257	361	244	357	0	0	170.3333
1997	2	7	82	133	151	249	549	230	440	30	1	22	158
1998	49	4	83	178	405	89	521	552	246	100	83	0	192.5
1999	0	0	0	21	428	348	553	282	361	368	13	0	197.8333
2000	13	44	172	189	491	165	197	359	216	278	0	0	177
2001	0	1	33	46	402	386	202	205	209	177	18	0	139.9167
2002	22	4	51	111	272	373	446	272	156	52	116	0	156.25
2003	0	25	96	123	140	473	191	202	264	134	0	45	141.0833
2004	0	0	9	167	162	476	295	191	839	208	0	0	195.5833
2005	1	3	155	91	291	259	542	361	514	417	3	0	219.75
2006	0	0	0	181	185	326	331	167	663	61	5	0	159.9167
2007	0	30	11	163	185	628	753	505	179	320	111	0	240.4167
2008	23	56	45	91	205	577	563	319	279	227	0	0	198.75
2009	1	1	43	14	168	170	676	482	298	74	4	0	160.9167
2010	0	48	22	37	177	308	167	340	169	174	0	81	126.9167
Avg	5.97	20.6	57.229	136.06	289	362	389.5	312	311.45	172	31	11.5	

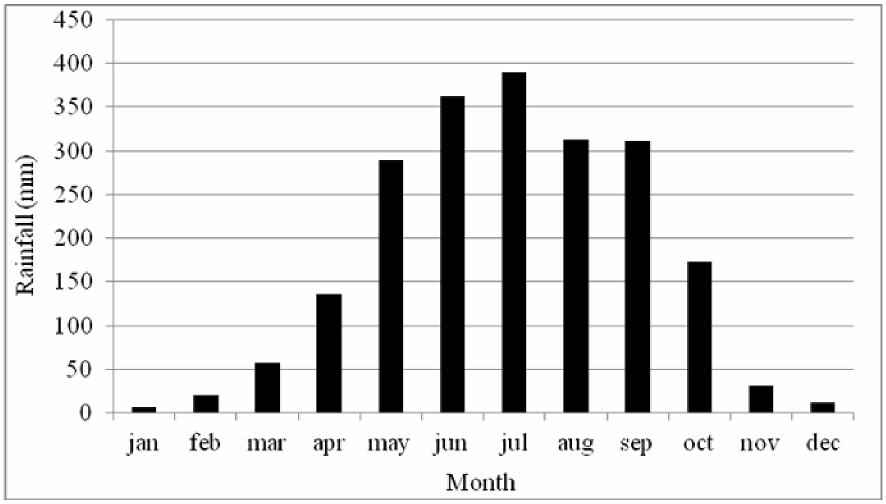


Figure 3: Average monthly rainfall for 50 years

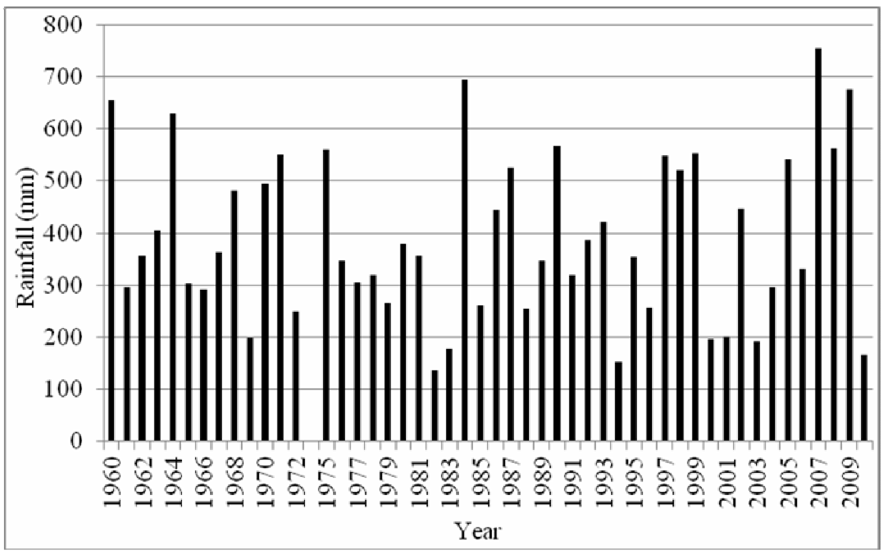


Figure 4: Variation of rainfall in July for 50 years

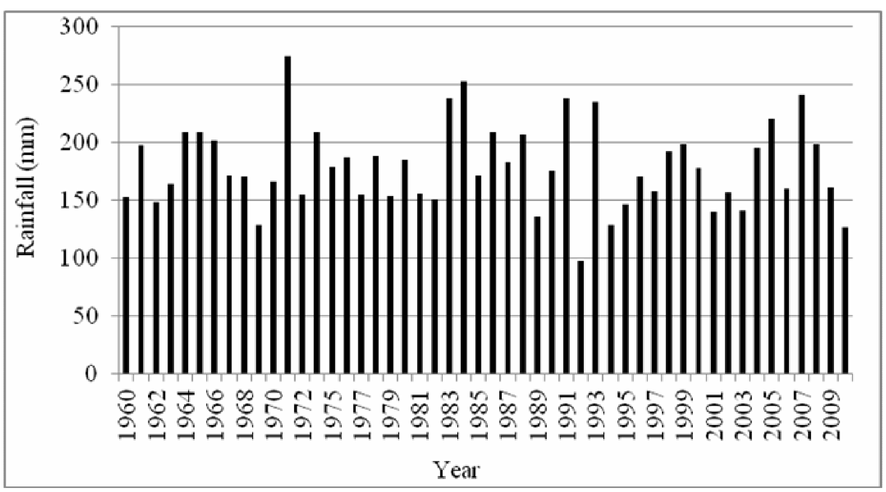


Figure 5: Deviation of average yearly rainfall for 50 years

7. CALCULATION

7.1 Assessment of Rainwater Availability

The availability of rainwater can be estimated from equation 1:

$$Q = CIA \qquad (1)$$

Where Q is the total quantity of rainwater available in m³/year, C is runoff coefficient, I is the rainfall intensity in m/year and A is the catchment area in m². Since a portion of the rainwater evaporates from the roof surface and a portion may be lost to the drainage system including the first flush i.e. washing the catchment area using first rain that produces inferior quality rainwater, a runoff coefficient depending upon catchment characteristics is introduced which plays a significant role in assessing the runoff availability. The value of runoff co-eficient for different types of catchment area is given in table 3.

Table 3: Runoff coefficient for different types of catchments (Patra and Gautam, 2011)

Types of Catchment	**Runoff Coefficient**
Roof top	0.75 – 0.95
Paved area	0.50 – 0.85
Green area	0.05 – 0.10

The amount of water from paved area (rooftop & roads) = 0.8 x 2.03132102 m/year x 31261.07 m²
= 50801.01489 m³/year
The amount of water from unpaved (green) area = 0.08 x 2.03132102 m/year x 90144.89 m²
= 14649.0568 m³/year
Total amount of harvested rainwater (in 8 months) = 65450.07168 m³/year. From this approximate amount of harvested rainwater on daily basis = 272.7086 m³/day.
Approximately 4 lakh litre or 400 m³ of water is required in IUT on daily basis. Required amount of water in 8 months = 400 x 8 x 30 m³ = 96000 m³.
Groundwater to be extracted after rainwater harvesting during 8 months = 96000 m³ - 65450.07168 m³
= 30549.928 m³

7.2 Energy & Cost Analysis

In IUT 3 pumps of 25 H.P. each runs 8 hours alternatively in everyday.
Power, P = 25 x 746 Watt [1 H.P. = 746 Watt] = 18650 Watt
The Kilowatt-hour or Unit can be estimated by equation 2.

$$W = \frac{Pt}{1000}\ Unit$$

(2)
Where, W is the work done in Kilowatt-hour , P is the power of pump in Watt and t is the time in hour.

Therefore, W = $\frac{18650 \times 8}{1000}$ Unit = 149.2 Unit
It is the amount of electricity Unit required for 1 pump running 8 hours. According to DESCO, The cost of each Unit for commercial or educational institution in Bangladesh is 5.85 BDT. For 8 months the cost of electricity required for 3 pumps = 149.2 x 3 x 30 x 8 x 5.85 BDT = 628430.4 BDT.

So, it is evident that during 8 months of a year 96000 m³ of groundwater extraction requires electricity cost of 628430.8 BDT. Therefore, groundwater to be extracted after rainwater harvesting i.e. 30549.928 m³ will require electricity cost of 199984.411 BDT. Annual electricity cost saving with rainwater harvesting = (628430.8 - 199984.411) BDT = 428446.389 BDT.

7.3 Storage Tank

A storage tank is the most important component of a rainwater harvesting system. The storage tank can be constructed from different materials depending on local situation. No fixed design or materials can't be selected as these technologies require field testing. For this case study selection of any water reservoir didn't required as the existing reservoir of IUT has the capacity to store and supply the collected rainwater as this study proposes to consume groundwater along with rainwater. There are two existing RCC water reservoir at IUT campus with the following requirements:

Water reservoir #1 having Length: 6.2 meter, Width: 8.21 meter, Height: 4.6 meter with total volume of 234.15 m^3 and Water reservoir #2 having Length: 6.12 meter, Width: 8.38 meter, Height: 4.6 meter with total volume of 235.914 m^3. Total volume of both water reservoirs = 234.15 + 235.914 = 470.064 m^3

Approximately total amount of average harvested rainwater on daily basis is 272.7086 m^3/day (from calculation). So, the existing water reservoirs are capable of storing and supplying the collected amount of rainwater without installing any other separate water reservoir. This will be also cost effective because the existing water distribution system of IUT is appropriate for the proposed RWH.

8. CONCLUSION

In Bangladesh, rainwater can be a good source of water supply in coastal, hilly and acute arsenic affected areas. It was clear that for the water scarce areas and the arsenic contaminated areas, the rainwater harvesting was very useful and acceptable as a low cost water supply technique. In the arsenic contaminated areas, this source might be an alternate option of water supply system. The study basically focuses on the minimization of groundwater consumption and energy saving due to rainwater harvesting for an educational institution. The research paper clearly finds out the RWH as a potentially safe, reliable and cost effective source of non-potable uses of water for at least 8 months of the year. Through rainwater harvesting the present consumption of groundwater and electricity can be reduced for sustainable development and the proposed RWH scheme for IUT may act as a model RWH scheme for other institutional organizations. Thus a huge amount of groundwater could be saved and a lot of electricity which may be used for other purposes or may be stored for future generation to keep the natural and environmental resources sustainable.

ACKNOWLEDGEMENTS

The authors would like to thank Md. Mafizur Rahman (Building Engineer of IUT), Mirza Mohammad Jahangir (Assistant Building Engineer of IUT), Md. Anwarul Islam (Pump operator of IUT), Mr. Dulal (Plumber of IUT) for their supports.

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ANALYZING DRAINAGE CHARACTERISTICS OF A SELECTED WARD IN KHULNA CITY CORPORATION

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ABSTRACT

In this study the existing drainage facilities of ward no. 22 of Khulna City Corporation (KCC) is studied, analyzed and a rainfall-runoff model is proposed to relate the rainfall-runoff response. To analyze the drainage characteristics efficiently, the study area is sub-divided into 8 sub-catchments. Horton's infiltration model and Manning's formula are used in the model to calculate the runoff losses and outlet capacity of drain, respectively. It is found that the existing drainage facilities are not adequate for the study area and the water logging condition happens if a rainfall exceeds intensity of 2 in/hr and duration of 2 hr, no water logging condition below this level and very negligible at this level. It is concluded that the proposed rainfall-runoff model could be used to predict the rainfall-runoff response for other urban catchment also.

Keywords: *Catchment area, Drainage characteristics, Excess rainfall, Infiltration, Rainfall intensity, Runoff.*

1. INTRODUCTION

Heavy rainfall is more likely to occur as a consequence of global warming (Mcbean, 2006; Mailhot et al., 2007). Moreover, heavy rainfall induced inundation is becoming more serious in urban areas which makes it necessary to urgently appraise and redesign the infrastructure system to drain the storm water more efficiently (Pan et. al., 2011). For most severe urban inundations, the rainfalls often exceed design rainfall values. But for many other minor cases where the rainfalls are below design standard, the urban inundations are caused by failure or improper operation of street inlets, pipe systems, and drainage river system. It is, therefore, necessary to re-design and re-operate infrastructure systems to drain the storm water more efficiently. To facilitate the optimal design and operation of urban drainage system, a sophisticated urban drainage model is required. In fact, the urban storm water flows through considerable complex pathways such as roofs, parking lots, squares, yards, roads, drainage pipelines, flyovers, pump stations, etc. (Hsu et al., 2000), and the flow regime changes substantially as the water moves along the various boundaries, which makes the urban storm water modeling a challenging task. A lot of research work has been done and a large number of urban storm flow simulation models were proposed. Hsu et al. (2000) used Storm Water Management Model (SWMM) and the two-dimensional diffusive flow model for the coupled simulation of surface inundation in the downtown area of Taipei caused by the overflow of the sewer network. Schmitt et al. (2004) built a dual drainage model RisUrSim which incorporated the detailed surface flow simulation and the interaction between surface and sewer flow. Nasello and Tucciarelli (2005) built a dual model of a double network formed by an upper network of open channels (street gutters) and a lower network of closed conduits (sewer pipes). Fang and Su (2006) built a coupled model of the surface and sewer network to simulate the inundation in the city of Beaumont, Texas caused by a tropical storm. In this study the existing drainage facilities of ward no. 22 of KCC is studied, analyzed and a rainfall-runoff model has been developed to relate the rainfall-runoff response.

2. METHODOLOGY

Ward. no. 22 has been selected for this study. Data such as KCC map, Ward no. 22 map, and other relevant information have been collected from Khulna City Corporation. Field work is also conducted to find out the length, dimensions and flow direction of various drains. Then drainage parameters and properties for the study area are analyzed. After that a rainfall-runoff model has been developed for an indicated rainfall intensity and duration. The steps in the development of the rainfall-runoff model are as follows:

i) selection of a particular rainfall with intensity (I inch/hr) and rainfall duration (t hr).
ii) sub-divide the rainfall duration into the smallest time interval Δt.

iii) calculation of losses using Horton's Infiltration Model for every time interval Δt by the following equation.

$$f = f_c + (f_0 - f_c)e^{-Kt} \tag{1}$$

where,
- f = infiltration after time "***t***" (in/hr)
- f_c = final infiltration capacity (in/hr)
- f_0 = initial infiltration capacity (in/hr)
- K = constant depending upon the soil characteristics
- t = duration of rainfall (hr)

The value of f_c, f_0 and K can be calculated from Table 1.

iv) calculation of excess rainfall by subtracting the losses from actual rainfall for every time interval Δt.
v) determination of outlet discharge capacity using Manning's equation for every time interval Δt.

Table 1: Horton's Infiltration Parameters (Akan, 1993)

Soil Type	f_0 (in/hr)
Dry sandy soil with little to no vegetation	5
Dry loam soil with little to no vegetation	4
Dry clay soil with little to no vegetation	1
Dry sandy soil with dense vegetation	10
Dry loam soil with dense vegetation	6
Dry clay soil with dense vegetation	2
Moist sandy soil with little to no vegetation	1.7
Moist loam soil with little to no vegetation	1
Moist clay soil with little to no vegetation	0.3
Moist sandy soil with dense vegetation	3.3
Moist loam soil with dense vegetation	2
Moist clay soil with dense vegetation	0.07

Soil Type	f_c (in/hr)	K (hr^{-1})
Clay loam, Silty clay loam	0-0.05	4.14
Sandy clay loams	0.05-0.15	4.14
Silt loam, loam	0.15-0.30	4.14
Sand, loamy sand, sandy loams	0.30-0.45	4.14

$$\text{Outlet discharge capacity (inches)} = \frac{Q}{A_c} \times \Delta t \tag{2}$$

Runoff depth (t+Δt) = Runoff depth (t) + Excess Rainfall (Δt) if any − Outlet Discharge (Δt) (3)

Where, Q = outlet discharge capacity by Manning's equation (ft^3/s) $= \frac{\varphi}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$ (4)
- A_c = catchment area (ft^2)
- φ = 1.486 (1 for SI unit)
- n = Manning's roughness coefficient
- A = cross-sectional area of drain (ft^2)
- R = hydraulic radius of drain (ft)
- S = slope of drain (--)

vi) calculation of runoff depth in the catchment at any time interval (t+Δt) by subtracting the runoff depth at Δt time earlier plus excess rainfall increment if any and minus the outlet discharge increment.
vii) The runoff depth in the catchment will be treated as water logging depth for the catchment.

The model is then applied to the catchments for different rainfall duration and intensity to relate the rainfall-runoff response. Finally, a graphical representation for every identical rainfall has been developed.

3. STUDY AREA AND DATA COLLECTION

Khulna City Corporation (KCC) is third biggest city corporation in Bangladesh. It is situated at the bank of the river Rupsha and between 21.38^0 and 23.1^0 north latitude and 88.58^0 east latitude. It is declared as City Corporation on August 6, 1990. The area of this City Corporation is 45.65 km^2 and population is over 1.4 million. There are 31 wards in Khulna City Corporation (KCC). The study area is ward no. 22, which starts from Circuit House More and ends at Rupsha Ferri Ghat. It has boundary with ward no. 21, ward no. 23 and ward no. 29. The eastern part of the ward is in touch with river Rupsha. There are three big outlets situated in this ward carrying all the water coming from the main Khulna City. The water comes from rainfall and human being discharge into the nearby river Rupsha. The total length of drain in this ward is 27.5 km and area covered is $1.70m^2$. The map of ward no. 22 is shown in Figure 1.

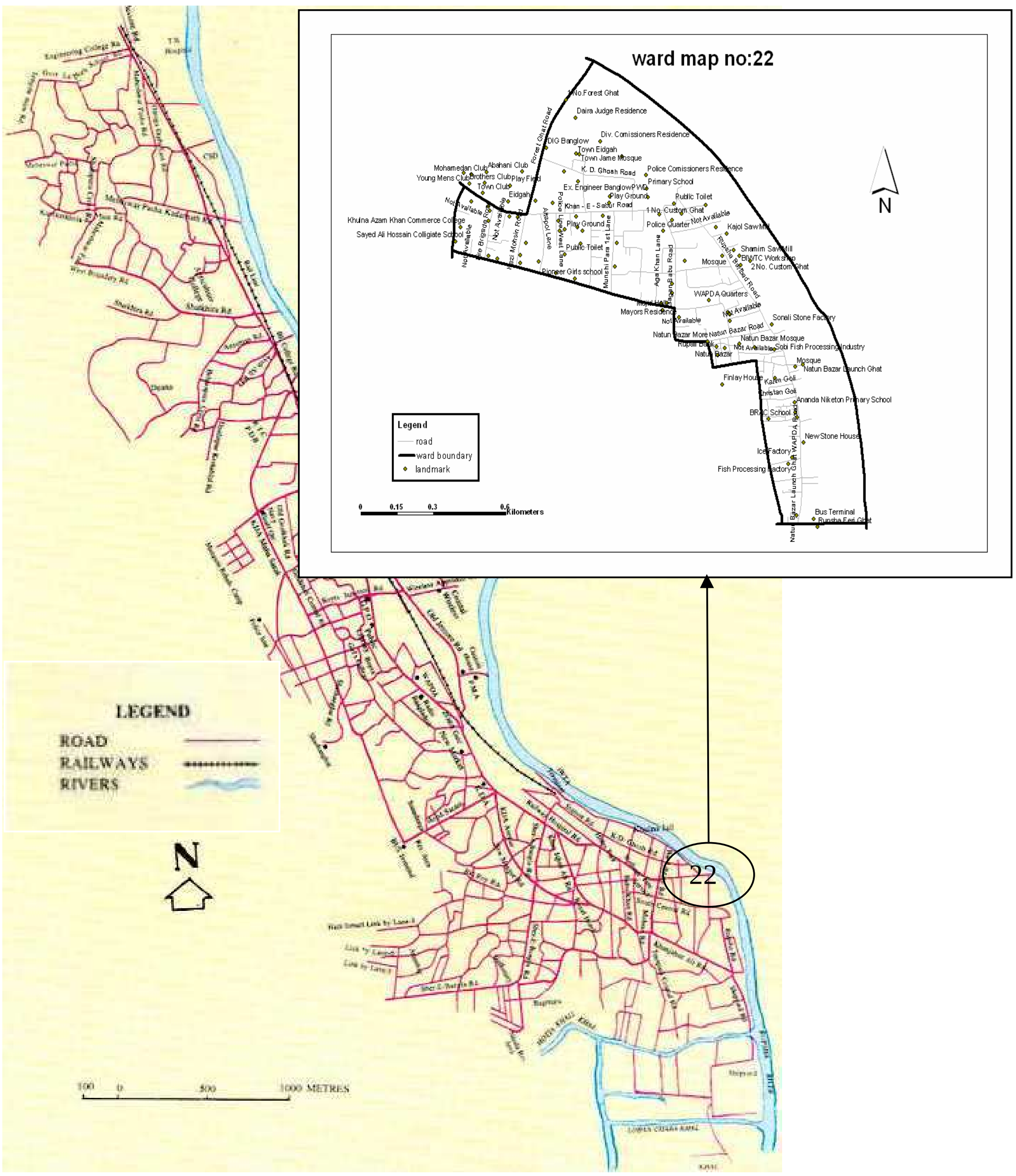


Fig. 3.1: Location and map of the study area

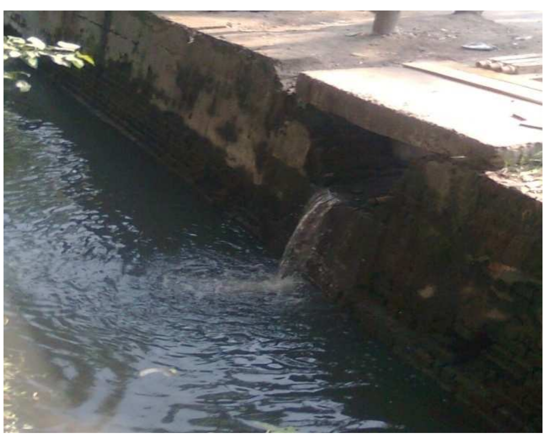
Data such as KCC map, ward map etc. have been collected from Khulna City Corporation. Field work has been carried out to collect data such as dimensions, length and flow directions of all drains in the ward. Photographs of drains and outlets at different locations are also taken. The length of various types of drains is also obtained The area occupied by the study area is also obtained from the map by using map scale. The dimensions of different types of drains are given in Table 2. There are 3 outlets through which whole runoff from the study area discharge in to the river Rupsha. The pictorial view of various important locations has been collected during the study. Some of those pictures are shown in Figure 2.

Table 2: Dimensions of Different Types of Drains

Drain type	D_1	D_2	D_3	D_4	D_5	D_6	D_7
Width (ft)	6	4	5	4	3	4	2
Depth (ft)	6	6	6	4	4	2	2
Length (km)	0.5	4.8	2.7	8.3	3.5	2.7	4.5

(a) Flow reduced due to garbage at Munsi Para

(b) Outlet point at 2 no. custom Ghat



(c) Drain at 1 no. Custom Ghat

(d) Drain at 1 no. Custom Ghat

Figure 2: Some pictorial views of drains at the study area

4. ANALYSIS AND RESULTS

In this study the existing drainage facilities of ward no. 22 of KCC is studied and analyzed. Data such as KCC map, Ward no. 22 map, and other relevant information have been collected from Khulna City Corporation. Field work is also conducted to find out the length, dimensions and flow direction of various drains. Three outlets are found through which whole runoff from the study area discharges in to the river Rupsha. To analyze the drainage characteristics efficiently, the total catchment area is sub-divided into 8 sub-catchments (Figure 3). The area occupied by each sub-catchments areas are also calculated. The Drainage network including different stream orders of the catchments for the ward no.22 of KCC are also shown in Figure 3. Drainage parameters and drainage properties for the catchments are given in Table 3.

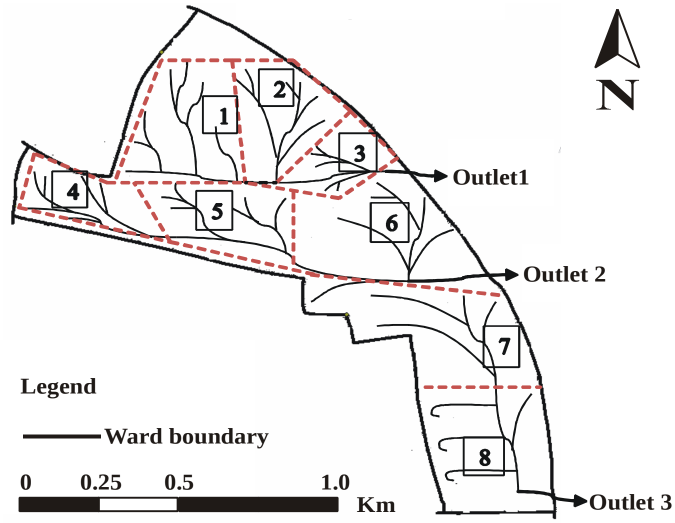
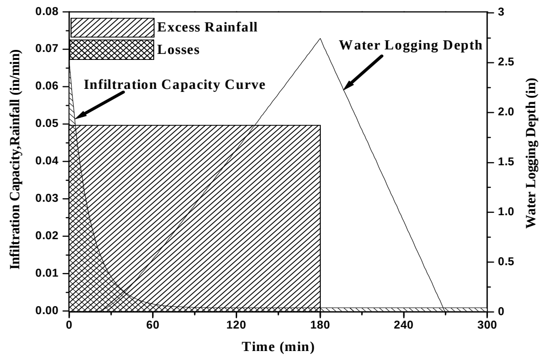


Figure 3: Drainage network for the Ward no. 22

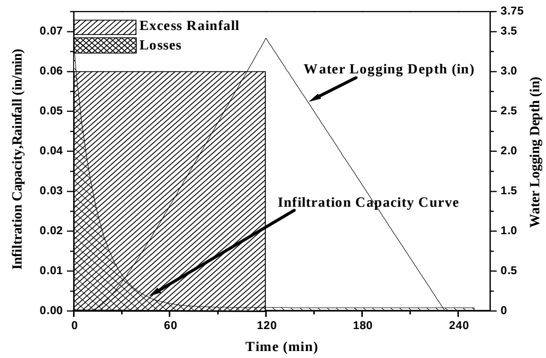
Table 3: Drainage properties for Ward no. 22 of KCC

Outlet	Sub-catchment area	Stream Length (km) and no. of streams in parenthesis			Occupied area (km^2)	Drainage density D_d (km/km^2)	Stream density D_s (1/km^2)
		Order 1	Order 2	Order 3			
1	1	0.215 (7)	0.175 (4)	0.124 (2)	0.054	45.4	240.7
	2	0.146 (6)	0.105 (3)	0.120 (1)	0.061	21.5	163.9
	3	0.317 (6)	0.215 (4)	0.110 (1)	0.021	136.8	381.0
2	4	0.175 (6)	0.100 (2)	0.120 (3)	0.093	17.3	118.3
	5	0.228 (4)	0.145 (3)	--	0.097	13.9	72.2
	6	0.165 (3)	0.095 (2)	--	0.122	5.6	41.0
3	7	0.195 (5)	0.090 (4)	--	0.072	18.5	125.0
	8	0.220 (5)	0.125 (4)	--	0.110	14.5	134.3

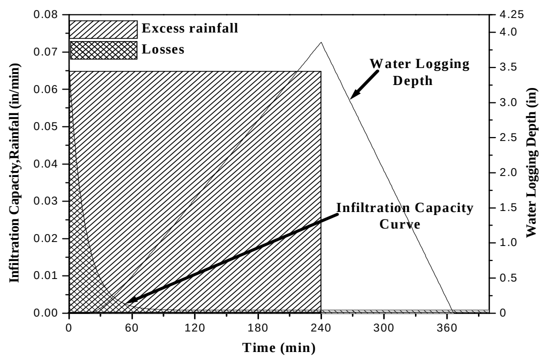
Figure 4 shows the graphical representation of the model out put for four different rainfalls. In field observation, the soil characteristics of the study area was found dry loam soil with little to no vegetation for which initial infiltration capacity, $f_0 = 4$ (in/hr), final infiltration capacity, $f_c = 0.05$ (in/hr) and constant depending upon the soil characteristics, $K = 4.14$ (hr^{-1}) were used for calculating the losses by Horton model (Table 1). For estimating the drain capacity by Manning's equation, the slope of drains is assumed as 1 in 400 (ft/ft) and the Manning's n is taken as 0.015 (Chow, 1959). The model is run for the incremental time interval of 'Δt' of 1 min. The model is analyzed for different 8 pairs of rainfall intensities and rainfall durations such as rainfall intensity = 2 in/hr and rainfall duration = 1 hr, rainfall intensity = 2 in/hr and rainfall duration = 2 hr, rainfall intensity = 3 in/hr and rainfall duration = 2 hr, rainfall intensity = 4 in/hr and rainfall duration = 2 hr, rainfall intensity = 3 in/hr and rainfall duration = 3 hr, rainfall intensity = 4 in/hr and rainfall duration = 3 hr, rainfall intensity = 3 in/hr and rainfall duration = 4 hr and rainfall intensity = 4 in/hr and rainfall duration = 4 hr. Table 4 shows the above 8 pairs of rainfall intensities and rainfall durations input and corresponding rainfall-runoff response from the model. The rainfall duration is divided into minutes and rainfall intensity is converted into in/min. The initial infiltration capacity $f_{0,}$ final infiltration capacity f_c and constant depending upon the soil characteristics K are converted to in/min and min^{-1}, respectively. Horton's infiltration losses is calculated in in/min also. The model shows water logging depth in the catchment after initiation of any identical rainfall in inches. The analysis also shows how long it takes to remove stagnant water after initiation of rainfall. Moreover, the analysis shows that for which rainfall water logging condition will happen and for which rainfall water logging condition will not happen. It was found that the water logging condition happens when rainfall intensity is above 2 in/hr and duration is above 2 hr. Water logging depth and time requirement for draining out stagnant water for different rainfall intensities and rainfall durations are shown in Table 5.



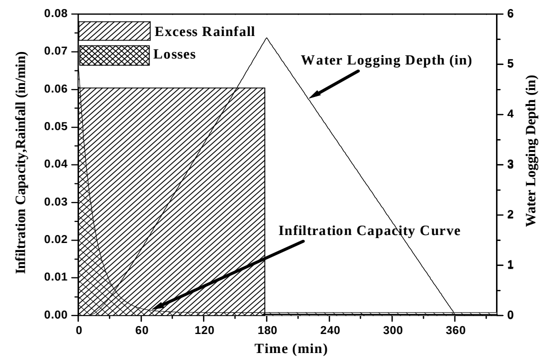
(a) Rainfall Intensity = 3 (in/hr); Duration = 3hr



(b) Rainfall Intensity = 4 (in/hr); Duration = 2hr



(c) Rainfall Intensity = 3 (in/hr); Duration = 4 hr



(d) Rainfall Intensity =4 (in/hr); Duration = 3 hr

Figure 4: Results from the developed rainfall-runoff model for typical indicated rainfalls.

Table 4: Rainfall-Runoff response from the model for Ward no. 22 of KCC

Rainfall intensity (in/hr)	2	2	3	4	3	4	3	4
Rainfall Duration (hr)	1	2	2	2	3	3	4	4
Duration (hr)	Water logging depth in the catchment after initiation of rainfall (in)							
0	0	0	0	0	0	0	0	0
0.25	0	0	0	0.044	0	0.044	0	0.044
0.5	0	0	0.073	0.361	0.073	0.361	0.073	0.361
0.75	0	0	0.277	0.815	0.277	0.815	0.277	0.815
1.0	0	0.005	0.529	1.317	0.529	1.317	0.529	1.317
1.25	0	0.024	0.798	1.837	0.798	1.837	0.798	1.837
1.5	0	0.049	1.074	2.362	1.074	2.362	1.074	2.362
1.75	0	0.077	1.351	2.89	1.351	2.89	1.351	2.89
2.0	0	0.105	1.63	3.418	1.63	3.418	1.63	3.418
2.25	0	0	1.171	2.959	1.908	3.947	1.908	3.947
2.5	0	0	0.712	2.501	2.187	4.475	2.187	4.475
2.75	0	0	0	1.736	2.652	5.357	2.652	5.357
3.0	0	0	0	1.583	2.745	5.533	2.745	5.533
3.25	0	0	0	1.124	2.286	5.074	3.023	6.062
3.5	0	0	0	0.666	1.827	4.615	3.302	6.59
3.75	0	0	0	0.207	1.369	4.157	3.581	7.119
4.0	0	0	0	0	0.91	3.698	3.86	7.648
4.25	0	0	0	0	0.451	3.239	3.401	7.189
4.5	0	0	0	0	0	2.781	2.942	6.731
4.75	0	0	0	0	0	2.322	2.484	6.272
5.0	0	0	0	0	0	1.863	2.025	5.813
5.25	0	0	0	0	0	1.404	1.566	5.354
5.5	0	0	0	0	0	0.946	1.108	4.896
5.75	0	0	0	0	0	0.487	0.649	4.437
6.0	0	0	0	0	0	0.028	0.19	3.978
6.25	0	0	0	0	0	0	0	3.52
6.5	0	0	0	0	0	0	0	3.061
6.75	0	0	0	0	0	0	0	2.602
7.0	0	0	0	0	0	0	0	2.143
7.25	0	0	0	0	0	0	0	1.685
7.5	0	0	0	0	0	0	0	1.226
7.75	0	0	0	0	0	0	0	0.767
8.0	0	0	0	0	0	0	0	0.309
8.25	0	0	0	0	0	0	0	0
8.5	0	0	0	0	0	0	0	0

Table 5: Summary of rainfall-runoff response from the model for Ward no. 22 of KCC

Rainfall Intensity (in/hr)	2	2	3	4	3	4	3	4
Rainfall Duration (hr)	1	2	2	2	3	3	4	4
Maximum water logging Depth (in)	0	0	1.7	3.5	2.75	5.6	4.0	7.75
Time requirement to drain out storm water (hr)	0	0	2.5	3.75	4.25	6	6	8

5. Conclusions

In this study, a rainfall-runoff model is proposed to investigate the rainfall-runoff response of the study area. Horton's infiltration model and Manning's formula are used to calculate the runoff losses and outlet capacity of the drain, respectively in the model. It is concluded that the existing drainage facility is not adequate for the study area, ward no. 22 and the water logging condition would be happened if a rainfall exceeds intensity of 2 in/hr and duration of 2 hr, no water logging condition below this level and very negligible at this level. The analysis also shows how long it takes to remove stagnant water after initiation of rainfall. The developed rainfall-runoff model could be used to predict the rainfall-runoff response for the other urban catchment also.

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WATER DESALINATION USING BASIN TYPE SOLAR STILL

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ABSTRACT

The aim of this study is to design and construct a Basin Type Solar Still (BSS) evaluating the production rate and cost compared with Tubular Type Solar Still (TSS). In this study two types of BSS, conventional (BSS1) and stepped (BSS2), were designed, constructed and field experiments have been carried at KUET) since January 2008 to evaluate the performances of the stills. The average daily productions are found as 3.97 and 2.97 lit/m^2- for the BSS1 and BSS2, respectively. Hence, daily production is found 12.1% higher for the BSS2 than of BSS1. The construction costs are found as Tk. 1500 and Tk. 1400 for the BSS1 and BSS2, respectively. The production cost of water from the still are estimated as 0.20 and 0.26 Tk./lit for the BSS2 and BSS1, respectively. Considering economic point of view, BSS2 is cost effective than BSS1. As the construction, operation and maintenance of stepped BSS are easy, it can be used for drinking and other purposes in remote, coastal and arid areas or in an emergency to meet the small scale fresh water demand.

Keyword: *Basin type solar still (BSS), Conventional basin type still (BSS1), Stepped basin type still (BSS2), Solar distillation.*

1. INTRODUCTION

The scarcity of fresh water is increasing day by day in arid and remote areas of Bangladesh. In south western region of Bangladesh, shallow aquifers contain arsenic, exceeding the allowable limit for Bangladesh standard (0.05 mg/l) and highly saline water exists in deep aquifers. Salinity and arsenic in water poses a serious problem for the development of appropriate water supply system in such region. In these areas desalination techniques could be applied to meet fresh water demand produced from brackish or saline water. Most desalination techniques such as reverse osmosis, electro-dialysis, multi-stage flash etc. consume a huge amount of external energy e.g. fossil fuel/electricity. Therefore, finding methods of using renewable energy to power the desalination process is desirable. Solar distillation is a simple desalination technique in which only solar energy is needed. A basin type solar still (BSS) is the most popular method of solar distillation compared with others due to its simplicity. It could be one of the viable options for providing drinking water for a single house or a small community in arid, remote and coastal regions. For better understanding the production mechanism and improve the performance of a solar still, many researchers (Chaibi, 2000; Clark, 1990; Cooper, 1969; Dunkle, 1961; Hongfei et al., 2002; Malik et al., 1982; Shawaqfeh and Farid, 1995 and Islam et. al., 2007) have mainly conducted experimental and theoretical studies on conventional basin type still rather than other types. On the other hand, uses of water from underground and surface water sources are not always desirable or possible because of the presence of large amount of salts especially in coastal areas. Excessive salinity causes various health hazard and diseases in coastal region. In these areas basin type solar still (BSS) could be a suitable technique for supplying potable water. The method will serve the community with fresh water reducing the harmful effect. In this study, two basin types solar stills (conventional and stepped type), were designed, constructed and field experiments have been carried out on the roof top of Civil Engineering building of Khulna University of Engineering & Technology (KUET) since January 2008.

1. METHODOLOGY

Two basin types solar still (conventional and stepped type), were designed and constructed using locally available materials. They consisted of rectangular ferrocement basin with a transparent glass cover on its top and all for sides and rectangular ferrocement trough(s) for storing saline water. Field experiments are conducted using the constructed stills. Daily distilled water production and hourly production of some typical days are recorded. Collected data was analyzed and correlations are proposed for daily output. Finally, the water production cost is estimated and conclusions are drawn. The steps of the methodology are shown in the flow chart given in Figure 1.

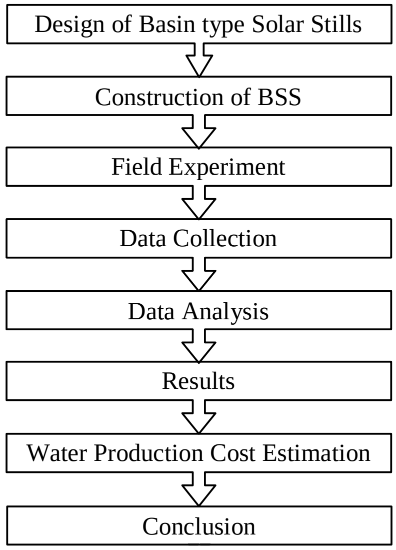
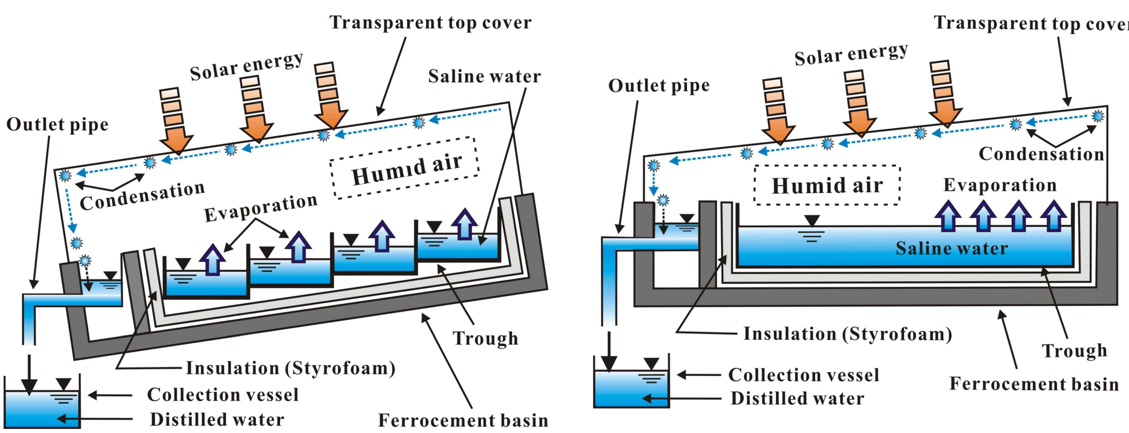


Figure 1: Flow chart showing steps of the work

2. PRODUCTION PRINCIPLE OF BASIN TYPE SOLAR STILL (BSS)

Production principle of basin type solar still (BSS) is illustrated in Figure 2. The solar radiation, after transmission through a transparent glass cover, is mainly absorbed by saline water in the trough. The glass cover and trough absorb the remaining small amount of the solar energy. Thus the water in the trough is heated and then evaporation begins. Many types of heat transfer occur inside the glass and outside, e.g., evaporative heat transfer from the saline water to the humid air, condensative heat transfer from the humid air to the glass cover, convective heat transfer between the saline water and the humid air, the trough and the water, the trough and the humid air, the humid air and the cover and the glass cover and the atmosphere, radiative heat transfer between the water surface and the glass cover and the glass cover and the atmosphere. The evaporated water vapor is transferred to the humid air and then finally condensed on the top glass cover inner surface, releasing its latent heat of vaporization. The condensed water trickles down the bottom of the glass cover inner surface due to gravity and is stored in a collection bottle through a pipe provided at the lower end.



a) Stepped basin type still b) Conventional basin type still

Fgure 2: Production principle of basin type still

3. MASS AND ENERGY BALANCE EQUATIONS

3.1 Assumptions

The mass and energy balance equations are made up on the following assumptions:

i) Heat and mass transfer in a BSS are formalized using the representative temperature of the saline water, humid air, trough and tubular cover and the relative humidity of the humid air.
ii) Water vapor on the water surface is saturated.
iii) There is no water vapor leakage across the still.
iv) The absorption of the solar radiation in the humid air is negligibly small.

Figure 3 shows the mass and energy transfer of a BSS.

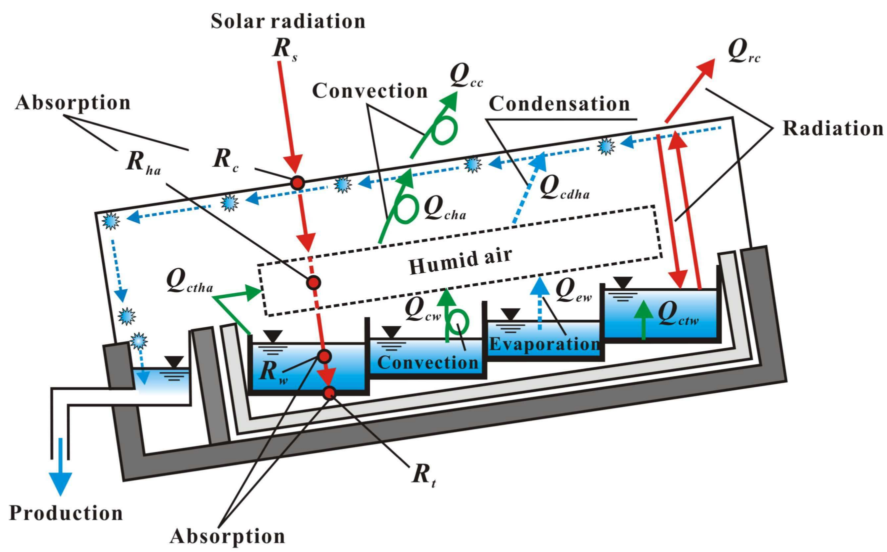


Figure 3: Mass and energy transfer of a basin type solar still

3.2 Mass Balance Equations

Saline Water in Trough

The mass reduction of the saline water in a trough is prescribed by the evaporation from the saline water surface and the time rate of the mass reduction is written as:

$$\frac{dh_w}{dt} = -\frac{m_{evap}}{\rho_w} \tag{1}$$

where, h_w is the depth of water in trough (m^3), m_{evap} is the evaporation flux (kg/m^{2}s), ρ_w is the density of water (kg/m^3) and t is time (s).

Water Vapor in Humid Air

The mass balance of water vapor in a BSS is prescribed by the evaporation from the saline water surface and the condensation on the inner cover surface and the time rate of the mass of water vapor can be expressed as the difference between the evaporation and the condensation, i.e,

$$V_{ha}\frac{\partial \rho_{vha}}{\partial t} = A_w m_{evap} - A_c m_{cond} \tag{2}$$

where, V_{ha} is the volume of humid air (m^3), ρ_{vha} is the vapor density of the humid air (kg/m^3), A_c is the top condensation cover surface area (m^2), and m_{cond} is the condensation flux (kg/m^{2}s). m_{evap} and m_{cond} may be calculated from the following equations, respectively:

$$m_{evap} = h_{ew}\left(\rho_{vw} - \rho_{vha}\right) \tag{3}$$

$$m_{cond} = h_{cdha}\left(\rho_{vha} - \rho_{vc}\right) \tag{4}$$

where, h_{ew} is the evaporative mass transfer coefficient (m/s), h_{cdha} is the condensative mass transfer coefficient (m/s), ρ_{vw} is the vapor density on the saline water surface (kg/m^3) and ρ_{vc} is the vapor density on the condensation cover (kg/m^3).

3. 3 Energy Balance Equations

The energy balance equations of the TSS may be expressed according to the mass and energy transfer shown in Figure 3.

Saline water: $$(\rho C)_w \frac{\partial (V_w T_w)}{\partial t} = R_w + Q_{ctw} - Q_{ew} - Q_{cw} - Q_{rw} \tag{5}$$

Trough: $$(\rho CV)_t \frac{\partial T_t}{\partial t} = R_t - Q_{ctw} - Q_{ctha} \tag{6}$$

Humid air: $$(\rho CV)_{ha} \frac{\partial T_{ha}}{\partial t} = R_{ha} + Q_{ew} + Q_{cw} + Q_{ctha} - Q_{cha} - Q_{cdha} \tag{7}$$

Top cover: $$(\rho CV)_c \frac{\partial (T_c)}{\partial t} = R_c + Q_{cha} + Q_{cdha} + Q_{rw} - Q_{cc} - Q_{rc} \tag{8}$$

where,

$Q_{cc} = h_{cc} (T_c - T_a) A_c$
$Q_{cdha} = Lh_{cdha} (\rho_{vha} - \rho_{vc}) A_c$
$Q_{cha} = h_{cha} (T_{ha} - T_c) A_c$
$Q_{ctha} = h_{ctha} (T_t - T_{ha}) A_{tha}$
$Q_{ctw} = h_{ctw} (T_t - T_w) A_{tw}$
$Q_{cw} = h_{cw} (T_w - T_{ha}) A_w$
$Q_{ew} = Lh_{ew} (\rho_{vw} - \rho_{vha}) A_w$
$Q_{rc} = h_{rc} (T_c - T_a) A_c$
$Q_{rw} = h_{rw} (T_w - T_c) A_w$

in which,

A	:	area	(m^2)
A_{tha}	:	contact surface area between trough and humid air	(m^2)
A_{tw}	:	contact surface area between trough and water	(m^2)
C	:	specific heat capacity	(J/kg ^oC)
h_{cc}	:	convective heat transfer coefficient between cover and atmosphere	($W/m^2\ ^oC$)
h_{cdha}	:	condensative mass transfer coefficient from humid air to tubular cover	(m/s)
h_{cha}	:	convective heat transfer coefficient between humid air and tubular cover	($W/m^2\ ^oC$)
h_{ctha}	:	convective heat transfer coefficient between trough and humid air	($W/m^2\ ^oC$)
h_{ctw}	:	convective heat transfer coefficient between trough and water	($W/m^2\ ^oC$)
h_{cw}	:	convective heat transfer coefficient between water surface and humid air	($W/m^2\ ^oC$)
h_{ew}	:	evaporative mass transfer coefficient from water surface to humid air	(m/s)
h_{rc}	:	radiative heat transfer coefficient between tubular cover and atmosphere	($W/m^2\ ^oC$)
h_{rw}	:	radiative heat transfer coefficient between water surface and tubular cover	($W/m^2\ ^oC$)
$\boldsymbol{L}$	:	latent heat of vaporization	(J/kg)
Q_{cc}	:	convective heat transferred between tubular cover and atmosphere	(J/s)
Q_{cdha}	:	condensative heat transferred from humid air to tubular cover	(J/s)
Q_{cha}	:	convective heat transferred between humid air and tubular cover	(J/s)
Q_{ctha}	:	convective heat transferred between trough and humid air	(J/s)
Q_{ctw}	:	convective heat transferred between trough and water	(J/s)
Q_{cw}	:	convective heat transferred between water surface and humid air	(J/s)
Q_{ew}	:	evaporative heat transferred from water surface and humid air	(J/s)
Q_{rc}	:	radiative heat transferred between tubular cover and atmosphere	(J/s)
Q_{rw}	:	radiative heat transferred between water surface and tubular cover	(J/s)
T	:	temperature	(^oC)
V	:	volume	(m^3)
ρ	:	density	(kg/m^3)

The subscripts *a*, *c*, *ha*, *t* and *w* denote atmosphere, cover, humid air, trough and saline water, respectively. For details of mass and energy balances of a still reader are referred to Islam et. al. (2007).

4. DESIGN, CONSTRUCTION AND FIELD EXPERIMENT

4.1 Design and Construction of Basin Type Solar Stills

Two basin type solar still, conventional type treated as BSS1 and stepped type treated as BSS2, were designed and constructed using locally available materials. Both stills are 100cm long, 70cm wide, 10 cm deep and made of 2.5cm thick ferro-cement materials. All four sides and the top of basins are covered with 4mm thickness trasparent glass. The inclination of the top glasses is 10° with horizontal. The height of glasses in all four sides of the BSS2 and the minimum height of glass downward side of the top glass cover of the BSS1 are 2.5 inch. A rectangular trough 90cm long, 60cm wide, 5cm deep and made of 2.5cm thick ferro-cement materials is placed inside the main basin. The trough for the BSS2 is comprised of four sets of stepped rectangular tray of 90cm long, 13cm wide and 5cm deep. Inside surface and bottom of the main basin are insulated by styrofoam (known as cork sheet) of thickness 2.5cm to prevent the heat losses from inside the still. The distillated water is collected into a bottle everyday approximately two hours after the sunset. The distilled water collection bottle is put in insulation box in order to collect the amount of distilled water from the still accurately.

In 2010, the basin was stepped and comprised of 86cm long, 66cm wide and 22cm deep as ferro-cement basin, which is covered with glass as a top cover only. The still had rectangular small basin 74cm long, 51cm wide and 5cm deep, which contained four black rectangular troughs of 70cm long, 10.5 cm wide and 5cm deep for storing saline water. In 2008, the basin was stepped and composed of rectangular basin and three sets of rectangular trough for storing saline water. The rectangular basin was 60cm long, 45cm wide and 8cm deep and made of 2.5cm thickness Ferro-cement materials. The trough was made of 0.25mm thick GI sheet 53cm long, 13cm wide, 4cm deep and painted black. Only the top of basin is covered with 4mm thickness trasparent glass and no sides were covered by glasses.

4.2 FIELD EXPERIMENT

The field experiments have been carried out on the roof top of the Civil Engineering building of Khulna University of Engineering and Technology (KUET) since January 2008. Figure 4 shows the photograph of the field experiment. The vessel for distilled water collection was put in a wooden box covered (inside) with styrofoam (heat insulator) in order to collect the amount of distilled water from the BSS accurately. One end of the TSS was kept fixed and other end could open to clean the trough or to remove the accumulated brine and to feed the saline water in the trough. The daily output from the still is collected approximately two hours after sunset. The hourly output was also measured for some typical days. The hourly outputs were also measured in some typical days to observe the hourly variation of the productivity of the still. Solar radiation flux and ambient air temperature were also measured at one minute interval using a data logger. A pyranometer and thermocouple were used to measure the solar radiation flux and temperature, respectively.

5. ANALYSIS AND RESULTS

Hourly and daily-distilled water output from the BSS are used to calculate the daily and hourly production rate per unit surface area of the saline water in the trough. Figure 5 shows the variations of hourly production rate (Kg/m^2-hr) for three typical days, from March 16 to 18 of 2010 for the stepped BSS. In a particular day (March 18) solar radiation flux and air temperature are also shown. It is observed from the figure that the solar radiation flux and air temperature increased gradually in the morning till 12.00, then decrease in the afternoon. Whereas, the production was recorded from 9.00 in the morning, increased gradually up to 14.00 and then declined in the afternoon. It was also seen that the slope of the hourly production rate in the morning is steeper than that of the afternoon.

Figure 6(a) and (b) show the variation of hourly production rate and ambient air temperature for May 02, 03 and 10 of 2011 for the BSS1 and BSS2, respectively. It is observed from the figure that the air temperature increases gradually in the morning, peak approximately 13.00 and then decreases in the afternoon. Whereas, the production recorded from 8.00 in the morning, increases gradually and peak between 14.00 to 15:00 and then decreases. It is also seen that the slope of hourly production rate in the morning is steeper than that of the afternoon. The daily productions for May 02, 03 and 10 of 2011 are found as 3.55, 2.96, 4.08 lit/m^2-day (1.92, 1.60, 2.20 lit/day) and 3.66, 3.73, 4.48 lit/m^2-day (1.71, 1.75, 2.10 lit/day) for the BSS1 and BSS2, respectively. The average daily productions for these 3 days are found as 3.53 lit/m^2-day (1.91 lit/day) and 3.96 lit/m^2-day (1.85 lit/day) for the BSS1 and BSS2, respectively. Hence, daily production is found 12.1% higher for the BSS2 (stepped type BSS) than of conventional BSS.

Figure 7 shows the variations of the daily production for the conventional and stepped type BSS from January to May of 2011, 2010 and 2008. It is seen from the figure that the production rate for all cases increases gradually from January to May. The average daily production rates are found as 3.97, 3.56, 2.97 and 1.83 lit/m^2-day (1.86, 1.05, 1.60 and 0.38 lit/day) for the stepped 2011, stepped 2010, conventional 2011 and stepped 2008 basin type solar stills, respectively. It is seen from the figure that the output from the stepped type sill is higher than the conventional type. I is also clear that the production is increased after using the side walls as a glass cover.

Figure 4: Photograph of field experiment carried out at KUET from 2008 to 2011.

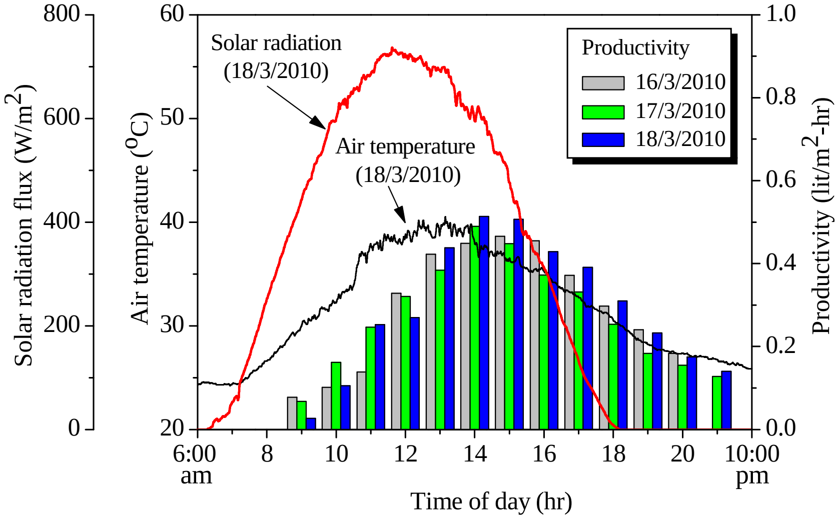
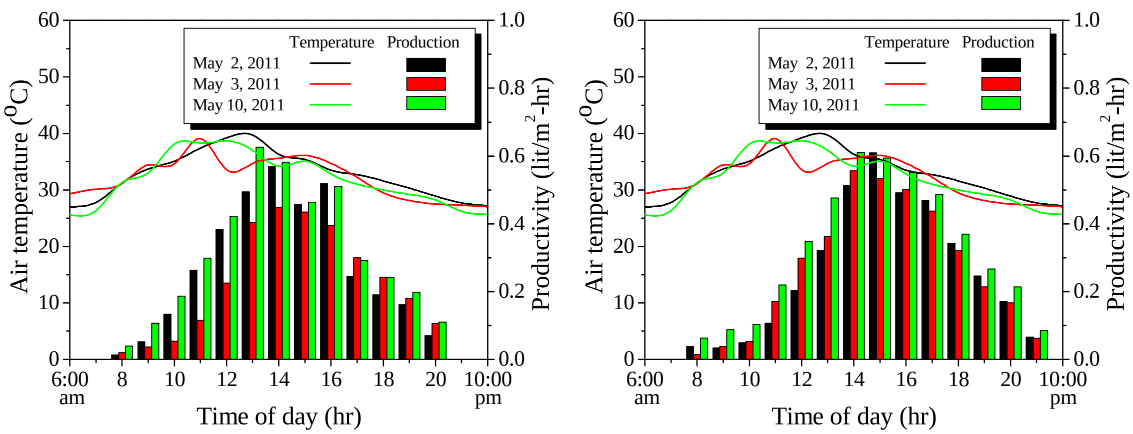


Figure 5: Observed diurnal variations of ambient air temperature, radiation flux and hourly production for the stepped BSS at KUET, Khulna from March 16 to 18 of 2010.



a) Conventional type (BSS1)

b) Stepped type (BSS2)

Figure 6: Variation of hourly production rate and ambient air temperature for May 02, 03 and 10 of 2011 for the conventional and stepped type still

6. COST ESTIMATION

The construction cost of each stepped and conventional type BSS are found as Tk. 1400 and Tk. 1500, respectively (Table 1 and 2). The production cost of water from the still are estimated as 0.20 and 0.26 Tk./lit for the stepped and conventional BSS, respectively (Table 3). Table 4 shows the summary of the cost analysis of the different type of solar stills. Though the total cost including the initial cost of a Tubular Solar Still (TSS) is very low but the production cost of the water is higher than that of the BSS.

Table 1: Cost estimation for conventional type BSS (BSS1)

Sl. No.	Item Description	Unit	Rate (Tk.)	Quantity	Amount in Tk.
1	Cement	kg	10	35	350
2	Sand	cft	20	2	40
3	G.I. wire etc.	kg	75	2.5	188
4	Transparent glass	sft	40	16	640
5	Cork sheet	--	150	1	150
6	Miscellaneous				132
				Total =	1500

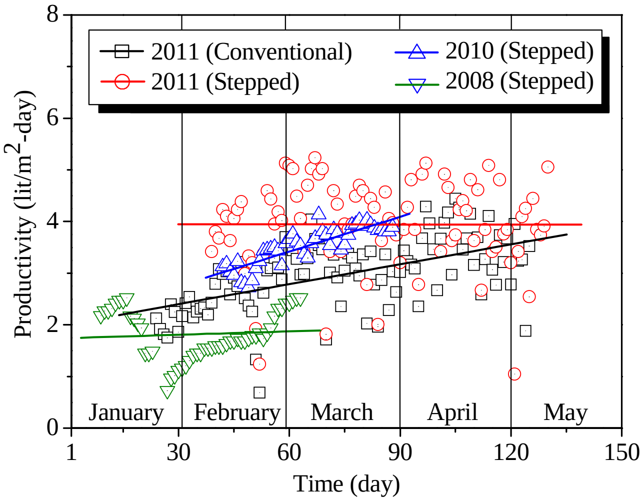


Figure 7: Variation of hourly production rate from January to May of 2008, 2010 and 2011 for the conventional and stepped type still

Table 2: Cost estimation for stepped type BSS (BSS2)

Sl. No.	Item Description	Unit	Rate (Tk.)	Quantity	Amount in Tk.
1	Cement	kg	10	40	400
2	Sand	cft	20	2	40
3	G.I. wire etc.	kg	75	2.5	188
4	Transparent glass	sft	40	12	480
5	Cork sheet	--	150	1	142
6	miscellaneous				256
				Total=	1400

Table 3: Production cost of water for Stepped and conventional type BSS

Description	Stepped type BSS	Conventional BSS
Design life of a BSS (years)	10	10
Construction cost of the Still including maintenance (Tk.)	1400	1500
Average production of water (lit/day)	1.86	1.60
Total production of water in the design life (lit)	= 1.86×10×365 = 6789	= 1.60×10×365 = 5840
Production cost of water (Tk./lit)	= 1400/6789 = 0.20	= 1500/5840 = 0.26

Table 4: Summary of cost analysis of different type of solar stills

Type of Still	Design life (years)	Initial cost (Tk.)	Total cost in design life (Tk.)	Average production of water (lit/m^2-day)	Production cost of water (Tk./lit)
BSS1	10	1500	1500	3.97	0.26
BSS2	10	1400	1400	2.97	0.20
TSS (not described here)	2	80	110	2.43	0.39

BSS1 = Conventional basin type still, BSS2 = Stepped basin type still and TSS = Tubular solar still

2. CONCLUSIONS

It is concluded that the production rate of water is higher for basin type solar stills. Though the construction cost of TSS is low but the water production cost is higher than BSS. Among basin type stills, water production cost in stepped type still is lower than conventional type. Considering economic point of view, it is clear that stepped BSS is cost effective than other type of stills. The production rate of a solar still is mainly depends on the intensity of solar radiation, so the production rate will be higher in clear weather than rainy and foggy weather when sun light is absent. As the construction, operation and maintenance of stepped BSS is easy and can be constructed using locally available materials, so for drinking and other purposes in remote, coastal and arid areas or in an emergency it could be a better solution to meet the small scale fresh water demand.

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Transportation Engineering

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EXPERIMENTAL BEHAVIOR OF BITUMINOUS MIXES WITH WASTE CONCRETE AGGREGATES

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ABSTRACT

Waste concrete originates from construction and demolition debris that has been crushed to produce Waste Concrete Aggregate (WCA). The lightweight, porous cement mortar attached to WCA causes crushed concrete aggregates to have a lower specific gravity and higher water absorption than comparatively sized fresh aggregates. Initially WCA were used as land filling materials and after many research works it is now being utilized as road base material and in non-structural concrete applications. The demand of stone aggregates for road construction can be reduced by recycling of WCA as coarse aggregate. Due to the booming of real estate business, generation of WCA are increasing day by day in Bangladesh. In most of the old buildings, brick chips and curshed gravels were used as coarse aggregate of concrete. Therefore, this investigation was carried out to check the possibility of using indigenous WCA in bituminous concrete. Test procedures specified by AASHTO standards were followed for the determination of stability, stiffness, compressive strength and stripping values of bituminous mixes with WCA used in this investigation. From experimental results, it was found that most of the characteristic test properties were within the acceptable recommended limits for the WCA sample used in this study.

Keywords: WCA, Bituminous mix, Stability, Stiffness, Compressive strength and Stripping.

1. INTRODUCTION

Generally, aggregates are collected by cutting mountains or breaking river gravels or boulders, or by breaking clay bricks. A significant amount of natural resource can be saved if the WCA is recycled for new constructions. In addition to the saving of natural resources, recycling of WCA will also provide other benefits, such as creation of additional business opportunities, saving cost of disposal, saving money for local government and other purchaser, helping local government to meet the goal of reducing disposal, etc. Demolished concrete structures are proven to be a good source of construction material. Torring and Lauritzen (2002) reported that at present, the amount of global demolished concrete is estimated at two to three billion tons. WCA is produced by the process of crushing demolished concrete elements. WCA differ from fresh aggregates due to the amount of cement paste remaining on the surface of the original natural aggregates after the process of recycling. This highly porous cement paste contributes to the lower particle density and higher porosity, variation in the quality of the WCA and the higher water absorption. WCA were initially used as land filling material and after many research works (McGrath, 2001 and Huang et al., 2002) it is now being utilized as road sub-base material and in non-structural concrete applications. Yanagibashi et al. (2002) noted that sixty to seventy percent of demolished concrete is used as sub-base aggregates for road construction. Public Works Technical Bulletin (PWTB, 2004) reported that about eighty percent of all cement concrete debris that is recycled is used as road base due to its availability, low transportation cost, and good physical properties. Saeed (2008) reported that recycled concrete aggregate (RCA) has a proven history of use as base, sub-base, land filling and drainage layers within the pavement structure; construction and performance have been excellent.

In recent years the use of WCA for road construction has grown spectacularly in many countries of the world. Due to scarcity of natural stone aggregates, WCA are used for the construction of base and sub-base courses of flexible pavements. Attempts are being made recently in different countries to construct 'Dense Bituminous Macadam' (DBM) using WCA. In Bangladesh, the volume of demolished concrete is increasing due to the deterioration of concrete structures as well as the replacement of many low-rise buildings by relatively high-rise buildings due to the booming of real estate business. Most of these demolished concretes were prepared using crushed gravels and brick chips as coarse aggregates. Studies related to the recycling of demolished concrete are generally found for basalt aggregates made concrete. This paper presents some of the findings of an investigation of WCA available in Bangladesh on properties of bituminous mixes. Bituminous mix used as dense bituminous macadam base course or surface course of flexible pavement in which the aggregates are bounded together by bituminous material. Natural stones are generally used as coarse aggregates in bituminous mixes. Recycling of WCA in bituminous pavement can offer engineering, economic and environmental benefits.

The quality and durability of a bituminous mix is influenced by many factors including types of aggregate in mix, compacting energies and presence of water in bituminous mix. For road constructions, locally available lightweight aggregates having angular particles, rough texture and having affinity for bitumen can be used as alternate of fresh stone aggregates. The influence of the amount of compaction energy and absorbed water upon the quality of bituminous mix contains lightweight aggregates has been subject of active interest in many countries. The characteristics of WCA from crushed gravels and brick chips are like lightweight aggregates. Due to heavy compacting energy, porous cement mortar attached onto WCA may be crushed which reduces the stability and stiffness of bituminous mix. The presence of water in bituminous mix can result in problems within the bond between the aggregates and the bitumen. The progressive functional deterioration of a pavement mixture by loss of the adhesive bond between the bitumen binder and the aggregate surface from the action of water, is referred to as stripping, where the bitumen binder is 'stripped' from the aggregate. Stripping is considered a great economic loss and engineering failure although the mix design was proper.

Hoque (1976) reported that brick-aggregate asphaltic concrete mixes are suitable for using in the surface courses of asphalt concrete pavements from the stand points of stability, stiffness and deformation characteristics. Laboratory studies performed by Petrarca et al. (1989) to determine the feasibility of using recycled concrete aggregates (RCA) in asphaltic concrete have indicated that RCA mixes had 1.5 to 2.0 times the stability of crushed stone-natural sand mixes. Fowler et al. (2003) reported that the use of crushed concrete materials in asphalt mixes resulted in a slight increase of the optimum asphalt content but higher strength and stability. Paranavithana and Mohajeani (2006) investigated the effect of RCA on the properties of asphalt concrete and the results found in this study are encouraging. Biswas (2011) reported that brick aggregates and RCA from concrete of brick aggregates can be used in the surface course of bituminous pavement for light to medium traffic roads at places where stone aggregates are costly and subjected to lower soaking periods.

2. MATERIAL PROPERTIES

A bituminous mix is normally composed of aggregates and bitumen. Aggregates are generally divided into coarse, fine and filler fractions according to the size of individual particles. Aggregates have to bear load stresses occurring in the roads and have to resist wear due to abrasive action of traffic. Bitumen content in mix ensure proper bond together with durable pavement under suitable compaction. Thus the properties of aggregates and bitumen are of considerable significance for proper bituminous mix design.

2.1 Materials

WCA used in the present investigation were collected from crushed cement concrete cylinders. Cement concrete cylinder (1:2:4) were casting in laboratory and were crushed after 28 days curing. Crushed cylinders in which crushed gravels and brick chips were coarse aggregates separately were sieved to different fractions. Particles retained on 2.36 mm sieve were taken as coarse aggregates from WCA and coarse aggregates from fresh crushed gravel for comparison in the present investigation. According to the recommendation of the Asphalt Institute (1984), particles retained on 2.36 mm sieve were regarded as coarse aggregate. Fine aggregate portion of the aggregate blend (2.36 mm to 0.075 mm sieve) was taken from coarse sand. Non-plastic sand finer than 0.075 mm sieve was collected from fine sand of Padma River and used as a mineral filler in the mix. Properties of mineral matter were determined according to the test procedures specified by AASHTO standards and results are summarized in Table 1.

Bituminous binders are the product of the petroleum crude oil refining process. The bitumen binder produced during the refining process can be modified in many different ways to meet the specifying agency's specifications. These different modification methods will change the moisture sensitivity of the binder. Petersen (2002) reported that if the crude oil is not desalted after the caustic treatment, these salts will remain in the crude oil which have caused stripping in asphalt mixtures. The properties such as stability, void content and flow of a bituminous mix are dependent on the type and amount of bitumen.

The binder material used for this investigation was of 80/100 penetration grade bitumen and collected from Eastern Refinery, Chittagong. Routine tests as per AASHTO test standards were performed on the bitumen sample and get the following properties: Specific gravity, 1.02; Penetration value (0.1mm), 98; Ductility value, 100+ cm; Solubility value, 99.8%; Flash point, 290^0C and Fire point, 310^0C.

Table 1: Properties of mineral matter

Properties	Coarse aggregates			Fine aggregate	Mineral filler
	Fresh	WCA from concrete of			
	Crushed gravels	Crushed gravels	Brick chips	Coarse sand	Fine sand
Bulk specific gravity	2.62	2.27	1.94	2.46	---
Apparent specific gravity	2.73	2.66	2.44	2.66	2.63
Absorption of water, percent	1.45	6.38	10.56	3.10	---
L. A. abrasion (Grade A), percent	26	40	42	---	---
Soundness (MgSO4, 5 cycle), percent	5	19	22	---	---
Ten percent fines value, kN	190	90	80	---	---

2.2 Mix types

One of the main objectives of this investigation was to make a comparative study of WCA bituminous mixes with bituminous mix from fresh aggregates. Three types of bituminous mixes were studied and these were designated as I, II and III. Mix I consisted of fresh crushed gravels, coarse sand and fine sand as filler. Mix II contained WCA from crushed gravels, coarse sand and fine sand as filler. Mix III is composed of WCA from brick chips, coarse sand and fine sand as filler. Sobhan (2011) studied the effect of gradation and compaction effort on the properties of bituminous mixes with waste concrete aggregates and suggested that aggregate gradation for dense bituminous concrete (25 mm nominal size) recommended by the Asphalt Institute is suitable for bituminous mixes with WCA. The particle size distribution of this gradation is shown in Figure 1. This gradation was used for the preparation of bituminous mixes with above three mix types. Coarse sand (2.36 mm to 0.075 mm sieve) and non-plastic sand from fine sand (filler passing 0.075 mm sieve) were mixed with selected coarse aggregates to achieve the required aggregate combination.

In order to study the effect of binder content on compacted bituminous mixes containing WCA, bituminous specimens from Mix II were prepared with 7.0%, 7.5%, 8.0%, 8.5% and 9.0% bitumen contents and from Mix III were prepared with 8%, 9%, 10%, 11% and 12% bitumen contents respectively. Specimens from Mix I were also prepared with 5.5%, 6.0%, 6.5%, 7.0% and 7.5% bitumen contents for comparison of mix properties.

3. INVESTIGATIONS AND TEST RESULTS

To investigate the behaviour of bituminous mixes with WCA, Marshall test specimens were prepared using Mix type II and III separately according to the selected aggregate grading and also with Mix type I for comparison. Marshall test specimens of 101.6mm diameter and 63.5 mm thick were prepared for medium traffic requires 50 blows per side of the specimen as per AASHTO T245-82 by varying bitumen content. The number of blows for the preparation of Marshall test specimen was selected corresponding to 690 kN/m^2 (100 psi) tire pressure. The heavy vehicles which on the roads of Bangladesh have tire pressures in the range of 415-485 kN/m^2 (60-70 psi). So the assumption of 690 kN/m^2 tire pressure seems to be quite safe and appropriate. At least 3 specimens were prepared for each bitumen contents and at least 5 bitumen contents were used with increments of 0.5 percent for fresh crushed gravels and WCA from concrete of crushed gravels and 1 percent for WCA from concrete of brick chips by weight of total mix. Brick aggregates absorbed more bitumen due to its higher porosity. So, the variation of mix properties with brick aggregate for 0.5 percent increments of bitumen were not mentionable that's why 1 percent increment of bitumen was select for this mix. The bulk specific gravities of fresh compacted specimens were determined according to the test procedure specified by ASTM 2726 Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-dry Specimens. After determination of the bulk specific gravity, the specimens were then subjected to Marshall stability and flow tests as per AASHTO T245-82. Voids analysis was made for each series of test specimens after the completion of the stability and flow tests. For the determination of optimum bitumen content, the variations of bulk density, Marshall stability and air voids in total mix with bitumen contents were plotted and shown in Figure 2, Figure 3 and Figure 4 respectively.

Optimum bitumen content (OBC) was determined as follows: The bitumen contents at maximum density and at maximum stability were determined from Figures 2 and Figure 3 respectively. For bituminous concrete, bitumen content, at 4 percent (median of 3-5 percent range) voids in total mix were determined from Figure 4. The average of these three bitumen contents was taken as optimum bitumen content. At optimum bitumen content, the values of bulk density, Marshall stability, flow, percentage of air voids in total mix (V_a,%), percentage of voids in mineral aggregates (VMA,%), percentage of voids filled with bitumen (VFB,%), Marshall stiffness and bitumen required (kg per m^3 of mix) for three mix types are given in Table 2.

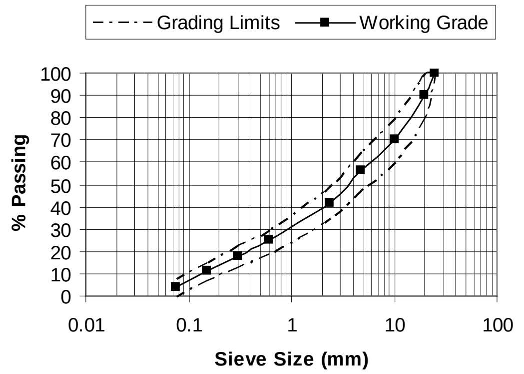

Figure 1: Particle size distribution for three mixes

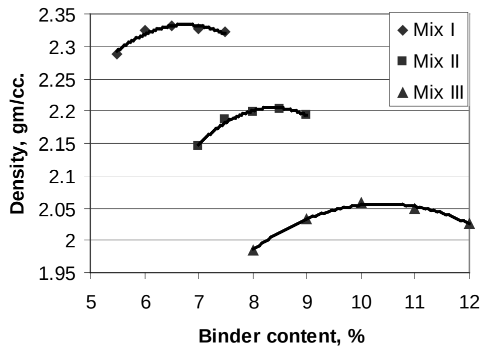

Figure 2: Relation between density and binder content

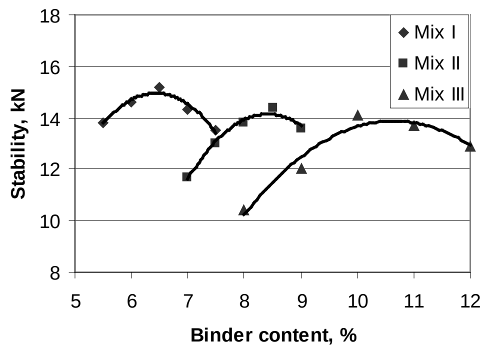

Figure 3: Relation between stability and binder content

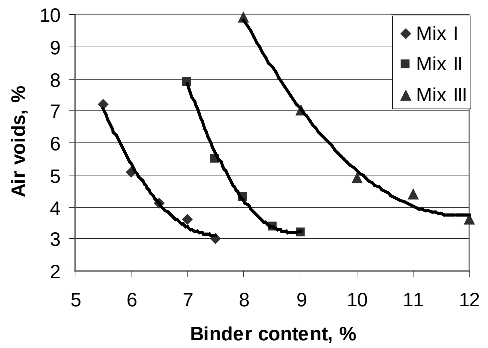

Figure 4: Relation between air voids and binder content

Table 2: Characteristics of compacted bituminous mixes for 50 blows

Properties	Bituminous mix types		
	I	II	III
OBC, % (of total mix)	6.5	8.5	10.5
Bulk density, gm/cc	2.331	2.204	2.059
Marshall stability, kN	15.0	14.3	13.6
Flow (0.25 mm)	13	14.8	14.3
V_a, %	4.1	3.4	4.5
VMA, %	14.8	14.3	15.0
VFB, %	72	76	73
Marshall stiffness, kN/mm	4.6	3.9	3.8
Bitumen, kg per m^3 of mix	150	190	220

Compaction of bituminous mix is a mechanical process by which the aggregates are constrained to be packed more closely together by reducing the air voids. Not all mix design methods use the same compaction effort for mixtures. For instance, a mixture designed using the Marshall mix design procedure for heavy traffic requires 75 blows per side of the specimen with the Marshall compaction hammer. The same mix designed for medium traffic and light traffic requires 50 blows per side and 35 blows per side respectively.

To study the effect of compaction efforts on the behaviour of bituminous mixes with WCA, specimens were prepared for each mix using 35, 50 and 75 number of blows for light traffic, medium traffic and heavy traffic respectively. Bitumen content was kept constant at their OBC calculated from Marshall mix design for 50 blows to give higher stability. The variations of bulk density and Marshall stability with different number of blows for three mix types are shown in Figure 5 and Figure 6 respectively for comparison. Marshall stiffness of compacted specimens with three mixes for three compaction efforts are given in Table 3.

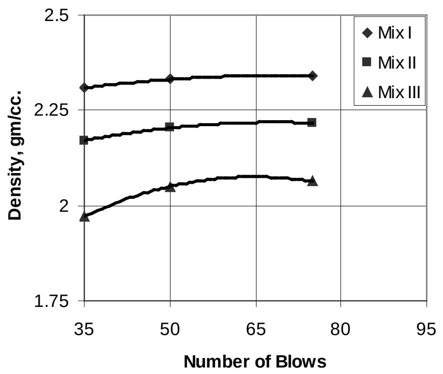

Figure 5: Relation between density and No. of blows

Figure 6: Relation between stability and No. of blows

Table 3: Marshall stiffness of compacted bituminous mixes for different compaction efforts

Mix type	I			II			III		
Bitumen content, percent	6.5			8.5			10.5		
Compaction, (No. of blows)	35	50	75	35	50	75	35	50	75
Marshall stability, kN	10.8	15.2	16.8	10.2	14.4	14.7	10.1	14.0	14.3
Flow, (0.25 mm)	13.3	13.0	13.5	15.2	14.8	16.2	14.4	14.1	15.6
Marshall stiffness, kN/mm	3.2	4.7	5.0	2.7	3.9	3.6	2.8	4.0	3.7

The moisture-induced damage within hot-mixed asphalt pavement is a national issue that decreases the lifespan of the highway of a country. Highly porous cement paste and dusty surface are the main characteristics of WCA. The National Asphalt Pavement Association (NAPA, 1992) recognized that dust coating of the aggregate can inhibit the adhesion of the asphalt binder, thereby allowing water to penetrate to the aggregate surface. This is a problem most associated with WCA produced by crushing demolished concrete.

In order to study the effects of water on bituminous mixes containing WCA, specimens of 100 mm diameter and approximately the same height of 95 mm were prepared and tested according to the test procedure specified by AASHTO T167-84. Bitumen contents were kept constant at OBC to give higher strength. Axial compression load on the specimens were applied at a uniform rate of vertical deformation of 1.3mm per minute per 25 mm of height without lateral support. Test procedure specified by AASHTO T165-86 has been followed for the determination of the compressive strengths and the effects of water on the compressive strength of bituminous mixes with WCA. The numerical index of resistance of bituminous mixtures to the detrimental effect of water is called index of retained strength. The index of retained strength is determined dividing the compressive strength of immersed (24 hours) specimen by the compressive strength of dry specimen and multiply the value by 100. Index of retained strength for three mix types are recorded in Table 4.

Table 4: Index of retained strength for three mixes

Mix type	I	II	III
Compressive strength of immersed (24 hrs.) specimen, kN/m^2	1724	1551	1528
Compressive strength of dry specimen, kN/m^2	1935	1868	1850
Index of retained strength, percent	89.1	83.0	82.6

4. ILLUSTRATIONS

Table 1 presents the specific gravities, water absorptions and strength properties of coarse aggregates used in this study. WCA was found to have lower specific gravity and higher absorption values than typical crushed gravels. Test results indicate that WCA are weak than fresh aggregates due to the easy separation of the mortar attached to WCA under compression, crushing and abrasion. Hansen (1992) reported that Los Angeles abrasion results dependent on the strength of the original concrete, where stronger breaks up less than weaker concrete.

As seen in Figures 2 and Figure 3, the densities and stabilities of the bituminous compacted specimens increase initially with the increase of binder content, reach a maximum value and then decrease. With the increments of binder content, the batter compactions were done as a result the density and stability increased. For further increments of binder, the thickness of bitumen film increased as a result the density and stability decreased. These variations are similar for mixes contain WCA and fresh aggregate. Gallaway and Harper (1968) reported that cohesion of mixtures containing lightweight aggregate (high absorbent) as the coarse fraction generally increases with increasing bitumen content. They further noted that the density of specimens increases with increasing bitumen content, reaches a maximum value and then decreases. In this sense, WCA mixtures behave as fresh aggregate mixtures. It is also seen from Table 2 that the required bitumen (kg/m^3 of mix) is 1.25 times and 1.45 times more in bituminous mixes with WCA from crushed gravels and brick chips respectively than mix contain fresh crushed gravel aggregates due to highly porous mortar attached onto WCA. Figure 4 shows the variation of air voids with binder content for Mix I, Mix II and Mix III. Air voids decreases with the increase of binder content and also with the level of compaction. For same binder content, air voids of mixes containing WCA are considerably higher than the mix containing fresh aggregate due to the highly porous cement paste attached on the surface of fresh aggregates. It is seen from Table 2 that the percentage of air voids at OBC for three mixes are satisfy the limiting value (3 % to 5 %) recommended by the Asphalt Institute (1984). For the interpretation of Marshall test results, Lees (1983) considered the stiffness (the ratio of stability to flow) of the mix should not be less than 2.1 kN/mm (120lb/0.01″) for the design tire pressure of 100 psi. From Table 2 it is seen that the Marshall stiffness for bituminous mixes with WCA are above the required value 2.1 kN/mm.

Results shown in Figure 5 indicates that the density of the compacted specimens increases with the increase of compaction efforts because the percentage of air voids in the total mix decrease with the increase of compaction efforts. Similar variation takes place for mix contain WCA and for mix contain fresh aggregates. Figure 6 shows that the Marshall stability of the compacted specimens increases with the increase of compaction efforts. In highly compacted bituminous mixes, the interlocking of the aggregate particles and their frictional resistance to displacement is very high and as a result the stability increases with the increase of compaction efforts. Densities increases significantly with the increase of compaction efforts but there are slight increase in stability from 50 blows to 75 blows for mix contain WCA. Separation of cement mortar from the aggregate surface occurs due to higher compaction energy which decreases the interlocking of the aggregate particles and their frictional resistance to displacement Excess fine particles in the bituminous mix due to higher compaction energy also promote the rutting in the bituminous surface course at higher temperature. Results recorded in Table 3 shows that the Marshall stiffness for compaction efforts of 75 blows are less than that for compaction efforts of 50 blows in mix contains WCA.

The term stripping denotes the occurrence of adhesion failure or a weakening of the cohesive bonds within the asphalt aggregate system. Adhesion failure is affected by many factors including material characteristics, construction techniques and diversified environmental conditions. Four basic hypotheses about the mechanics of adhesion failure are displacement, film rupture, detachment and pore pressure theories. Adhesion of bituminous binder to aggregate is a surface phenomenon related to the physio-chemical properties of the two materials. The lack of adhesion in a bituminous mixture results from either improper joint formation or stripping of the binder from the aggregate in the presence of water. The detachment may be related to the absorption of water into the aggregate surface structure and cause a rise in the PH value. The pore water pressure build up in the pavement due to the dynamic effect of wheel load application is believed responsible for adhesion tension and interfacial tension failures in bituminous mixtures. Replacement of the bitumen by a thin film of water that may come from either outside or from within the aggregate while the bitumen coating remains intact. The characteristics of the interface are believed to be very important in the detachment process. The water reaching the interface becomes intimately associated with the lattice of the mineral surface. Index of retained strength is the measure to ascertain the detrimental effect of water in the bituminous mix. Results recorded in Table 4 shows that the index of retained strength of bituminous mixes with WCA satisfy the limiting value 75 % recommended by the Asphalt Institute (1981).

5. CONCLUSIONS

This paper has presented some of the experimental behaviour of bituminous mixes with waste concrete aggregates (WCA). Characteristics of bituminous mixes containing WCA and fresh aggregate as coarse aggregates were investigated. Although densities of the compacted bituminous mixes containing WCA were lower than that for the compacted bituminous mix containing fresh aggregates due to porous mortar attached onto WCA but their stability values were very close. However, the bitumen requirements for one cubic meter mix is 1.25 times and 1.45 times greater in bituminous mixes with WCA from crushed gravels and brick chips respectively than mix contain fresh crushed gravel aggregates due to highly porous mortar attached onto WCA. Bituminous mix with WCA, coarse sand and fine sand as filler satisfies all the requirements of Marshall design criteria for medium traffic condition recommended by the Asphalt Institute.

Marshall stiffness of bituminous mixes containing WCA for compaction efforts of 75 blows were less than that for 50 blows. For batter stiffness and to avoid rutting of bituminous surface course at higher temperatures, 50 blows compaction efforts can be adopted for the design of WCA bituminous mixes used in this study. Compressive strength of bituminous mix decreases due to the presence of water in the mixes. Stripping potential of bituminous mixes with WCA was high but index of retained strengths for 24 hours immersion were satisfactory. The experimental results found for unbound WCA, used in this study, were compared with those found for fresh crushed gravel aggregates. Due to the highly porous, low dense cement mortar attached onto WCA particles, the strength properties of WCA were relatively much lower and the water absorptions of WCA were much higher. The results found in this investigation are encouraging; however, further study is required to investigate the application of current mix design methods for bituminous mixtures containing WCA.

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EFFECTS OF WASTE POLYETHYLENE ON BITUMINOUS MIXES

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ABSTRACT

Waste polyethylene that has been used previously in domestic purposes are biologically non-degradable and posed an ominous environmental problem which led to severe environmental impact. But molten polyethylene has a binding property which can be reused with bitumen to reduce the cost of bituminous mix. At the same time the recycling of waste polyethylene save disposal sites. The bitumen binder can be modified in many different ways and these different modifications will change the properties of binder. waste polyethylene was used in bitumen with different percentage for the production of modified bitumen for the present investigation. The investigation carried out to observe the properties of modified bitumen and finally to check the use of waste polyethylene by adopting Marshall design methods of bituminous mixes according to the test procedure specified by AASHTO. Some of the measured properties of bituminous mix with modified bitumen used in this study were within the acceptable recommended limits. On the basis of experimental results of this investigation, it is concluded that the dense graded bituminous mixes with bitumen containing polyethylene up to 10% can be used for bituminous pavement construction in warmer region from the points of view of stability, stiffness and voids characteristics.

Keywords: *Waste polyethylene, modified bitumen, bituminous mix, recycling, stability.*

1. INTRODUCTION

The amount of waste polyethylene is increasing day by day as the availability of the polyethylene is enormous. They either get mixed with Municipal Solid Waste or thrown over land area. This increased waste create problems in the flow of drainage water, loss of fertility in agricultural land and polluting the environment. To encounter this trend, considerable effort is being put into recycling waste, turning it into re-usable by products. Waste polyethylene derived from household wastes, domestic and commercial uses. Waste polyethylene on heating molten at around 100°C to 260°C. Moreover, the molten polyethylene has a binding property. Hence, the molten polyethylene materials can be used as a binder and they can be mixed with binder like bitumen to enhance their binding property. Polyethylene is a commodity and inexpensive polymer being used in bitumen modification. Moreover, there is a large volume of recycled polyethylene (Jew et al.1986 & Little, 1993). This may be a good modifier for the bitumen, used for road construction. Many investigations have found that the strength of the paving mixes can be enhanced by use of a binder formed by modifying available bitumen with certain additives like organic polymer. Punith and Veeraragavan (2011) describes the procedure for modification of 80/100-paving grade bitumen cement using reclaimed polyethylene (PE) derived from low-density polyethylene carry bags collected from domestic waste. Kalantar et al. (2010) describes the properties of bituminous binder modified with waste polyethylene terephthalate. They also investigate the possibility of using Polyethylene Terephthalate as polymer additives in bituminous mix. Awwad and SHbeeb (2007) studied the use of polyethylene in hot Asphalt mixture. They concluded that modifying asphalt mixture with HDPE polyethylene enhances its properties far more than the improvements realized by utilizing LDPE polyethylene. Grinding the polyethylene provide better physical properties. Sabina et al. (2009) stated the performance evaluation of waste plastic/polymer modified bituminous concrete mixes. They also describe the comparative performance of bituminous concrete mixes containing plastic/polymer (PP) (8% and 15% by wt. of bitumen) with conventional bituminous concrete mixes (prepared with 60/70 penetration grade bitumen). The quality and durability of a bituminous mix is influenced by many factors including types of aggregate mix, compacting energies, water soaking periods and binder materials. The prime objectives of this investigation were to find out the properties of modified bitumen after mixing of polyethylene and to check the design criteria of bituminous mixes with modified bitumen. And also to check the modified bitumen is suitable or not for construction.

2. MATERIAL PROPERTIES

Materials used for this investigation were bitumen, waste polyethylene for the preparation of modified bitumen and mineral aggregate for the preparation of Marshall test specimen. The description of the materials are discussed below.

2.1 Mineral aggregate

In this investigation, crushed basalts retained on 2.36 mm sieve were regarded as coarse aggregate (The Asphalt Institute, 1984). Fine aggregate portion of the aggregate blend (passes 2.36 mm and retained on 0.075 mm sieve) was taken from coarse sand. Non-plastic sand finer than 0.075 mm sieve was used as mineral filler. Properties of mineral matter were determined according to the test procedure specified by American Association of State Highway and Transportation Officials (AASHTO) and results are given in Table 1.

Table 1: Properties of mineral matter

Properties	Stone chips (Coarse aggregate)	Coarse sand (Fine Aggregate)	Non-plastic sand (Mineral Filler)
Bulk specific gravity	2.79	2.46	…
Apparent specific gravity	2.86	2.66	2.63
Water absorption, %	1.45	3.10	…
Loss Angeles Abrasion , %	30	…	…

2.2 Bitumen

The bitumen used for this investigation was of 80-100 penetration grade bitumen and collected from Eastern Refinery, Bangladesh. Routine test as per AASHTO were performed on the bitumen samples used in this study. Test results of bitumen are presented in Table 2.

Table 2: Properties of bitumen

Properties	AASHTO designation	Bitumen
Specific gravity	T229	1.0225
Penetration (0.1 mm)	T49	83
Ductility (cm)	T51	100^+
Solubility (%)	T44	97.93
Softening point(°C)	T53	52.5
Flash point(°C)	T48	295
Fire point (°C)	T48	305

2.3 Modified bitumen

Modified bitumen refers the combination of bitumen and modifier. In this investigation waste polyethylene collected from domestic wastes was used as modifier. Polyethylene is semi-crystalline materials with excellent chemical resistance, good fatigue and wear resistance and molten polyethylene has a binding properties. Polyethylene are biologically non-degradable and poses a severe effect on environmental problem. To overcome this problem polyethylene used as a modifier in this study. The waste polyethylene was cleaned properly and then shredded to 2 to 3 mm size of sieve which was used as recycled polyethylene (Sabina et al. 2009). The polyethylene was heated till it fully liquefied and was in a state to dissolve polyethylene in bitumen. Waste polyethylene were gradually mixed in bitumen The modified bitumen samples were designated as $MB_{2.5}$, MB_5, $MB_{7.5}$, MB_{10}, $MB_{12.5}$, MB_{15}, $MB_{17.5}$ and MB_{20}. Where, MB indicates that the modified bitumen and the suffix indicate that the percentage of polyethylene in modified bitumen. The specific gravity and melting point of the waste polyethylene were determined (ASTM D70 and ASTM D3418) and these values were 0.945 and 115°C respectively.

3. TEST RESULTS

To investigate the effect of waste polyethylene, variation of properties of modified bitumen with respect to percentage of waste polyethylene are shown in Figures 1, 2, 3, 4, 5, and 6.

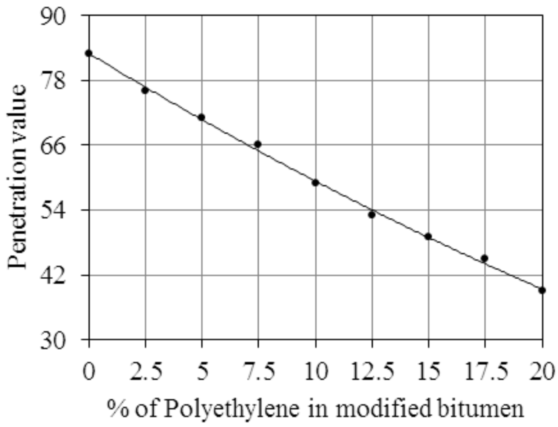

Figure 1: Relationship between penetration value and % of polyethylene in modified bitumen

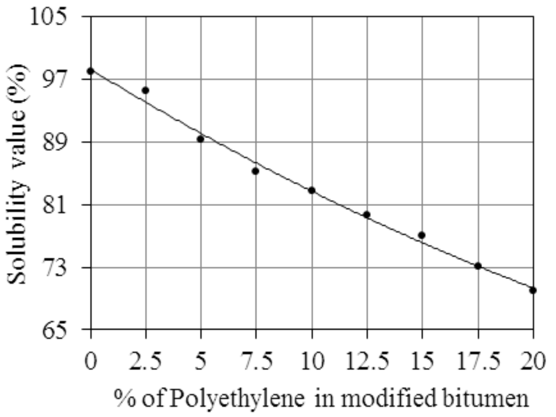

Figure 4: Relationship between solubility value and % of polyethylene in modified bitumen

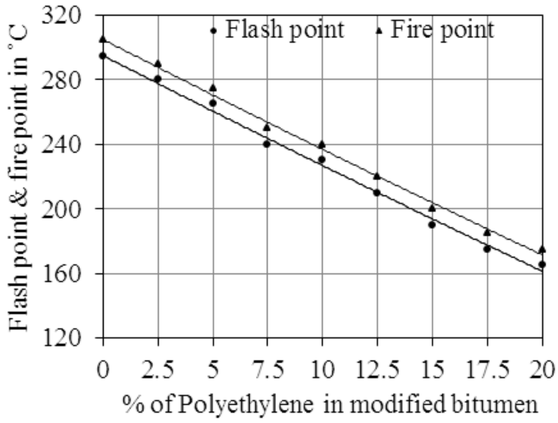

Figure 2: Relationship between flash point & fire point and % of polyethylene in modified bitumen

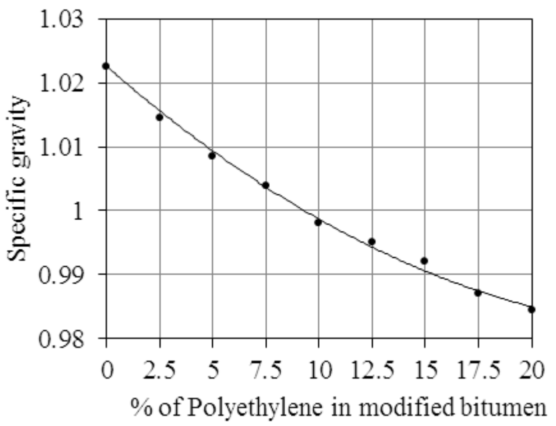

Figure 5: Relationship specific gravity and % of polyethylene in modified bitumen

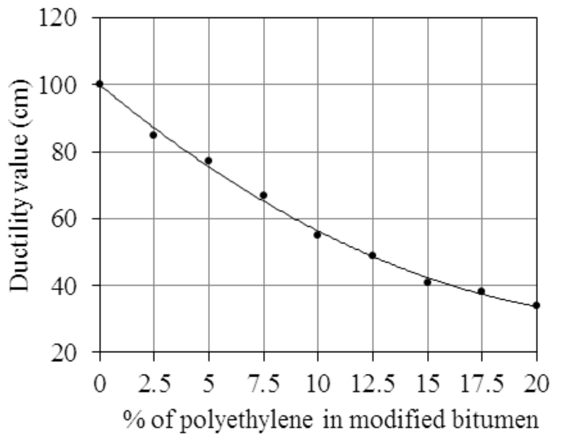

Figure 3: Relationship between ductility value and % of polyethylene in modified bitumen

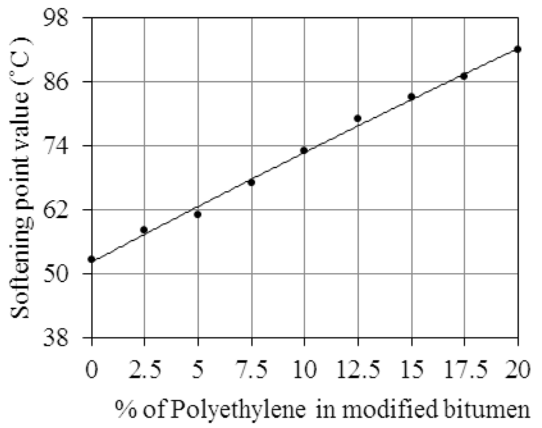

Figure 6: Relationship between softening point value and % of polyethylene in modified bitumen

To investigate the effects of waste polyethylene on bituminous mixes, Marshall test specimens of 101.6 mm diameter and 63.5 mm thick were prepared for medium traffic requires 50 blows per sides and tested as per AASHTO T245, 1982. The selected gradation used in this study according to The Asphalt Institute (1984) shown in Figure 7. Initially for determination of Optimum Bitumen Content (OBC), Marshall test specimens were prepared by fresh bitumen and then subjected to specific gravity, stability and flow test as per Marshall mix design procedure specified by AASHTO and the variation of unit weight, stability and air voids in total mix with bitumen content were plotted and shown in Figures 8, 9 and 10 respectively. OBC was calculated from the average of bitumen content at maximum unit weight, bitumen content at maximum stability and bitumen content for 4% air voids. Calculated optimum bitumen content was 5.4%. This bitumen content (5.4%) was kept constant for preparing the further specimens, only changing the percentage of polyethylene content. Marshall test specimens were prepared with OBC 5.4% for $MB_{2.5}$, MB_5, $MB_{7.5}$, MB_{10}, $MB_{12.5}$, MB_{15}, $MB_{17.5}$ and MB_{20} respectively. After determination of the bulk specific gravity, these specimens were subjected to Marshall stability and flow tests as per AASHTO T245-82. Voids analyses were made for each series of test specimens. The variations of unit weight, stability, flow, air voids in total mix, Voids in Mineral Aggregate (VMA) and Void Filled with Bitumen (VFB) with percentage of polyethylene in modified bitumen were plotted and shows in Figures 11, 12, 13, 14, 15 and 16 respectively.

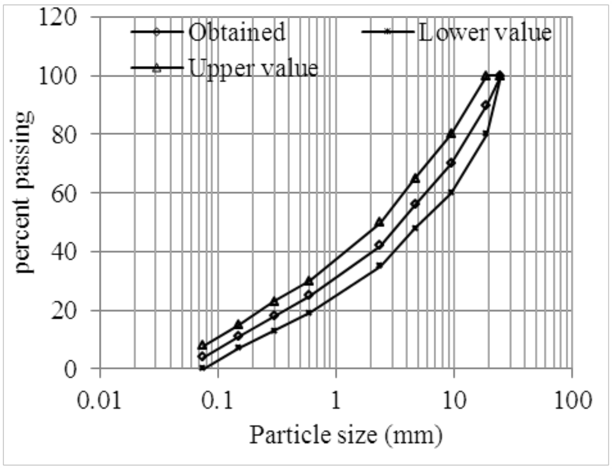

Figure 7: Grain size distribution of aggregate gradation

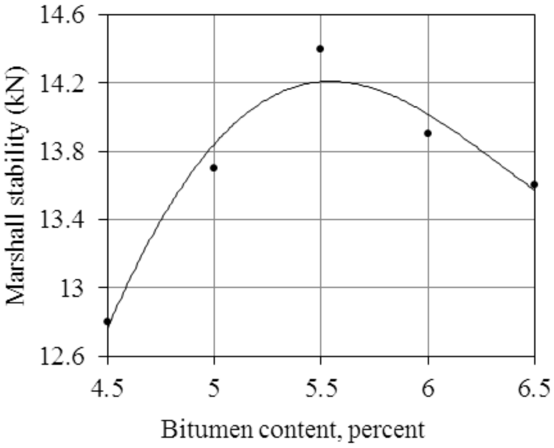

Figure 9: Relationships between Marshall stability and bitumen content

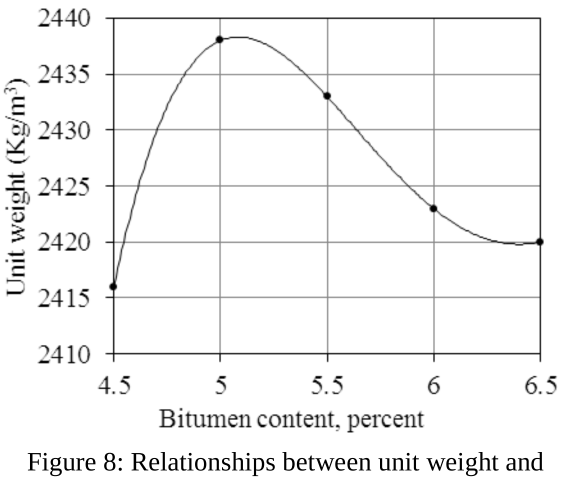

Figure 8: Relationships between unit weight and bitumen content

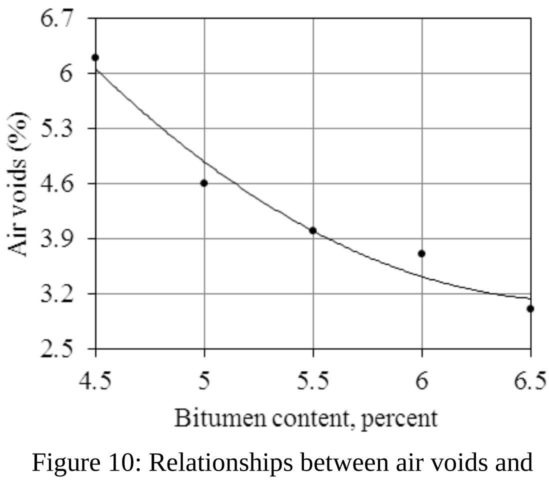

Figure 10: Relationships between air voids and bitumen content

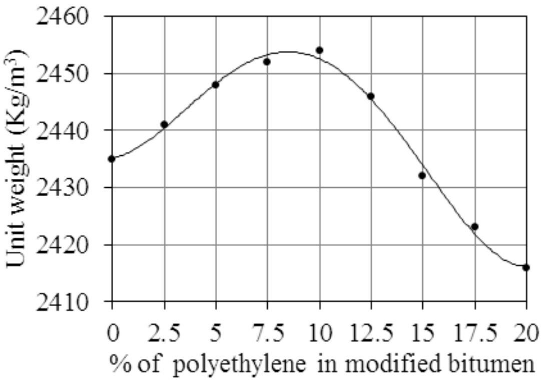

Figure 11: Relationships between unit weight and % of polyethylene in modified bitumen

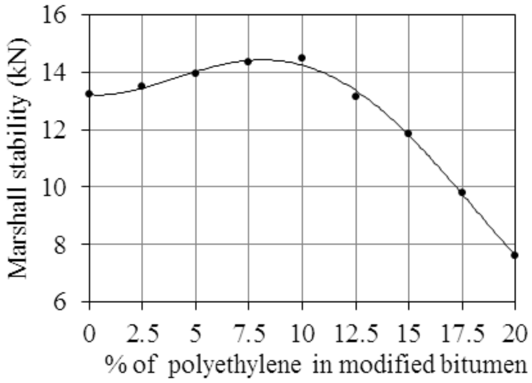

Figure 12: Relationships between Marshall stability and % of polyethylene in modified bitumen

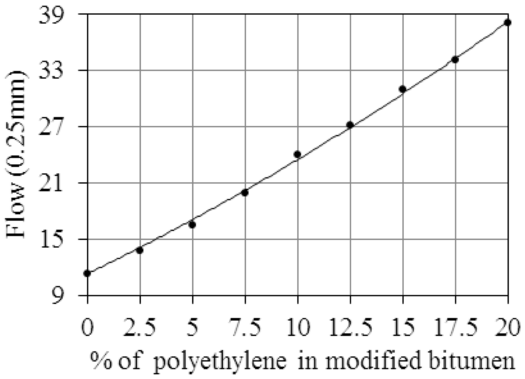

Figure 13: Relationships between flow value and % of polyethylene in modified bitumen

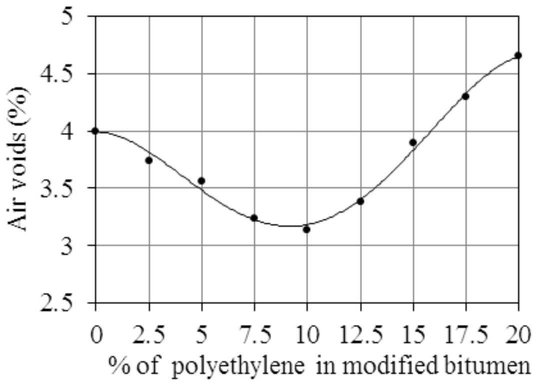

Figure 14: Relationships between air voids and % of polyethylene in modified bitumen

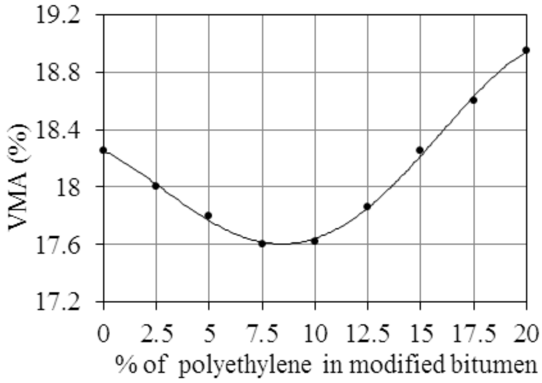

Figure 15: Relationships between %VMA and % of polyethylene in modified bitumen

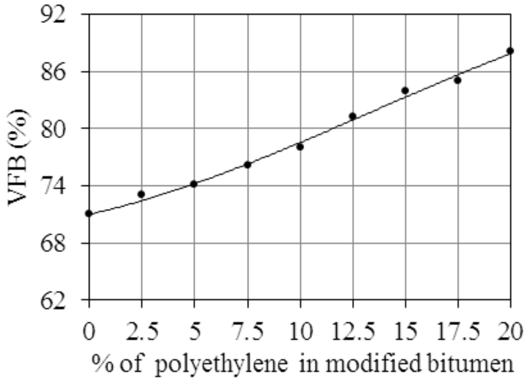

Figure 16: Relationships between VFB (%) and % of polyethylene in modified bitumen

4. ANALYSIS AND DISCUSSION

4.1 Effects of polyethylene on properties of bitumen

Results shown in Figures 1, 2, 3, 4 and 5 indicates that penetration value, flash & fire point, ductility value, solubility value and specific gravity decreases with the increase of percentage of polyethylene in modified bitumen due to lower specific gravity and melting temperature of waste polyethylene than that of pure bitumen. On the other hand the softening point value increases (Figure 6) with the increase of waste polyethylene in modified bitumen due to higher elastic property of polyethylene. Lower penetration value, lower ductility value and higher softening point indicates that the modified bitumen can be used in the warmer region.

4.2 Effects of polyethylene on bituminous mixes

Result shown in Figures 11 and 12 indicates that, the unit wt. and stabilities of the compacted specimens increase initially with the increase of polyethylene in modified bitumen, reach a maximum value and then decrease. With the increment of polyethylene in modified bitumen, the better compactions were done as a result the unit wt. and stability increased. For further increment of polyethylene in modified bitumen, the waste polyethylene has a tendency to segregate from hot bitumen and this segregate polyethylene increase the volume of specimens as a result unit wt. of the mix decreases with these excess segregated polyethylene. This segregated polyethylene acts as a emulsion in the mix which reduced the stabiltiy.

Figure 13 indicates that, the with the increment of polyethylene in bituminous mix the flow value increases due to emulsifying action of polyethylene. For interpretation of Marshall test results, Lees (1983) considered the stiffness (the ratio of stability to flow) of the mix which can be related to tyre pressure. In order to prevent permanent deformation of the mix under high stress, the Marshall stiffness should not be less than 2.1 kN/mm (120 lb/0.01″) for the design tyre pressure of 100 psi. It is seen that the Marshall stiffness are above required value 2.1 kN/mm up to 10% of polyethylene in bitumen. The air voids record of the mix with various percentage of polyethylene in modified bitumen reported in Figure 14 shows that the percentage of air voids in the total mix initially decreases with the increase in percentage of polyethylene in modified bitumen content. But for further increase of polyethylene in modified bitumen the percentage of air voids increases due to internal air voids in excess segregated polyethylene. It is seen that the percentage of air voids in bituminous mix for all modified bitumen satisfies the limiting values (3 to 5%) recommended by The Asphalt Institute, 1984.

Figure 15 shows that percentage of %VMA initially decreases with the increase of polyethylene in modified bitumen, reach a minimum value and then increase. With the increment of polyethylene in modified bitumen the void in mineral aggregate decrease but after 10% of polyethylene in bitumen causes the segregation of modified bitumen from mix so that %VMA increased. On the other hand percentage of %VFB increases with increase of polyethylene in modified bitumen shown in Figure 16. The %VFB in bituminous mix up to 10% polyethylene in bitumen satisfied the limiting value (65 to 78%) recommended by The Asphalt Institute, 1984.

5. CONCLUSIONS

On the basis of experimental results of this investigation, the following conclusions are drawn:

1. Specific gravity of modified bitumen decreases but stiff property gains with the increase of waste polyethylene in bitumen.
2. The effect of modified bitumen on the behavior of bituminous mix is reasonably good from the considerations of Marshall test properties.
3. The dense graded bituminous mixes with modified bitumen containing polyethylene up to 10% can be used for the construction of bituminous roads in warmer region from the stand points of stability, stiffness and voids characteristics.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECT OF WASTE CONCRETE AGGREGATES ON THE COMPRESSIVE STRENGTH OF BITUMINOUS MIXES

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ABSTRACT

Waste concrete aggregates (WCA) refers to aggregates that have been used previously in civil engineering constructions. WCA differ from normal aggregates due to the cement paste attached to the surface of the normal aggregates after the process of recycling. Iinitially WCA were used as a fill material and after many research works it is now being utilized as granular base and sub-base in road construction and in non-structural concrete applications. Attempts are being made recently to construct bituminous surfacings useing WCA. The compressive strength of bituminous mix reduced by the loss of adhesive bonds resulting from the action of water, called stripping. Porous and dusty surface of WCA loves water which is the culprit causing stripping. This paper presents some of the results of an investigation on effect of WCA on the compressive strength of bituminous mixes for different soaking periods. It was found that the index of retained strength of compacted bituminous mix specimens containing WCA for different soaking periods were relatively lower compared with that values found for similar specimens contain normal crushed gravel aggregates. Some of these experimental values were above the acceptable recommended limit 75 percent for the WCA sample used in this study.

Keywords: *WCA, Bituminous mix, Compressive strength, Soaking period, Stripping, Index of retained strength.*

1. INTRODUCTION

WCA differ from normal aggregates due to the cement paste attached to the surface of the original aggregates after the process of recycling. This highly porous cement paste and other contaminations contribute to the lower particle density and higher porosity, variation in the quality of the WCA and the higher water absorption. WCA were initially used as a fill material and after many research works (McGrath, 2001; Mroueh & Wahlstram, 2002) it is now being utilized as road sub-base material, and in non-structural concrete applications. In recent years the use of WCA for road construction has grown spectacularly in many countries of the world. Due to scarcity of natural stone aggregates, WCA are used for the construction of base and sub-base courses of flexible pavements. Paranavithana & Mohajerani, 2006 studied the effect of recycled concrete aggregates (RCA) on properties of asphalt concrete and reported that the properties of RCA asphalt concrete are encouraging. Attempts are being made recently in different countries to construct dense bituminous macadam and bituminous surfacing using WCA. The stability and durability of bituminous mix is influenced by many factors including types of aggregates and presence of water in mix. The asphalt cement adheres very well to most aggregates provided that they are clean and dry. However, the presence of water can result in problems with the bond between the aggregates and the asphalt cement. Failure of the bond already formed, resulting in the displacement of the asphalt cement from the aggregates, is referred to as stripping. When this occurs, the asphalt pavement will lose rock particles under traffic loads and the result is raveling. Stripping is considered a great economic loss and engineering failure in terms of proper mix design. Asphalt pavement performance is related to cohesive and adhesive bonding within the asphalt-aggregate system. Martin et al., 2003 mentioned that the moisture-induced damage within hot-mixed asphalt pavement is a national issue that decreases the lifespan of the nation's highways. Mejidzadeh & Stander, 1969 stated that the factors affecting the adhesion failure phenomenon are innumerable. They include the material characteristics, construction techniques and diversified environmental conditions. Terrel & Al-Swailmi, 1994 stated that when moisture is introduced and transported through the mixture and individual materials, deterioration may occur in the form of detachment, displacement film rupture, spontaneous emulsification, pore pressure or hydraulic scour. The National Asphalt Pavement Association (NAPA, 1992) recognizes that dust coating of the aggregate can inhibit the adhesion of the asphalt binder, thereby allowing water to penetrate to the aggregate surface. This is a problem most associated with crushed aggregates. Highly porous cement paste and dusty surface are the main characteristics of WCA.

There are primarily three prerequisites for the occurrence of stripping phenomena in the bituminous mixes are: presence of water in the pavement, repeated load application and the physio-chemical nature of the bitumen-aggregate system. To study the effect of water on the compressive strength of bituminous mixes, the index of retained strength (the percentage of the original strength that is retained after the immersion period) should be investigated. Hoque, 1987 reported that, the index of retained strength of recycled asphalt concrete for 24 hours immersion is 92 percent. From the investigated results Biswas, 2011 concluded that, the index of retained strength of bituminous mixes with RCA from brick chips satisfy the recommended limiting value (75%) up to 48 hours immersion. This paper presents some of the results of an investigation on the effect of WCA contain porous and dusty surface on the compressive strength of bituminous mixes for different soaking periods.

2. LABORATORY STUDY AND TEST RESULTS

A bituminous mix is normally composed of aggregates and bitumen. Aggregates are generally divided into coarse, fine and filler fractions according to the size of individual particles. Aggregates have to bear load stresses occurring in the roads and have to resist wear due to abrasive action of traffic. Bitumen content in mix ensure proper bond together with durable pavement under suitable compaction. Thus the properties of aggregates and bitumen are of considerable significance for proper bituminous mix design.

2.1 Materials Properties and Preparation

Coarse aggregates in most of the demolished concrete works in Bangladesh are crushed gravels and crushed bricks. For the present study, WCA were collected from the crushed cylinders in which crushed gravels and crushed bricks were coarse aggregates separately. Asphalt Institute, 1984 recommended that, particles retained on 2.36 mm sieve were regarded as coarse aggregate. Fractions of crushed cylinders retained on 2.36 mm sieve were considered as coarse aggregates called WCA for this study. Fine aggregates portion of the aggregate blend (passes 2.36 mm and retained on 0.075 mm sieve) were taken from coarse sand. Mineral fillers finer than 0.075 mm sieve were selected from fine sand. Properties of mineral aggregates were determined according to the test procedures specified by AASHTO and results are given in Table 1. Binder material was penetration grade bitumen that was collected from crude oil refinery. Routine tests as per AASHTO were performed on the bitumen sample used in this study and get the properties: Specific gravity, 1.02; Penetration value (0.1mm), 98; Ductility value, 100+ cm; Solubility value, 99.8%; Flash point, 290^0C and Fire point, 310^0C.

Table 1: Properties of mineral matter

Properties	Coarse aggregate			Fine aggregate	Mineral filler
	Normal	WCA from concrete of			
	Crushed gravels	Crushed gravels	Crushed bricks	Coarse sand	Fine sand
Bulk specific gravity	2.62	2.27	1.94	2.46	---
Apparent specific gravity	2.73	2.66	2.44	2.66	2.63
Absorption of water	1.45	6.38	10.56	3.10	---
L. A. Abrasion (Grade A), percent	26	40	42	---	---
Soundness (MgSO4, 5 cycle), percent	5	19	22	---	---
Ten percent fines value, kN	190	90	80	---	---

The following three bituminous mixes were prepared in this investigation.

Mix A: Coarse aggregates from normal crushed gravels + coarse sand + fine sand
Mix B: Coarse aggregates from WCA contain crushed gravels as coarse aggregates + coarse sand + fine sand
Mix C: Coarse aggregates from WCA contain crushed bricks as coarse aggregates + coarse sand + fine sand

At least 3 specimens were prepared for each bitumen contents and at least 5 bitumen contents were used with increments of 0.5 percent for Mix A and Mix B and 1 percent for Mix C. Sobhan, 2011 studied the effect of gradation and compactive effort on the properties of bituminous mixes with waste concrete aggregates and suggested that aggregate gradation for dense bituminous concrete (25 mm nominal size) recommended by the Asphalt Institute is suitable for bituminous mixes with WCA. The particle size distribution of this gradation is shown in Figure 1. This gradation was used for the preparation of bituminous mixes with above three mix types. Coarse sand (2.36 mm to 0.075 mm sieve) and fine sand (filler passing 0.075 mm sieve) were mixed with selected coarse aggregates to achieve the required aggregate combination.

2.2 Laboratory Investigation

To ascertain the optimum bitumen content for selected bituminous mixes, Marshall test specimens of 101.6mm diameter and 63.5 mm thick were prepared for medium traffic requires 50 blows per side of the specimen as per AASHTO T245-82 by varying bitumen content. The number of blows for the preparation of specimen was selected corresponding to 690 kN/m^2 (100 psi) tire pressure. The heavy vehicles which on the roads of Bangladesh have tire pressures in the range of 415-485 kN/m^2 (60-70 psi). So the assumption of 690 kN/m^2 tire pressure seems to be quite safe and appropriate. The bulk specific gravity of fresh compacted specimens were determined according to the test procedure specified by ASTM 2726 Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-dry Specimens. After determination of the bulk specific gravity, the specimens were then subjected to Marshall stability and flow tests as per AASHTO T245-82. Voids analysis were made for each series of test specimens after the completion of the stability and flow tests. The variations of bulk density, Marshall stability and air voids in total mix with bitumen contents were plotted and shown in Figure 2, 3 and 4 respectively. Optimum bitumen content is determined as follows: the bitumen contents at maximum density and at maximum stability are determined from Figures 2 and 3 respectively. For bituminous concrete, bitumen content, at 4 percent (median of 3-5 percent range) air voids in total mix are determined from Figure 4. The average of these three bitumen contents is taken as optimum bitumen content. Optimum bitumen content (OBC) for Mix A, B and C were found 6.5 percent 8.5 percent and 10.5 percent by weight of total mix respectively.

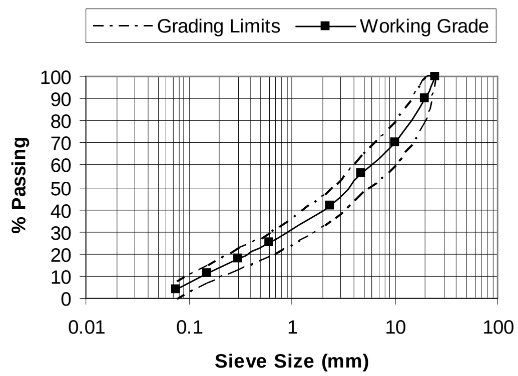

Figure 1: Particle size distribution for all mixes

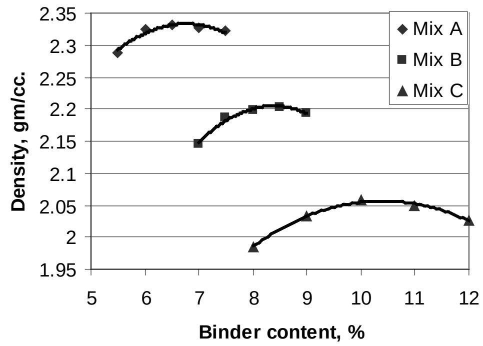

Figure 2: Relation between density and binder content

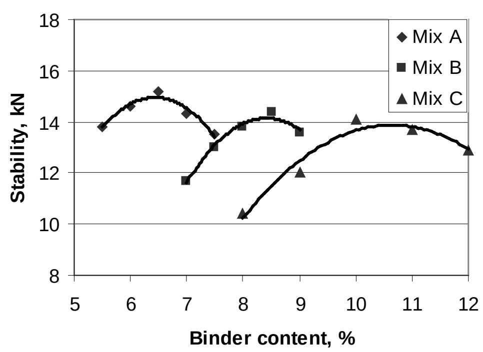

Figure 3: Relation between stability and binder content

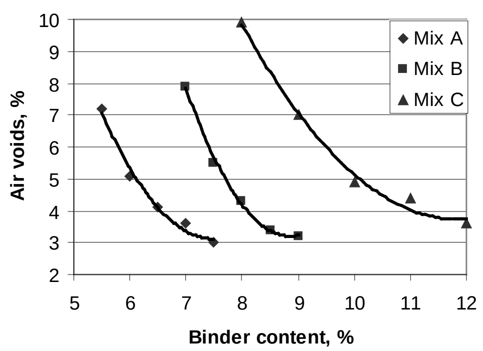

Figure 4: Relation between air voids and binder content

To investigate the reduction of compressive strength of bituminous mixes with WCA contains porous and dusty surface for different soakung periods, compacted bituminous mix specimens of 100 mm diameter and approximately of the same height of 95 mm were prepared according to the procedures specified by AASHTO T167-84. Specimens were prepared for Mix A, B and C. Bitumen content was kept constant at OBC to give higher strength. Procedures specified by AASHTO T165-86 have been followed to investigate the effect of water on the compressive strength of bituminous mixes. Test specimens for each mix types were grouped into five groups. The first group consisting of 3 specimens were used to determine the dry compressive strength. The second, third, fourth and fifth groups consisting of 3 specimens, which were used to determine the compressive strength after 24 hrs., 48 hrs., 72 hrs. and 96 hrs. soaked in water respectively. The numerical index of resistance of bituminous mixtures to the detrimental effect of water (called Index of retained strength) is the ratio of compressive strength of immersed specimen to the compressive strength of dry specimen and expressed as percentage. Index of retained strength of three mix types for different soaking periods are recorded in Table 2. For comparison, the relationships of index of retained strength with soaking periods for three mix types are shown in Figure 5.

Table 2: Index of retained strength of three mix types for different soaking periods

Mix type	Bulk density, gm/cc	Compressive strength, kN/m^2 (Dry specimen)	Index of Retained Strength (IRS), percent			
			24 hours	48 hours	72 hous	96 hours
Mix A	2.290	1894	87.4	79.1	75.2	70.5
Mix B	2.155	1867	83.0	75.2	69.6	67.2
Mix C	1.965	1855	84.2	73.4	67.3	64.8

Performance of a structure under load depends to a large degree on the stress-strain relationship of the material from which it is made, under the type of stress to which the material is subjected in the structure. Since bituminous mix is used mostly in compression, its compressive stress-strain curve is of primary interest. Such a curve is obtained by appropriate strain measurement in compression test of cylindrical bituminous specimen. Figure 6 shows a set of stress-strain curves obtained for three mix types in dry and soaked (24 hrs.) conditions under compressive load at a uniform rate of deformation of 1.3 mm per min. per 25 mm of height without lateral support.

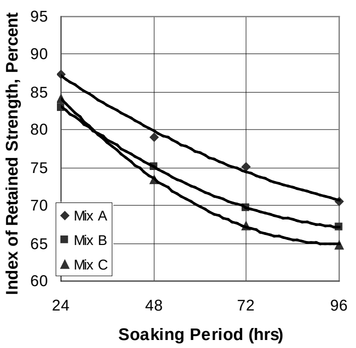

Figure 5: Relation between IRS and soaking period

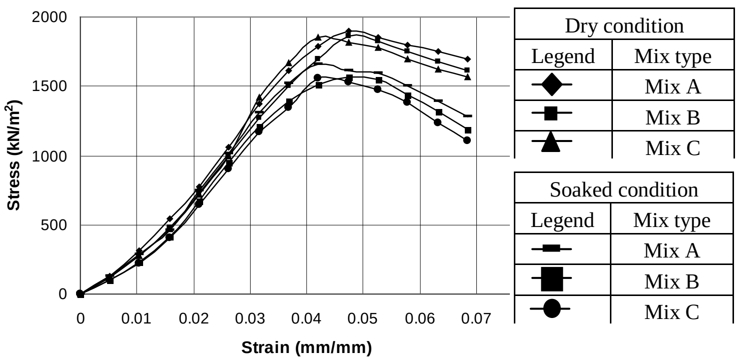

Figure 6: Stress- strain curves in dry and soaked condition

3. ANALYSIS AND DISCUSSIONS

Results shown in Figure 5 and Table 2 indicate that the index of retained strength decreases with the increase of soaking periods. With the increase of soaking periods, there are more opportunity for porous aggregates to absorb water and this absorbed water weakens the adhesive bonds between the bitumen binder and the aggregate surface rapidly. With the increase of soaking periods compressive strength reduction rate for WAC bituminous mixes is more than that for bituminous mix with normal crushed gravel aggregates.

There are many factors that affect the bond between asphalt and aggregate. Chemical bonding, as well as mechanical bonding, has been shown to be important to moisture damage resistance, but it is not only type of bonding that affects moisture damage. In the crushing process, dust is generated. The nature and extent of the dust can have a major effect on the moisture damage potential of the hot mix.

Dusty and dirty aggregate can promote moisture damage. Dust coating on the aggregate can prevent the asphalt binder from bonding directly to the surface of the aggregate. Logaraj, 2002 reported that asphalt by osmosis does allow water to pass through it. Consequently, water can get between the binder and the surface of the stone, stripping the asphalt from the aggregate surface. Aggregate that is coated with dust will slow the process of allowing the water to escape from the center of the aggregate when heated and dried in a hot-mix drum plant. In these cases, the asphalt is prevented from bonding well to the surface of the stone by the dust, and then the moisture in the aggregate weakens the bond as it escapes slowly from the mix. This problem is only significant when large amounts of dust are covering the aggregate. All processed aggregate will have some amount of dust, but only when it is caked on does the dust create a real problem with moisture damage. There are cases when even small amounts of dust can cause a problem. This happens when the dust is made up of small claylike particles. Clay acts as an emulsifier and will expand in the presence of water that can lift the asphalt off the surface of the aggregate. If this is combined with the action of traffic, the clay will emulsify the asphalt in the mix and cause severe stripping.

Usually high dust content reduces the void space in the combined aggregate. This void space, identified as the percentage of voids in the mineral aggregate or VMA, is the total volume in a combined aggregate that is available for air voids and asphalt binder. A mixture that is expected to perform adequately in service must balance the volume of air voids and the volume of asphalt binder for rutting resistance and durability. By contrast, a high dust content in a dense-graded mixture typically reduces the VMA of the mixture. At the same percentage of air voids, the dense-graded mixture will have a lower volume of asphalt binder (thinner film coating on the aggregates). This increases the potential of water penetrating the asphalt film.

Porous and dusty surface of WCA loves water which is the culprit causing stripping. Thinner film coating on the aggregates for lower volume of bitumen binder due to the dense gradation of aggregate selected for this study is also increases the potential of water penetrating the bituminous film.

The stress-strain curves as shown in Figure 6 have somewhat similar character for three mixes in dry and soaked conditions. They consist of an initial relatively straight elastic portion in which stress and strain are closely proportional, then begin to curve to the horizontal, reaching the maximum stress i.e. the compressive strength, at a strain of approximately 0.045 mm/mm, and finally show a descending branch. It is also seen that the residual strength (strength retain after failure) for soaked specimens with WCA are lower than that for soaked specimen contains normal crushed gravel aggregates. This occurs for relatively greater loss of adhesive bonds due to the highly porous cement paste and dust attached to the surface of the WCA. Moisture damage can manifest itself through various failure mechanisms. These include rutting, fatigue cracking, raveling, and potholes.

The study reveals that the index of retained strength (the percentage of the original strength that is retained after soaking) of compacted bituminous mix specimens containing WCA as coarse aggregate for soaking periods of 24 hrs, 48 hrs, 72 hrs and 96 hrs were relatively lower compared with that values found for similar specimens pepared with normal crushed gravel aggregates. Some of these measured values for the bituminous mix with WCA contain porous mortar and dusty surface used in this study were above the acceptable recommended limit 75 percent, specified by the Asphalt Institute, 1981.

4. CONCLUSIONS

One of the complex problems in the field of highway engineering, existing since bitumen paving technology came into existence is stripping. Stripping is considered a great economic loss and engineering failure in terms of proper mixture design. Moisture damage can be a significant problem that severely shortens a pavement's life. The causes of moisture damage are many and varied, ranging from basic materials to the design and construction process. Thus, it is critical that each aspect of the production process be managed properly.

Stripping potential of bituminous mixes contain WCA were found to be significantly higher and specimens with WCA tested were of greater variation in compressive strengths under soaked and dry conditions. The reduction of compressive strength of bituminous mixes with WCA and normal aggregates up to 24 hours immersion in water are almost near. Compressive strength of bituminous mix decreases with the increase of soaking periods. This variation is non-linear. Bituminous mix with WCA contains crushed gravel used in this study retain lower strength than the required 75 % when soaked for more than 48 hours but retained strength of bituminous mix with WCA from crushed bricks is less than 75 % before 48 hours soaking periods.

Relatively lower residual strength of bituminous mixes with WCA indicates that bituminous mixes with WCA tends to more brittle than that of bituminous mix contains normal crushed gravel aggregates due to presence of water in the mixes. Bituminous mixes with WCA used for the present study give satisfactory results when they are subjected to lower soaking periods. However, further study is required to investigate the other causes of moisture damage of bituminous mixes using WCA.

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EFFECTS OF BRICK CHIPS AS COARSE AGGREGATE IN BITUMINOUS MIXES

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ABSTRACT

The recycling of crushed brick has long been recognized to have the potential to conserve natural resources and to reduce energy used in production. In some countries it is a standard alternative for both construction and maintenance, practically where there is a shortage of natural aggregate. A few researches have been carried out to investigate their suitability and limited information is available about their performances. The objective of this paper is to determine the technical feasibility application of using the brick aggregate in road construction as an alternative for natural aggregate. This paper represents an experimental investigation of design characteristics and attempts were made to evaluate the brick chips as coarse aggregate in bituminous mix. Procedures specified by AASHTO were followed for the determination of properties of coarse aggregates and fine aggregates. Marshall mix design procedure specified by AASHTO T245, 1982 was followed in the design and testing of bituminous mixes. Some of the measured properties of bituminous mix with fresh and waste bricks used as a coarse aggregate in this study were within the acceptable recommended limits. Dense bituminous mixes using brick aggregate compacted with 50 blows is a feasible option from the standpoint of stability, stiffness, deformations and voids characteristics.

Keywords: *crushed brick aggregate, bituminous mixes, Marshall properties, Marshall design criteria, void characteristics.*

1. INTRODUCTION

In Bangladesh, natural stone are not locally available. Due to the scarcity of the natural stone and their high price urge the development of the locally available materials used as a coarse aggregate in bituminous mixes for the development of the road pavement. In recent years, the use of bricks for the road construction has grown spectacularly in many countries of the world. Especially in the developing countries in the south-east Asia, crushed brick are being used in many civil engineering works including road construction. The quality and durability of a bituminous mix is influenced by many factors including types of aggregate mix, compacting energies and water soaking periods. For road construction, locally available aggregate having angular particle, rough texture and having affinity for bitumen should be used. Paranavithana (2006) investigated the effect of recycled concrete aggregates on properties of asphaltic concrete. Test properties of recycled concrete aggregates asphaltic concrete compared with those of fresh aggregate asphaltic concrete. Test results found in this study are encouraging. Sobhan and Zakaria (2001) reported that bituminous macadam base course with picked brick aggregate can give satisfactory result when they are constructed using dense grading, good compaction and subjected to lower soaking period. Zakaria and Rauf (1986) reported that, good quality brick aggregate in unbound condition has found to be satisfactory from strength consideration, provided they are compacted in a dense grading applying appropriate compacting energy. Haque (1976) studied and compare the properties of brick-aggregate asphaltic concrete with those of natural aggregate asphaltic concrete. He reported that, brick-aggregate asphaltic concrete mixes are suitable for use in the surface courses of asphaltic concrete pavements from the stand points of stability, stiffness and deformation characteristics but the asphalt content requirements of brick-aggregate asphaltic mixes are much higher than those of natural aggregate mixes.

The prime objective in our investigation is to determine the brick chips as coarse aggregate in bituminous mix is suitable or not. This investigation includes the determination and comparison the physical properties of fresh aggregates and waste aggregates. As well as to investigate the characteristics of bituminous mixes with fresh aggregates and waste aggregates.

2. MATERIALS AND METHODS

2.1 Coarse aggregates

Coarse aggregates were collected from the brick-kiln and demolished building in Rajshahi city of Bangladesh. The collected samples were broken into pieces manually in 25.00 mm down grade (Sobhan et al. 2011). The aggregate were tested for engineering properties related to flexible pavement. Two types of coarse aggregates were used in this study and these were fresh brick and waste brick aggregates. Fresh brick aggregates consists of brick chips to the sizes of 25.00 mm and less, which were collected from AKC Bricks Manufacturer Company Ltd. Natore, Bangladesh. Waste brick aggregates consists of that aggregate which were collected from demolished buildings in Rajshahi city of Bangladesh. Then aggregates were crushed to obtain 25.00 mm downgrade size (Ruhi, 2004). Unit weight, specific gravity and water absorption of coarse aggregates were determined according to the procedure specified by AASHTO 133, AASHTO T19 and AASHTO T84 respectively. The abrasion value, soundness and ten percent fines value for different aggregates were determined by following test methods AASHTO T104 and BS 812 respectively. All the test results shown in Table 1.

Table 1: Properties of coarse aggregate

Properties	Coarse aggregate	
	Fresh brick	Waste brick
Unit weight, dense, (kg/m^3)	1160	1150
Unit weight, loose, (kg/m^3)	1040	1030
Bulk specific gravity	1.92	1.91
Apparent specific gravity	2.45	2.43
Absorption of water, %	11.30	11.34
Aggregate impact value, %	18	24
Aggregate crushing value, %	30	34
Los Angeles Abrasion, %	38	45
Flaking index, %	17	18
Soundness($MgSO_4$,5 cycle), %	24	27
Ten percent fines value, kN	100	50

2.2 Fine aggregates and filler

Fine aggregate portion of the aggregate passing through 2.36 mm and retained on 0.075 mm sieves were taken from coarser sand. Non-plastic sand finer than 0.075 mm sieve taken as mineral filler in different mix types (Aziz, 1995 and Rangwala, 2000). Specific gravity and water absorption of fine aggregates were determined according to the procedure specified by AASHTO T19 and AASHTO T84 respectively. The Unit weights of filler were ascertained by following test method specified by AASHTO T133. Test results are given in Table 2.

Table 2: Properties of fine aggregate and filler

Property	Fine Aggregate	Mineral Filler
Unit weight, dense, (kg/m^3)	1138.60	1200
Unit weight, loose, (kg/m^3)	1026.32	990
Bulk specific gravity	2.33	…
Apparent specific gravity	2.80	1.87
Absorption of water, percent	7.23	…

2.3 Gradation of aggregates

To investigate the behavior of bituminous mixes with different aggregates, continuously graded bituminous macadam is essential. In the continuously graded bituminous macadam, the aggregate blend is designed to be evenly graded from coarse to fine so as to arrive at a dense mix with a controlled void content, hence producing a stable and durable paving (Paul, 1994 and Sobhan et al. 2011). The gradation of aggregates in bituminous mixes in the present investigation are given in Table 3.

Table 3: Gradation of aggregates in bituminous mixes

Sieve mm	% passing by wt		Cumulative retain %	Individual retain %	% of C.A, F.A & M.F	Individual wt. for 1200 gm
	Specification	Blend				
25.0	100	100	00	00	C.A = 58%	00
19.0	80-100	90	10	10		120
9.50	60-80	70	30	20		240
4.75	48-65	56	44	14		168
2.36	35-50	42	58	14		168
0.60	19-30	25	75	17	F.A = 38%	204
0.30	13-23	18	82	07		84
0.15	7-15	11	89	07		84
0.075	0-8	4	96	07		84
					M.F = 4%	48

C.A.= Coarse Aggregate, F.A.= Fine Aggregate & M.F.= Mineral Filler.

2.4 Bituminous materials

The binder material used for this investigation was of 80-100 penetration grade bitumen and collected from Eastern Refinery, Bangladesh. Routine test as per AASHTO were performed on the bitumen sample and get the following properties: specific gravity, 1.022; penetration value (0.1mm), 84; ductility value, 100^{+} cm; solubility value, 99.85% and flash & fire point, 290°C/310°C.

2.5 Determination of optimum bitumen content

In this investigation for the bituminous mixes of all types of aggregate, a set of six curve were drawn showing the relationships of unit weight, Marshall stability, percentage air voids in total mix, flow value, percentage of voids filled with bitumen (%VFB) and percentage of voids in mineral aggregates (%VMA) with percentage of bitumen content (Chan, 2004 and Sobhan et al. 2011). Optimum bitumen content (OBC) for compacted bituminous mixes was calculated from the following equation.

$$OBC\ (\%) = \frac{BC\ for\ max.unit\ wt. + BC\ for\ max.\ stability + BC\ for\ max.\ 4\%\ air\ voids}{3} \tag{1}$$

3. RESULTS AND DISCUSSIONS

Results shown in Figures 1 and 2 indicates that the unit wt. and stabilities of the compacted specimens for all the mix types, increase initially with an increase in bitumen content, reach a maximum value and then decrease. With the increment of bitumen content, the better compaction was done as a result the density and stability increased. For further increment of bitumen, the thickness of the bituminous film increased as a result the density and stability decreased. From the Table 4, Marshall stability value for fresh and waste aggregate are 12.5 and 12.3 respectively both are greater than the minimum value 5.338 kN according to the Wright (1996) Marshall design criteria for compaction number of 50 blows each end of specimen. The numberof blows for preparation of Marshall test specimen was selected corresponding to 690kPa (100 psi) tyre pressure. The heavy vehicles which on the road of Bangladesh have tyre pressure in the range of 415 to 485 kPa (60 to 70 psi). So the assumption of 690 kPa tyre pressure seems to be quite safe and appropiate.

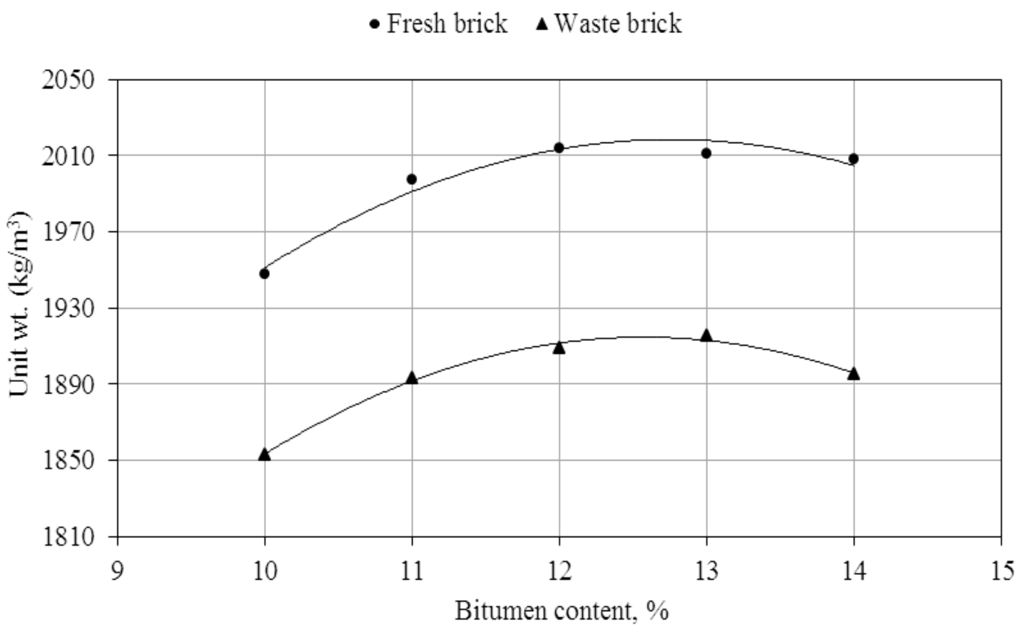

Figure 1: Relationships between unit wt. and bitumen content for different aggregate types

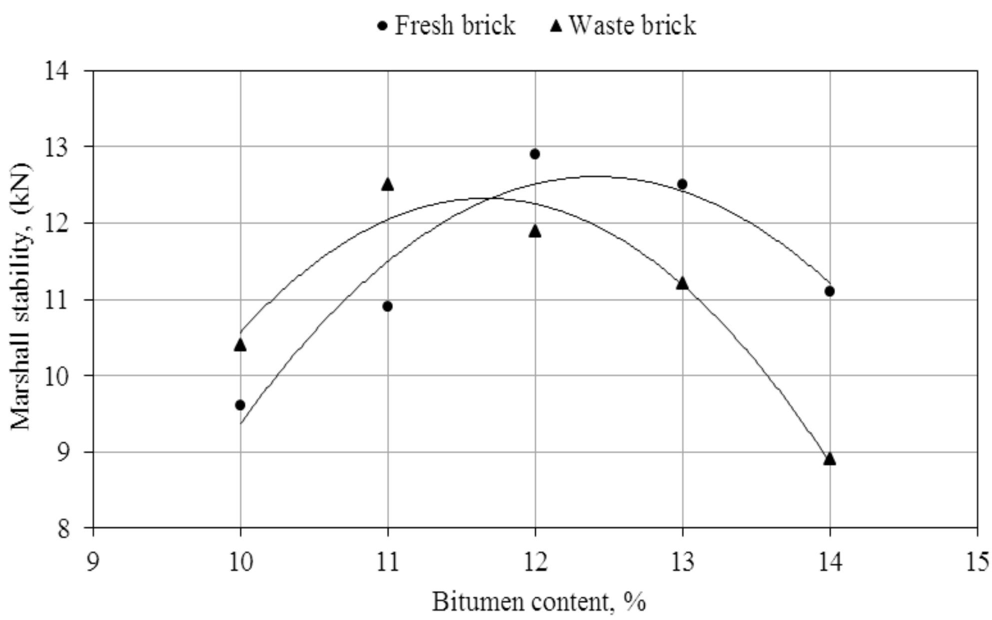

Figure 2: Relationships between Marshall stability and bitumen content for different aggregate types

The void records of the mix with different aggregates reported in Figure 3 shows that the percentage of void in the total mix decreases with increase in bitumen content. This is due to the increase in bitumen content the air voids in the mix decreases and slowly filled with bitumen. In case of waste aggregates, air voids is less than the fresh aggregate this is because the waste aggregates contain cement paste around it so that the voids in waste aggregate filled with cement paste. It is seem from Table 4 that the percentage of voids in total mix at optimum bitumen content for all the mix types are satisfy the limiting value (3 to 5%) specified by The Asphalt Institute (1984).

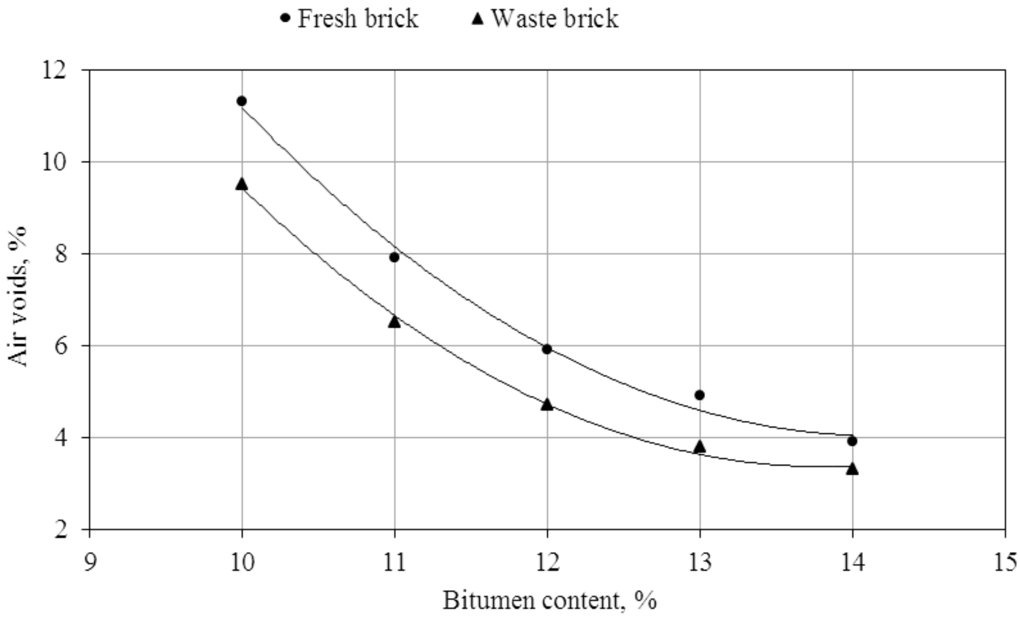

Figure 3: Relationships between percent air voids and bitumen content for different aggregate types

Result shown on Figure 4, flow value increases with increase in bitumen content. With the increment of bitumen in all types of mix, the deformation of the specimen increased. For interpretation of Marshall test results, Lees (1983) considered the stiffness (the ratio of stability to flow) of the mix which can be related to tyre pressure. In order to prevent permanent deformation of the mix under high stress, the Marshall stiffness should not be less than 2.1 kN/mm (120 lb/0.01″) for the design tyre pressure of 100 psi. From Table 4, it is seen the Marshall stiffness are above required value 2.1 kN/mm.

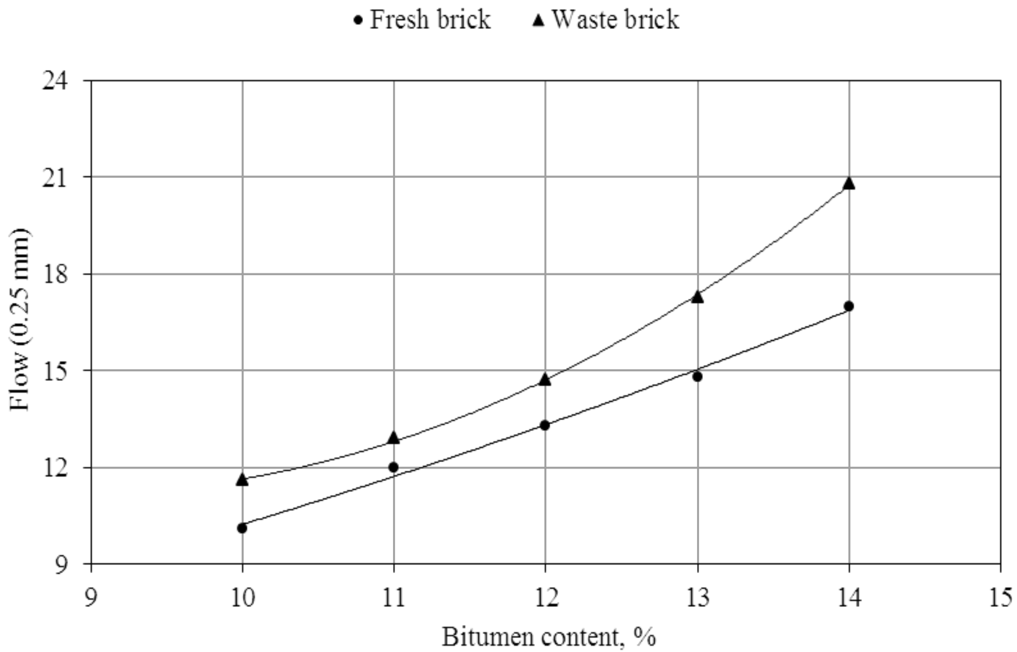

Figure 4: Relationships between flow value and bitumen content for different aggregate types

From the Figure 5 it is seen that, the percentage of voids filled with bitumen increases with the increment of bitumen content for both fresh and waste aggregate. Table 4 shows that, %VFB at optimum bitumen content for fresh and waste aggregates are 72% and 73% respectively. All these values satisfy the limits 65-78% specified by The Asphalt Institute (1984).

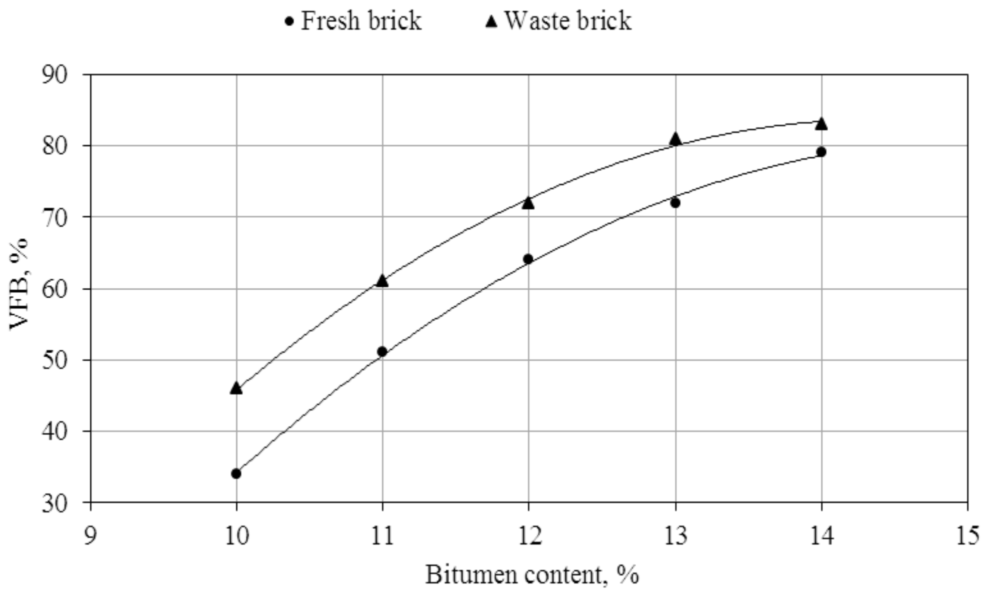

Figure 5: Relationships between %VFB and bitumen content for different aggregate types

From the Figure 6, it is shows that the %VMA initially decreases with the increase in bitumen content but after attain certain minimum value it is increases for further increase in bitumen content for all types of mixes. Table 4 shows that, the % VMA at optimum bitumen content is 16.9% for both fresh and waste aggregates which is greater than the minimum value of 12% specified by The Asphalt Institute (1984).

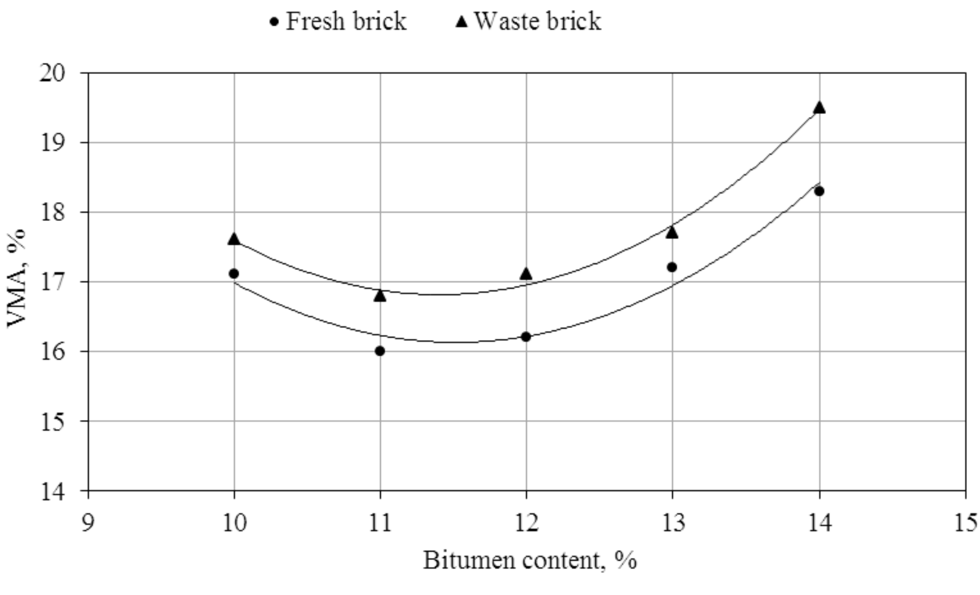

Figure 6: Relationships between %VMA and bitumen content for different aggregate types

Table 4: Comparison of the Marshall characteristics of various types of aggregates

Types of aggregates	OBC (%)	Unit wt. (kg/m^3)	Marshall stability, (kN)	Flow (0.25mm)	% V_a	% VMA	% VFB	Marshall stiffness (kN/mm)
Fresh brick	13	2017	12.5	15	4.6	16.9	72	3.33
Waste brick	12	1910	12.3	14.8	4.8	16.9	73	3.32

4. CONCLUSIONS

On the basis of experimental results in this investigation, it is concluded that the dense graded bituminous mixes with fresh and waste bricks as a coarse aggregate is a feasible option. From the experimental results it shows that, bituminous mixes with fresh brick aggregates and waste brick aggregates both satisfy all the requirement of a bituminous binder course for 690 kPa (100 psi) tyre pressure which having higher optimum bitumen content (13% and 12% respectively) and that can be used for bituminous pavement construction from the stand point of stability, stiffness and voids characteristics.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECT OF FILLERS ON BITUMINOUS PAVING MIXES: AN EXPERIMENTAL STUDY

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ABSTRACT

Fillers play an important role on engineering properties of the bituminous paving mixes. Conventionally, cement, lime and stone dust are used as fillers. In this study, an attempt has been made to assess the effect of different types of fillers (e.g. non-conventional and conventional) on the Marshall properties of bituminous paving mixes. For this purpose, one non-conventional filler such as brick dust and two conventional fillers such as cement and stone dust were used. All of these materials were tested in the Transpotation Engineering laboratory of Khulna University of Engineering & Technology based on the standard test procedure of AASHTO. The results presented in this study were based on the Marshall mix design parameters. Total 15 sets of test specimens were prepared using different types of fillers by weight having different amount in the mix. The Marshall properties obtained for both types of fillers reveal that, non-conventional filler (brick dust) specimens found to exhibit higher stability value compared to conventional filler (cement and stone dust) specimens. In addition, bituminous mixes containing brick dust as filler showed maximum stability at 6.2% bitumen content and the percentage of air voids were found to be decreased with the increase of bitumen content.It can be concluded that the non-conventional filler(brick dust) generated as waste material can be utilized satisfactorily for paving purposes. Thus, the fillers used in this study are likely to solve the solid waste disposal problems to some extent.

Keywords: *Bituminous paving mixes, Brick dust, Cement, Filler, Stone dust, Marshall mix design*

1. INTRODUCTION

Asphalt pavements are a crucial part of our nation's strategy for building a high performance transportation network for the future. Asphalt pavement construction is fast and relatively simple; it is economical and the materials to make it are widely available. It has been recognized that the filler plays a major role in behavior of the asphalt mixtures. Fillers fill voids between coarse aggregates in the mixture and alter properties of the binder, because the filler acts as an integral part of the mastic (combination of bitumen, filler, and entrapped air). The quality of mastic influences the overall mechanical performance of asphalt mixtures (Kim et al. 2003). It plays a great role in both binding and aggregate interlocking effect (Terrwl and Epps, 1988). The filler has the ability to increase the resistance of particle to move within the mix matrix and/or works as an active material when it interacts with the asphalt cement to change the properties of the mastic (Kalkattawi, 1993).

According to the study of Craus et al. (1978), the interactive physio-chemical aspect between the filler and bitumen is related to adsorption intensity at the filler-bitumen interface, and higher surface activity significantly contributes to stronger bonds at the filler-bitumen interface. It can be inferred that the interactive role associated with the physio-chemical reaction is influenced by the type of bitumen and filler as well therefore; selecting the proper type of filler in asphalt mixture would improve its properties and thus enhance the mixture performance. Mineral fillers have traditionally been used in asphalt mixtures to fill the voids between the larger aggregate particles. Generally, the aggregate material passing the No.200 sieve is referred to as filler. In ASTM D242, mineral filler is defined as consisting of finely divided mineral matter, such as rock dust, slag dust, hydrated lime, hydraulic binder, fly ash, loess, or other suitable mineral matter.

In Bangladesh, conventionally cement, lime and stone dust are used as fillers. In this study, however, an attempt has been made to assess the effect of different types of fillers (e.g. non-conventional and conventional) on the Marshall properties of bituminous paving mixes. For this purpose, non-conventional filler such as brick dust and conventional fillers such as cement and stone dust were used. The main objective of this study was to investigate the potential use of non-conventional filler (brick dust) in asphalt concrete mixtures.

2. MATERIALS AND METHODS

Asphalt mixture mainly consists of two basic ingredients: mineral aggregate and asphalt binder. The process in asphalt mix design involves the determination of type of aggregate and asphalt binder and the proportion of these two ingredients to achieve the desired bituminous mixture performance. The most common methods used are the Hveem, Marshall and Superpave methods. In general, all mix design methods involve three basic variables, namely asphalt binder choice, aggregate selection, and determination of optimum asphalt content.

In this study 15 sets of test specimens were prepared by using different types of fillers by weight in the mix. Specimen preparation, compaction, and testing were conducted in accordance with ASTM D1559 (Marshall Mix design method). The materials used in the study include bitumen, coarse aggregate, fine aggregate and mineral filler.

2.1 Asphalt Binder

The bitumen used in the study was penetration grade 80/100. It was purchased from local distributor of the company. Table 1 presents bitumen properties.

Table 1: Bitumen properties

Property	Obtained Value	Standard values (According to ASTM)
Penetration at 25˚C (1/10 mm)	99	80-100
Softening point ˚C	48.50	45-52
Specific Gravity	1.03	1.03-1.06
Ductility, cm	102	Minimum 100 cm

2.2 Mineral fillers

The conventional filler material used for the preparation of specimen was cement and stone dust. As non conventional filler, brick dust was used. Cement used in this study was fine powder that is gay in color and was purchased from the local distributor of the Meghna Cement Mills Ltd. Stone dust and brick dust were also collected from different local sources. The filler materials should pass through No. 200 sieve. The specific gravity of different types of fillers is presented in Table 2.

Table 2: Specific Gravity of the used filler materials

Filler type	Specific gravity
Cement	2.75
Stone dust	2.68
Brick dust	2.70

2.3 Aggregates

In addition to the asphalt binder and mineralfillers used in the Marshall procedues, crushed stone was also used in the preparation of asphalt concrete specimens. This is as the aggregate make up 90 to 95 percent by total weight of the mixture; they have great influence in the performance of the mixture. Aggregates which possess sufficient strength, hardness, toughness, specific gravity and shape were chosen. The physical properties of aggregates were determined and are shown in Table 3. From the test results, it was found that the properties of aggregates were within the specified limits. Figure 1 shows the midline gradation of the aggregate used in the mix design.

Table 3: Physical properties of aggregates

Test description	Coarse aggregates	Fine aggregates	Standard values
Aggregate Crushing Value (%)	25	-	< 30
Aggregate Impact Value (%)	23	-	< 30
Specific gravity	2.60	2.67	2.6 – 2.9
Flakiness Index (%)	12	-	< 25
Elongation Index (%)	14	-	< 25

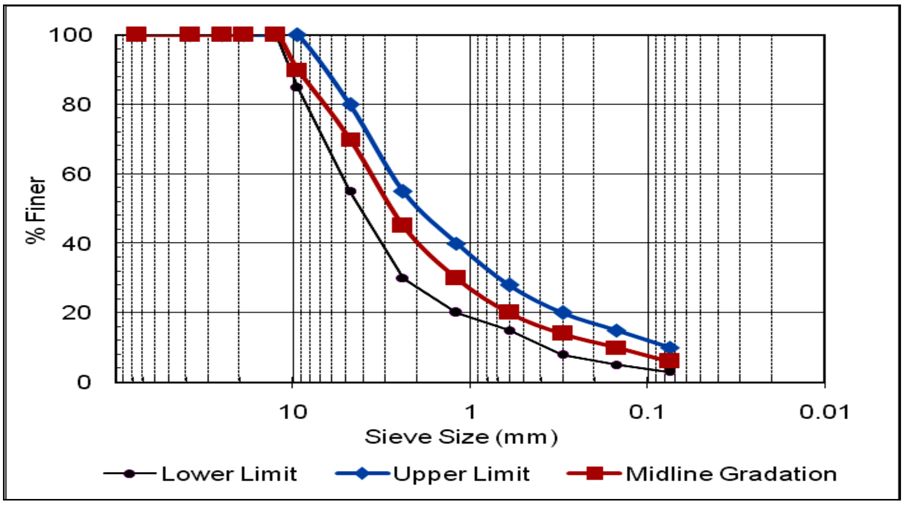

Figure 1: Aggregate gradation used in asphalt concrete Mixture

2.4 Marshall Mix Designs

Five percentages (4.5, 5, 5.5, 6 and 6.5 %) of asphalt cement were used for each filler type. The binder weighted to the desired amount and then added to the heated aggregates and filler in the mixing bowl. All components were mixed thoroughly until all the aggregate and filler particles were completely coated with asphalt. The mixing temperature was maintained within the required limit (155-165 °C) for the mixture. According to ASTM (D1559) cylindrical specimens which were 4 inch (101.6mm) in diameter and 2.5 inch (63.5mm) in height were prepared. Marshall Stability and flow test were performed in each specimen. The cylindrical specimens were then compressed on the lateral surface at constant rate of 2in/min. (50.8 mm/min.) until the maximum load (failure) is reached. The load resistance and the corresponding flow value were recorded. Three specimens for each combination were prepared and the average results were reported. The bulk specific gravity and density ASTM (D2726), theoretical (maximum) specific gravity of void less mixture were determined in accordance with ASTM (D2041). The percentage of air voids was then calculated.

3. RESULTS AND DISCUSSIONS

The relationships between unit weight versus asphalt content, stability versus asphalt content, flow versus asphalt content, voids in total mix (VTM) versus aspphalt content, and voids in mineral aggregate (VMA) versus asphalt content were developed for each mixture as a part of the Marshall Mix design method. The results of the Marshall test of individual specimens and average Marshall Properties of specimens prepared with brick dust, cement and stone dust as filler materials for varying bitumen contents are presented in Table 4, Table 5 and Table 6, respectively.

Table 4: Marshall Properties of samples with Brick dust

Bitumen (%)	Stability (kN)	Flow value (mm)	Unit weight (g/cc)	VTM (%)	VMA (%)
4.5	8.37	2.4	2.21	10.1	20.05
5.0	9	2.6	2.23	8.5	19.6
5.5	11.5	2.75	2.26	6.9	18.96
6.0	12.5	2.9	2.27	5.8	19.2
6.5	10.7	3.5	2.26	4.9	19.5

Table 5: Average Marshall Properties of samples with Cement

Bitumen (%)	Stability (kN)	Flow value (mm)	Unit weight (g/cc)	VTM (%)	VMA (%)
4.5	9.06	2.4	2.24	8.94	18.96
5.0	9.32	2.45	2.27	7.1	17.2
5.5	10.86	2.55	2.31	5.1	16
6.0	10.92	2.65	2.33	3.73	15.2
6.5	9.96	3.2	2.31	3.2	15.9

Table 6: Marshall Properties of samples with Stone dust

Bitumen (%)	Stability (kN)	Flow value (mm)	Unit weight (g/cc)	VTM (%)	VMA (%)
4.5	5.67	1.67	2.18	11.24	20.76
5.0	6.6	1.85	2.20	9.84	19.4
5.5	7.4	2.1	2.29	5.76	18.1
6.0	7.9	2.45	2.29	4.69	17.91
6.5	7.1	2.9	2.29	3.9	18.4

Figure 2 to Figure 4 show the Marshall mix design data obtained for the three different types of filler. An optimum asphalt content of 6.2% (by weight of aggregate) was obtained for the mixtures which contain brick dust as filler material (Figure 2). At this optimum value, the Marshall Stability,flow, unit weight, %VTM and %VMA values were 11.5 KN, 3.2 mm, 2.27 g/cm^3, 5.5% and 19.25%, respectively. An optimum asphalt content of 6.1% (by weight of aggregate) was obtained for the mixtures which contain cement as filler material (Figure 3). At this optimum value, the Marshall Stability,flow, unit weight, %VTM and %VMA values were 10.7 kN, 2.8 mm, 2.32 g/cm^3, 3.8% and 15.5%, respectively. When stone dust was used as filler then an optimum asphalt content of 6.3% (by weight of aggregate) was obtained for the mixtures (Figure 4). At this optimum value, the Marshall Stability, flow, unit weight, %VTM and %VMA values were 7.5 kN, 2.75 mm, 2.29 g/cm^3, 4% and 18.1%, respectively.

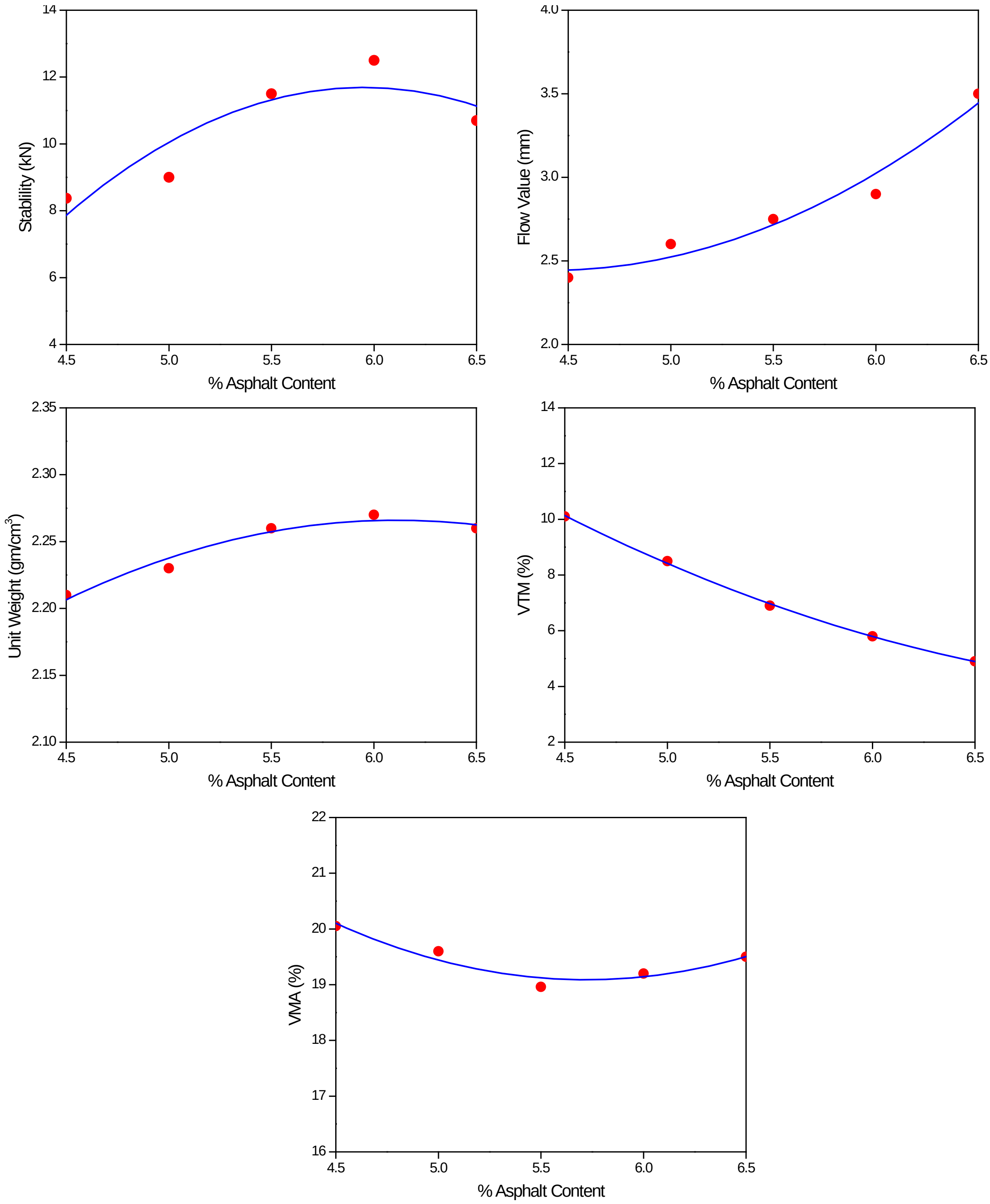

Figure 2: Mashall Mix Design properties using brick dust as filler

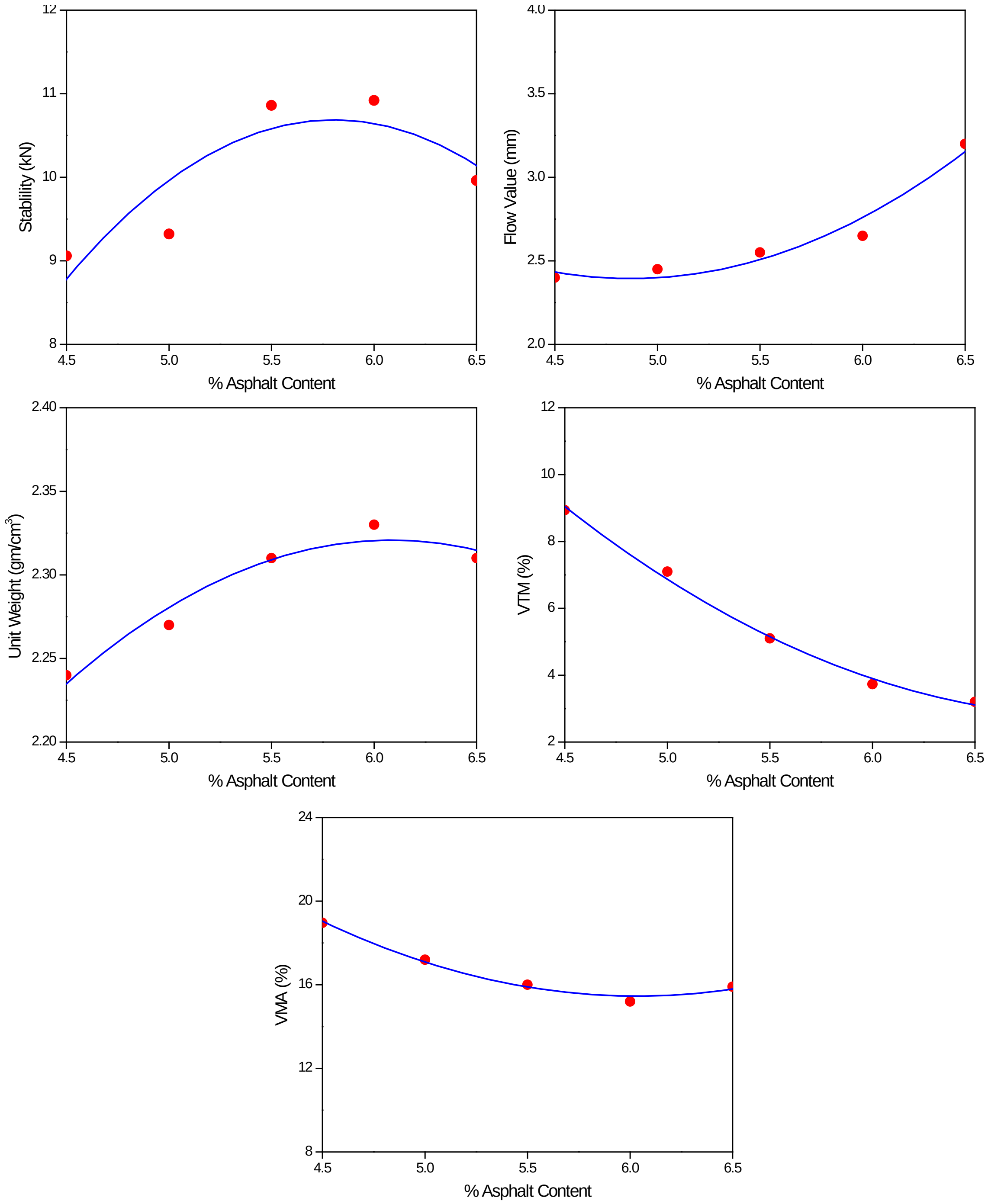

Figure 3: Marshall Mix Design properties using cement as filler

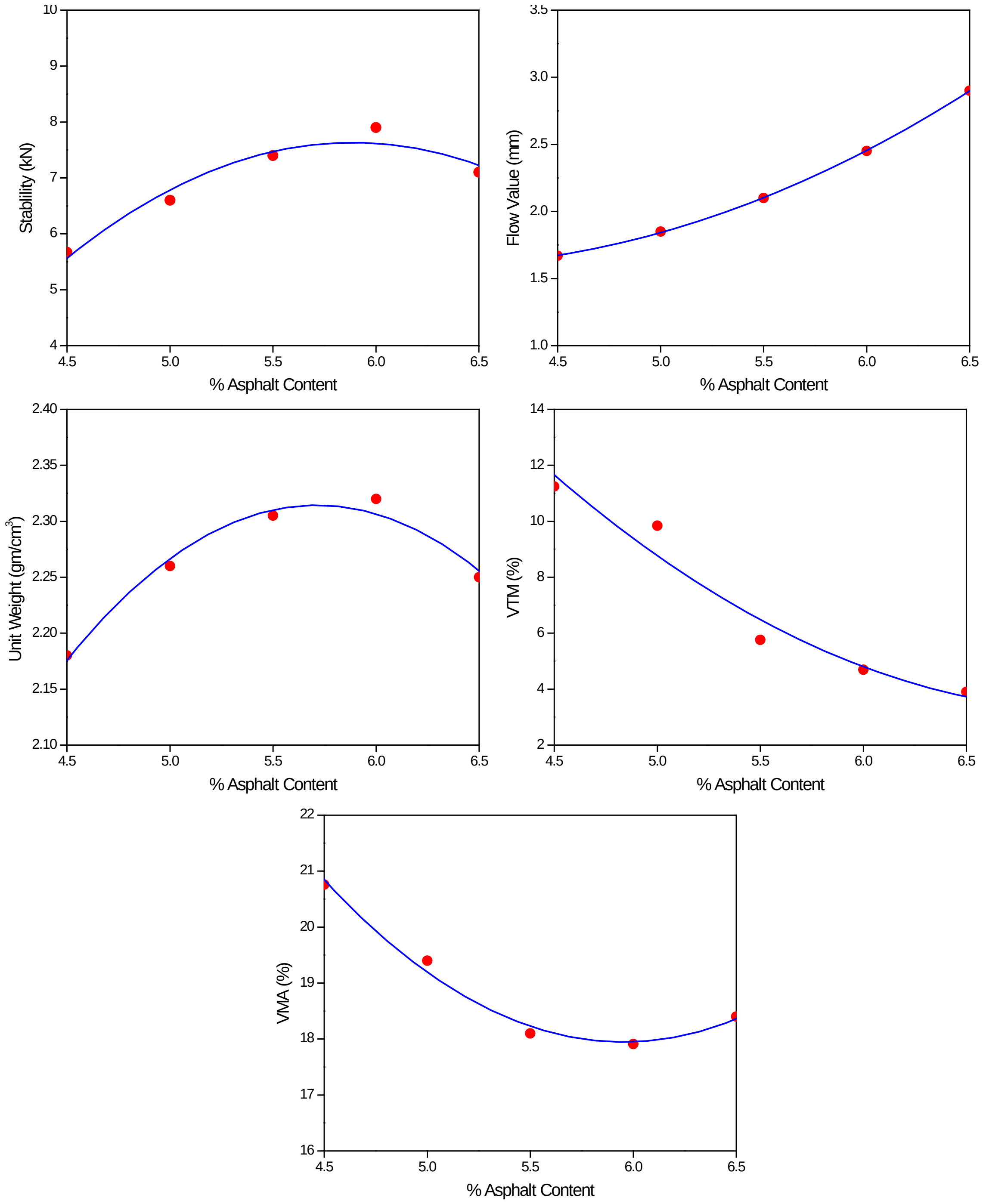

Figure 4: Mashall Mix Design properties using stone dust as filler

Table 7 shows the Asphalt concrete mix design properties by using brick dust, stone dust and cement as filler materials. Test results indicate that bituminous mixes with non-conventional filler (e.g. brick dust) are found to have satisfactory Marshall Properties, which are almost same as those of conventional fillers (e.g. cement and stone dust), thus substantiating the need for its use. The optimum bitumen content for brick dust is almost same as that of conventional fillers which indicates that brick dust will provide the same surface area to absorb bitumen. The Marshall properties obtained for both types of fillers reveal that due to having slightly higher bitumen content, non-conventional filler specimens are found to exhibit higher stability value compared to conventional filler specimens. In addition, bituminous mixes containing brick dust as filler showed maximum stability at 6.2% asphalt content showing increasing trend of flow value with bitumen content. It is seen that brick dust specimen display higher percentage of air voids and low unit weight in comparison with cement and stone dust. But the brick dust specimens have found to give satisfactory results with respect to Roads & Highways Department of Bangladesh, only the exception is %VTM which is slightly more than standard value.

Table 7: Asphalt concrete mix design result

Properties	Brick dust	Cement	Stone dust	Standard values (according to RHD, Bangladesh)
Optimum Asphalt Content, %	6.20%	6.10%	6.30%	--
Stability (kN)	11.5	10.7	7.5	3.43
Flow value (mm)	3.2	2.8	2.75	2-4
Unit wt (gm/cc)	2.27	2.32	2.29	--
VTM (%)	5.5	3.8	4	3-5
VMA (%)	19.25	15.5	18.1	15-20

4. CONCLUSIONS

Bituminous mixes containing brick dust as fillers are found to have Marshall properties almost same as those of conventional (stone dust, cement) fillers. However, the brick dust specimens provide almost same results according to the Roads & Highways Department specification of Bangladesh with an exception of %VTM, which is slightly more than that of standard value. Thus, this study concludes that brick dust can be successfully utilized in the production of bituminous concrete mixes for the highway construction in Bangladesh.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DEVELOPMENT OF PARATRANSIT AS A SUSTAINABLE AND EFFECTIVE MODE OF TRANSPORT- A STUDY BASED ON DHAKA CITY

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ABSTRACT

*Paratransit service especially NMPT (*Non-motorized public Transport *) has been a transportation planning issue for a long time in urban areas. And there has been always a tendency to neglect the paratransit service. But there are histories of successfull implementation of paratransit service along with other modes of transportation. Considering the economic, social and cultural significance, its environmental benefits and its sustainibility as a popular mode, its now time to reconsider the future of paratransit. Rather than neglecting or persuing tha paratransit service, the better approach may be to implement the servise as complementery rather than competitive force for a overall well-integrated transport system. The paper aims at assessing the present status of paratransit in Dhaka city, problems of paratransit system. Surveys were conducted for a better understanding of the present paratransit system. Analyzing the situation, problems were identified by each group of people directly related to paratransit system. The study concludes with scopes for betterment of paratransit service. This paper may be useful for the researchers, policy makers, urban planners and ministries to understand the present scenario and future work in transportation planning of metropolitan cities of Bangladesh.*

Keywords: *paratransit, Dhaka, Transportation, sustainability, conclusions*

1. INTRODUCTION

Paratransit is urban passenger transportation service mostly in highway vehicles operated on public streets on roads in mixed traffic; is provided by private or public operators and it is available to certain groups of users or to the general public, but its adaptable in its routing and scheduling to individual user's desires in varying degree.In Bangladesh, common paratransit modes are Cycle rickshaw, Auto-rickshaw, taxi and Human hauler.

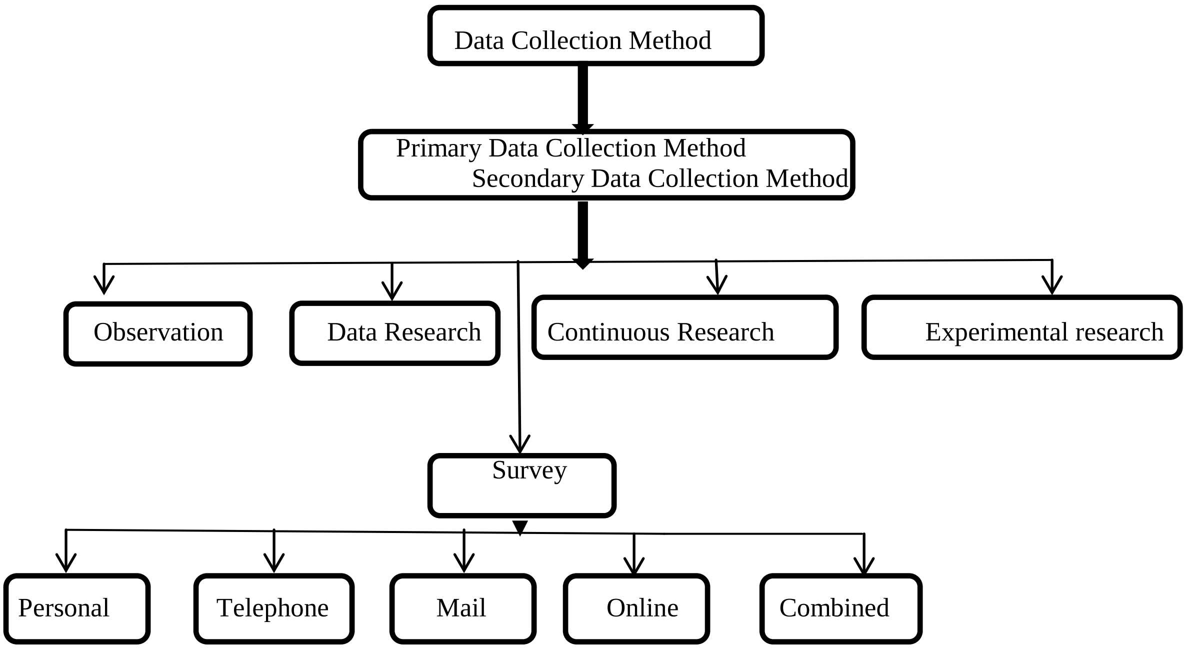
On last few decades as Dhaka is experiencing severe congestion and poor level of service in transportation sector due to rapid and imbalanced growth of developement of this sector. So, urban transport issues of mobility, congestion, safety and environmental aspects have become increasingly important. The most common response at policy level has been seen to focaus on public and private transportation mode and especially traffic control measures and mass transit systems. Inspite of the major contribution of paratransit service to the commuter group, paratranstit system is being looked as a backward, inhuman, inefficient and incosistant mode of transport. There were always issues that NMPT (Non-motorized public Transport) are congestion generators and hinders the stream flow. Despite of the negative pilicies, paratransit service continues as a dominant mode for non-walk trips. Especially rickshaw is the preferred mode for short trips and to special social groups like women, elderly and office goers. In addition paratransit service has grown an emerginng industry with huge workforce involvement. It has environmental benifits and ability to sustain the mobility needs of urban citizens.Considering the role of paratransit system, it is now high time to rethink about the future of paratransit in metropolitan cities where it is a major transportation mode. The approach for improvement of paratransit system should be complementary with other modes rather than competetive force in order to meet the urban transportation demand. The thesis was motivated by the present scenario of Dhaka city where more than 70 percent of road space is occupied by paratransits (Hoque, M.M. and Khondaker, B. and Alam, J.B., 2005). In this paper, information about paratransit service of Bangladesh was gathered. Then problems regarding paratransits were identified and options to alleviate or minimize the problems were suggested. This is an attempt for the developement of paratransit service so that it can serve as a sustainable and effective mode of transportation that would be well-integrated with te other modes of transport.

2. METHODOLOGY

It was indeed necessary to identify the approach of study for this paper on paratransit. The first object was to acquire proper knowledge about paratransit, its present condition and other important features related to it. This study involves the users, drivers and operators of paratransit. So interviewing all of them was an essential part of the study. For this purpose, proper questionnaires were prepared.

Data collection process included: Identification of type of data requirement both qualitative and quantitative, Selection of data items e.g. age, car ownership, income, level of use of paratransit, no. of trips per day, purpose of trips, income of paratransit drivers, data items for owners etc., Developing separate questionnaires for users, driver and operators of paratransit and Conducting the survey

The data collection method generally used is shown below



For data collection, Questionnaires were prepared for a clear and elaborated analysis. There were separate questionnaires for passengers and drivers. The layouts of the questionnaires were as following:

Table: Passenger's questionnaire

Sl. No.	Age	Sex	Education	Occupation	Present Mode Of Travel	Trips Per Day	Trip Purpose	Way Of Getting Mode	Waiting Time

Alternative Modes	Comparison Of Present Mode With Alternative Mode						Difficulties/ Problems	Are You Satisfied With The Service?
	Fare Rate Low/High	Travel Time More/ Less	Waiting Time More/ Less	Cleanliness Good/Bad	Driver's Behavior Good/Bad	Safety Good/Bad		

Table: Driver's Questionnaire

Sl. No.	Age	Sex	Education level	Previous occupation	Do you own this vehicle?	If taxi. AC/ non-AC	Income per month	Reason behind choosing this profession

Driving time	Working days a week	Starting point	preferred area of driving	How long are you in this profession	Difficulties	Any suggestion	Company's name

2. ILLUSTRATIONS

50 paratransit users were interviewed for the survey. Data were collected in questionnaire forms. Survey was conducted at Kamlapur Railway station, Hatirpool & dhanmondi residential areas. Passengers and driver's survey was conducted on July, 2010. Survey of owners of rickshaw, taxicab, CNG-auto and HH was conducted on December, 2010. Survey of taxicab operators was completed by telephone survey. Others were by personal interviews. Most of the data were collected at daytime.

2.1 Analysis of the passengers data

The passenger data were collected by questionnaire survey and then the were analyzed with microsoft excel. The organized data has given a clear conception about the present situation and how to cope up with it for further improvement.

Analyzed data are present as follows:

Modal Distribution:

Passengers were asked to tell about their present mode of transport and the mode of transport they frequently used. The data were organized and following result has been obtained.

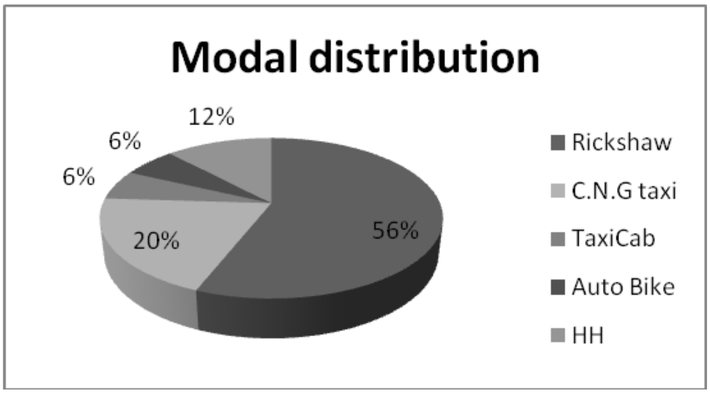


Figure: Modal distribution of paratransit

From this analysis, it is clear that rickshaw is the most preferred mode over the other paratransit. It is because of

- The availability of rickshaw: currently there are 79642 registered rickshaws in Dhaka city (BRTC). Practically the number is way over 10000. So rickshaws are pretty much available all the time excluding some exceptional events.
- Short travel distance: most of the trips in Dhaka city are short rips which are not more than 3 km. According to STP, majority of the people has an average trip length of 2.34 km. that is why rickshaw s preferred.
- Narrow roads unfit for bus service: About 69% of roads of Dhaka has road width of 8.75m or less (CASE, March 2009) which is unfit for bus service. Consequently rickshaws are chosen as an alternative mode of transport.

Car ownership:

There were some passengers who own private cars but yet they use paratransit. For a middle income group, single car ownership refers to the utility of that car to the entire family. But as the members have different travel time and travel purpose at diffefent locations of the city, some has to leave the opportunity of using car and has to use paratransit. Thus use of paratransit often becomes regardless of car ownership. It was a finding from the casual conversations with the passengers.

Income level:

From the trip purpose, education level and personality of the users, most of the passangers were assumed to be from middle income group as passengers were not willing to answer about their income level at all.

Distribution based on gender:

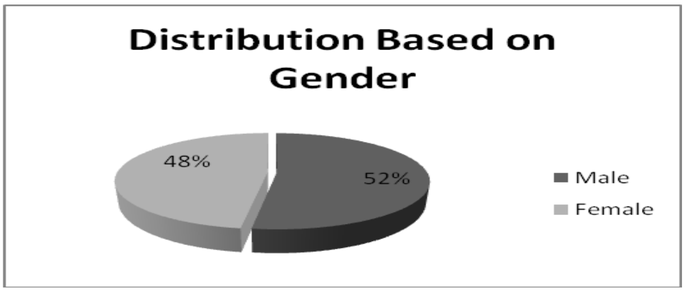


Figure: Gender distribution of users

Generally women prefer paratransit over public transport because of

- Safety issues, Social perspective
- Short distance travel e.g. picking up kids from school, shopping etc.

Reason Behind current trip of the passenger:

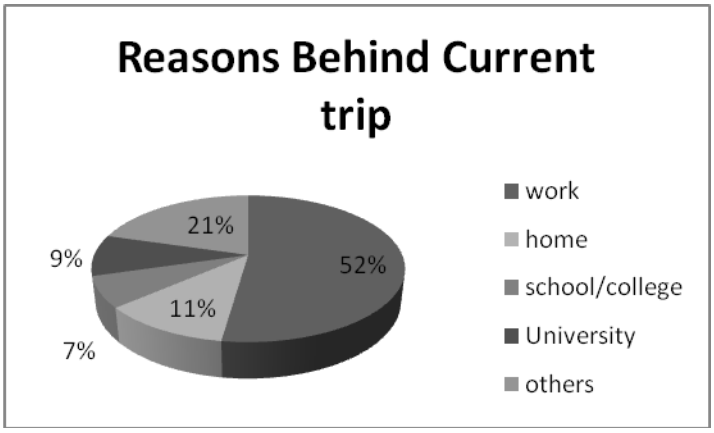


Figure: Trip purpose

Most of the people use paratransit for work trips. A significant no. of trips were used for other reasons like shopping, hospital recreation etc. It is because the trip lengths are short with an average trip length of 2.34 km (STP, 2005).

Overall comparison of different paratransit & bus:

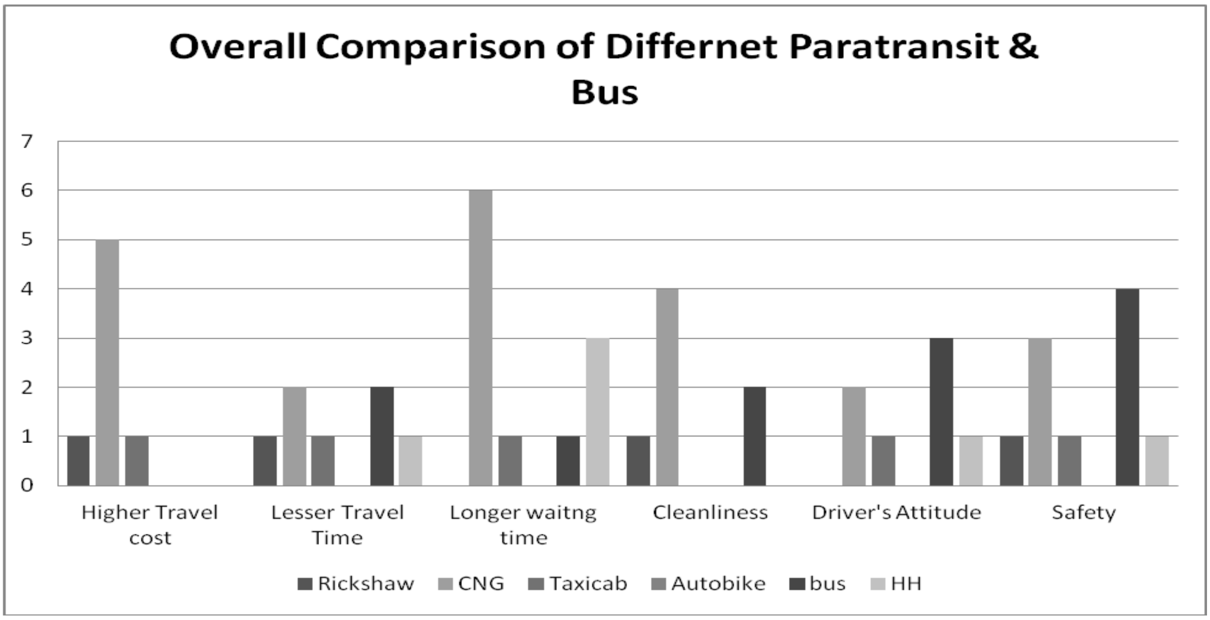


Figure : Overall comparison of different paratransit modes & bus

According to the overall comparison, bus or public transport should have been popular. From the perception of the passengers, bus has lesser travel cost, lesser waiting time and it is more clean and safe. But practically it is not due to

- Unreliable service
- Shortage of bus
- Poor facility
- Lack of routes
- Congestion problems
- Public’s negative perspective toward bus service

There were other data found from the questionnaire survey those were analyzed by pie-chart analysis as well. Those findings were as following,

- Age group of 20-40 is the major portion of the users because their activity involves more trip than others.
- Graduates and students are the major user of paratransit
- With An Weighted Average of 3.2 daily trips per person, most of the users have two daily trips which are HBW trips. More than two trips are generated by youth users.

- Less trip length generally 2-3 km, is a major reason behind using rickshaw. People also prefer rickshaw to walk because of lack of walking facilities, lack of bus routes, safety consideration e.g. crossing busy roads etc.
- CNG-auto, HH, taxicab are generally used for long trips and as an alternative of bus service.
- It has been seen that many people answered they have no alternatives for rickshaw. It means either they have to wait or they pay a higher fare to get the rickshaw.

2.2 Analysis of the Drivers data

In the survey, 35 drivers of different paratransit were interviewed according to the driver's questionnaire. The collected data were analyzed both qualititatively and quantitatively.

Major findings of the analysis of driver's data are following,

- lack of education and skills dragged the drivers especially Rickshaw-pullers into this profession
- they preferred this job over their previous job because of a pretty much higher income
- They demand for better traffic management and safety for their benefits.
- CNG-auto drivers have complaints about police requisition

2.3 Major problems identified by the passenger data:

There was the provision of suggestion from corresponding passenger for betterment of paratransit service. With the help of the survey results and passenger's suggestion following problems were identified;

- Fare is much higher than the bus fare
- At night, safety is a problem
- Reckless driving CNG-auto & Taxicab drivers
- problems in rickshaw free short trips, CNG-auto do not want to take passengers for short trips
- due to road condition, travel is not comfortable
- mixing of traffic with buses endangers life of passengers
- Safe boarding and get down is not considered in case of human hauler passenger

In spite of the problems, paratransits are still the most popular mode of transport. Almost 45% of total modal share is served by paratransit. According to HIS of JICA study team, the number of daily rickshaw trips amount to about 7.9 million.

2.4 Problems identified by drivers data

- though rickshaw require minimum skills of driving, it creates problem for seasonal pullers as they are in confusion about the roadway pattern, traffic regulation etc.

- banning of rickshaw on major trunk roads reduced the income of the puller by 25%-50%. As e result they demand higher fare rate from passengers
- lack of enforcement of law encourages them to ignore traffic rules and drive carelessly
- in order to increase no. of trips, human hauler drivers drive recklessly

2.5 Scope of improvements for the problems identified

2.5.1 Scope for the Betterment of Passengers Service

- ensuring effective meter based fare system of CNG-auto, Taxicab
- ensuring the assistance of traffic police so that rickshaw pullers cannot demand for excess fare.
- segregating rickshaw traffic from high speed vehicles by introducing rickshaw lanes and ensuring its effectiveness
- improving roadway conditions for comfortable movements of rickshaw passengers
- improving walkway facilities to encourage people walking for very short trips
- sufficient street lights and police surveillance at quiet roads to prevent hijacking and ensuring safety of women at night
- Proper stoppage with sheds and benches for human haulers should be constructed.
- To arrange workshops or seminars & refresher’s course for drivers by social welfare department to enlighten them with social norm and beliefs and traffic rules.
- At least crossing of rickshaws should be allowed at different crossing. It would make trip length shorter and thus lower fare rate. For example, if a passenger has to go to mogbazar from Gausia, now he has to travel via katabon, university area, kakarail and shantinogor. But if crossing is allowed at Banglamotor, his trip length will be almost one-third of the previous one.
- There is a road at Shanir Akrhra-Jatrabari route which is in poor condition and not used by any vehicles. Low-cost improvement at this road can allow paratransit to move along this route which will be a congestion relief to Chittagong road.

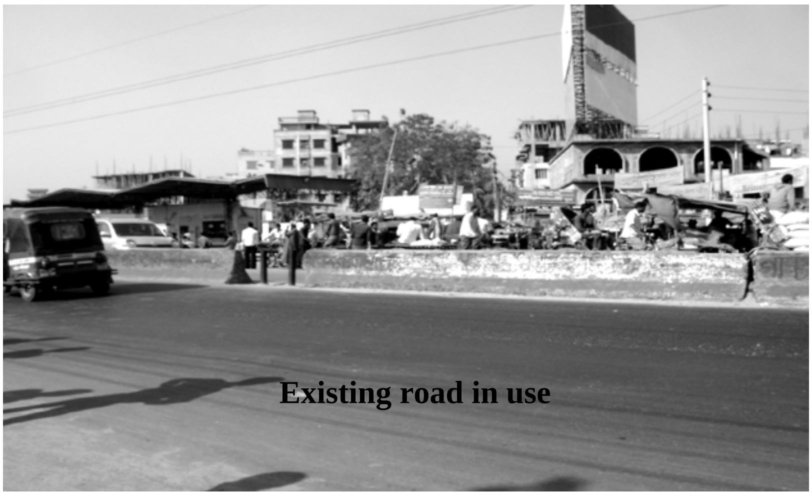


Figure: Existing Shanir Akhra-Jatrabari Road

Figure: NMT movement on both sides of main road

2.5.2 *Scope for the Betterment of Paratransit Service*

- Drivers should have a fixed salary as service holders. If this can be ensured, the tendency of reckless driving for excess trips would be decreased
- to arrange training programs so that drivers are skilled and aware of traffic rules
- Existing traffic rules should be enforced strictly
- Sufficient CC cameras should be installed to monitor traffic violation as well as activities of traffic police
- Number of traffic police should be increased. At present, they have to work for more than 14 hours per day that makes them stressed and somehow they are compelled to take bribes for their daily refreshment
- Ill fitness vehicles should be removed from roads. Government can take initiatives to import vehicles at lower price or manufacture vehicles locally
- Unauthorized rickshaws should be banned and job opportunities should be created to rehabilitate those pullers. Again initiatives should be taken to discourage people from coming to Dhaka city for their living
- To increase the speed and efficiency of rickshaw, better technology can be applied. For example, Pedi cabs can be introduced which are easier to maneuver, economical and need no regular part replacements. If charged overnight, it can travel 160 km overcast a day. A Pedi cab would cost $7000

2.6 Compatibility of scopes with transport strategies of Dhaka

1. The total length of the three proposed BRT corridor is 57 km, and currently 33 km of the Dhaka's road network is rickshaw free (Clean Air And Sustainable Environment (CASE) Preparation Project, Final Report, June 2009). Furthermore, provision of rickshaw stands, CNG-auto & taxicab stands at suitable locations along the corridor will ensure better integration and support BRT system to operate with added efficiency.

2. There are provisions for betterment of paratransit system in Strategic Directions for Transport Development Strategy (TDS) in DITS. They are as follows

- Relocation of important generators/attractors of traffic such as major markets
- Low cost grade separations to reduce the conflicts between slow moving vehicles and motorized vehicles

3. According to the Immediate Action Plan (IAP) in the Greater Dhaka Metropolitan Area Integrated Transport Study (DITS), the emphasis is on the management of the present system. The management issues those include paratransit system are; Management of pedestrian and slow moving vehicles, Road user's education and safety, Enforcement and vehicle quality, Traffic management and engineering and Roadside management
4. The Dhaka Urban Transport Project (DUTP) includes a broad spectrum of projects. The project of infrastructure development are as follows:
    - Traffic management measures and system improvements that includes improvement of junctions with pavement reconstruction, separation of motorized and NMT traffic, diversion of rickshaw at certain key junction and improved traffic control
    - Rehabilitation and improvement of existing roads
    - Rehabilitation of existing sidewalks, construction of about 40 km of new sidewalks along the existing major arterial roads, and construction of about ten pedestrian bridges
    - Improvement of the parking and traffic facilities for NMT on the secondary and local roads to reduce NMT at major arterials. Provision of additional road links and three underpasses to facilitate a pilot NMT network. Separation of rickshaws from motor vehicle by suitable barriers in some routes
5. In Strategic transport planning, there are certain recommendations to improve existing paratransit system as follows
    - Improving drivers behavior by instituting program to move forward in a positive and definite manner
    - A major commitment to improve all types of pedestrian facilities is recommended to encourage people who have a choice to walk more often
    - For rickshaws it is recommended to rely upon market demand and market forces to determine the number of rickshaws in operation, rather than through the issuance of licensing control
    - At the same time, the creation of a proper licensing system, coupled with an effective mechanism for enforcement of the regulations, will automatically reduce the number of rickshaws in operation as many of the vehicles would not pass a roadworthiness test
    - A parallel program of licensing of pullers should be introduced to train them properly the rules of the roads and prove their eligibility to operate the vehicles

3. CONCLUSION

The current state of paratransit system in Dhaka city shows its immense necessity to its users. It is clearly understandable that the system will sustain in Dhaka city in future too. Again, a significant portion of Dhaka populations relies directly or indirectly on paratransit including drivers, operators, their families etc.. Generally users of Dhaka have very supportive attitude towards paratransit especially rickshaw. But some users are generally not satisfied by the service but they have no other way but use paratransit. Again people are attracted to the profession of drivers of paratransit as it is a good source of income without much basic knowledge and skill. A huge industry concentrating paratransit service has been grown up and still emerging at a large scale resulting in blaming paratransit for cause of congestion. Poor traffic enforcement and management encourages the increasing number of paratransit. Despite this, there is definitely a massive generation of revenue from this field of transport. So significance of paratransit in Dhaka cannot be denied at all. It is indeed important to study the paratransit service in detail especially from economic perspective. Proper measures should be evaluated both technically and economically to ensure the smooth operation of paratransit in Dhaka as well as better integration with public transport. Benefit of each group; users, drivers and operators should be considered for a fair approach keeping in mind that paratransit plays a very vital role in our economy. Furthermore, Dhaka city could be the role model for traffic planning in other metropolitan cities of Bangladesh.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECTS OF CAR INVOLVEMENT IN CONGESTION AND ROAD ACCIDENT IN DHAKA CITY: A STUDY ON SUSTAINABLE SOLUTIONS

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ABSTRACT

The population of Dhaka increases very rapidly due to the opportunities of better social, employeemnt and economic activities. But its traffic congestion and road accident reduce the potential productivity and consequences are very alarming now a days. The growth of motor cycle and cars are very high in Dhaka city. The growth of cars is much more in this city than that of whole Bangladesh over the recent years. Eighty percent of total registered motor cars are from Dhaka city. It is also noticeable that fatal accident of cars is increasing in the recent years. Due to the long queue of vehicles, rear end and collision only accident are predominating. Lack of proper facility for pedestrian produce hit pedestrian accident which is also mentionable. In this paper the data of traffic and accident are taken from the database of BRTA and ARI of BUET. Car sharing, car pricing, parking & footpath management ,segregation of NMV vehicles, exclusive bus lane, proper scheduled public transport and information technology can be some sustainable management & safety measures to minimize suffocating congestion and severe road accident caused by increasing growth of cars in Dhaka city.

Keywords: *Car, congestion, road accident, sustainability, BRTA.*

1. INTRODUCTION

The population of cities increases very rapidly due to the opportunities of better social and economic activities. Urban transportation is quite challenging to maintain the increasing demand among peoples. When transportation system becomes failure to maintain a satisfactory level of services to the people, several problem arise including congestion, delay and consequent emission of pollutants (Alam & Habib, 2003). Traffic congestion reduces the productivity of urban agglomeration everywhere, but the consequences in developing country megacities are even greater (TRB 2006). To adjust with the hasty increasing transportation need, it is essential to utilize the new and available facilities properly. It is very important to find out the factors which are responsible for creating congestion and accidents and suitable measures must be taken against those. Comprehensive planning considering future demand and consequences would provide a better solution to the present situation.

Dhaka, the capital of Bangladesh has 15 million population and 2000 square kilometers of area. The road networks cover about 3000 km. Vehicle population of this city is around 603,200 up to year 2010 (BRTA, 2008). Metropolitan Dhaka accounts for nearly 40 percent of total urban population in Bangladesh. Due to the limitation of land acquisition, limited roads and streets cannot cope with the high demand, congestion occurs everyday almost every important points in the city. It is also a matter of great concern that car ownership increasing rapidly which is one of the major issues of traffic congestion in Dhaka city. The light vehicle proportion to the total vehicles is 38.5 percent. The cost of congestion and accidents in this city is US$ 520 million/ annum which is quite alarming. As this city continues its growth toward the anticipated 36 million people in 2024 and beyond, the requirement for a sustainable system will become even more acute. The challenge for the transport sector is to provide a safe, mobile, affordable, comfortable and available system to a large segment of the population. The negative impact of congestion is so intensive that people suffers hours after hours during traveling and lose time & money every day. A study revealed that the roads of this city are congested for almost 10-12 hours (Siddique 2010). Several steps have been undertaken to reduce traffic congestion in Dhaka city, such as imposing restriction on the entrance of trucks in Dhaka during daytime, banning of cycle rickshaws in different roads, changing the time table of the trains, construction of foot over bridges rather than zebra crossings, construction of flyovers, and so on but the congestion level did not reduce. Meanwhile, projects have been undertaken and are under consideration to, for example, provide more parking spaces and construct flyovers and an underground metro so that cars can move around freely on the roads. Only private cars have been virtually over looked from measures to reduce congestion.

In case of road accidents, According to the police statistics, road accident in Bangladesh claims on an averages 4,000 lives and injure another 5,000 in a year. However WHO estimates that the actual fatalities could well be 20,038 each year (WHO, 2009). In total 43,157 accidents occurred in Bangladesh during 1998-2009 and about seven percent of total accidents were caused by cars. There are 887 fatal, 970 grievous injuries, 264 simple injuries and 846 are collision type accidents(Ahsan, Raihan & Rahman,2011). According to BRTA(2008), about 15.8 percent of total casualty accidents and 14.4 percent of total fatal accidents have been occurred in Dhaka city in the year 2008. It is very essential to know the effects of car accidents in Dhaka city where huge number of cars runs every day.This paper focuses on only the effects of car involvement in congestion and road accidents and necessary solutions to reduce the negative impacts in roads of Dhaka city.

2. ROAD AND TRAFFIC CONDITIONS IN DHAKA CITY

The impact of rapid growth population and thus number of vehicles have major consequences on the ability of the transport sector to provide mobility for all people as they seek to take advantage of employment, education, health and social opportunities. The transportation system of Dhaka is predominantly road based and other modes are almost neglected to be improved. The different parts of Dhaka city have no good interconnection of roads. The major roads in Dhaka include: Mirpur Road (north-westerly); Begum Rokeya Sharani (northwesterly); Airport Road and Pragati Sharani (northerly); Dhaka-Chittagong Road (easterly) and Sylhet Road (north-easterly); Dhaka-Narayanganj Road (south-easterly); and Mawa Road (southerly.The two interchanges at Mohakhali and Khilgaon are the part of the improvement of our transport network but operational activities of Mohakhali interchange is like overpass of the rail crossing which generate itself additional congestion at the entrances of the ramps. The under constructed Jatrabari and Kuril interchanges are well designed but their effects on the networks are still unknown. Rests of the at grade junctions are controlled and regulated either by traffic police or traffic signal where most are not properly demand responsive .

BRTA register the degree of motorization in Dhaka city. With a population of approximately 15 millions, this results in an auto ownership of approximately 13 per 1,000 populations and a vehicle ownership (including buses, trucks, taxis, CNGs) of 32 per 1,000. According to the StrategicTransport Plan (DTCB, 2005), the whole Dhaka city have been equally divided in both north-south (Abdullahpur to Postagola) and east-west (Khilgaon to Turag river) direction by screen lines and traffic counts have been performed along this lines called screen line counts. Auto-rickshaws (with 36.8%) and Cars/Light Vehicles (with 43%) comprise a substantial proportion of all motorized vehicles crossing the screen lines; Cars/light vehicles (With 9.6%) and auto-rickshaws (with 4.5%) each serve relatively low proportions of road users.

3. DATA COLLECTION AND METHODOLOGY

Bangladesh Road Transport Authority (BRTA) has establishes in the year 1988 and it register different types of vehicles around the country. For this paper, the data about number of vehicles and accidents have been collected from the database of BRTA. Besides, some accident data were also collected from the Accident Research Institute (ARI) of BUET. Road Traffic accident data are collected by the Police working at thana level by filling in 'Accident Reporting Form (ARF)' one for each accident, which was introduced nation-wide in 1997. This form is written in Bangla and published by the Government of Bangladesh. ARFs are compiled at the Regional Accident Data Units (ADUs) maintained in six police Ranges and four Metropolitan Police offices. Then the data are entered into an electronic database called the 'Micro-computer Accident Analysis Package (MAAP).The customized software application (database) was developed by the Transport Research Laboratory (TRL, UK). These modified documents are then collected by BRTA and ARI of BUET for further analysis and research.

4. INVOLVEMENT OF NUMBER OF CARS IN DHAKA CITY

Along with aviation, "motor cars are increasingly the favored modes for passenger transport but are also significantly the most damaging" (Chapman, 2007). A key factor influencing the increase in private cars is the construction of new roads Bangkok's car ownership level stands at 399 per 1000 population compared with Singapore's 152 per thousand, although Singapore is a more economically developed country (Amin, 2009).

This is in sharp contrast with the observed positive relationship between motorization and the "level of economic development. By 2020, it is predicted that about 60 percent of the major roads will become highly congested with a speed of less than 5 km/hr during peak hours in Dhaka city (Alam & Habib, 2003). It is clearly observed that light vehicles with 38.5% percent comprise a substantial proportion of all motorized vehicles within Dhaka city but those serve relatively very low proportions of users with 9.6 percent.

In Figure 1. it is observed that the increase of motor cycle is highest in Bangladesh over the years 2003 to 2010. The next highest growth occurs for car and auto rickshaw. The average growth rate of cars over the years is ten percent. In the year 2008 to 2010 the growth rates of cars over eleven percentage of total. It is clear to understand that the increase of number of cars is ascending. Among all types of vehicles the registered cars is twenty eight percent of total which is the second highest numbers in Dhaka city as shown in Figure 2. The rests are less than twelve percent of total except motorcycle. Although the numbers of motor cycle (with 41%) are huge, those occupy very few road spaces. In spite of motor cycle, cars consume much more space and its occupancy is not full in most of the case. Out of 207,500 numbers of motor cars, 166,840 numbers are registered in Dhaka city which is 80.4 percent of total motor cars.

In case of Dhaka city the picture is more crucial. Here again the highest number of vehicle is motorcycle but the growth of cars is much higher than that of whole Bangladesh (Figure 3). From year 2008 to 2010 the growth rate of cars goes from eleven percent to thirteen percent. The average growth rate is more than nine percent.

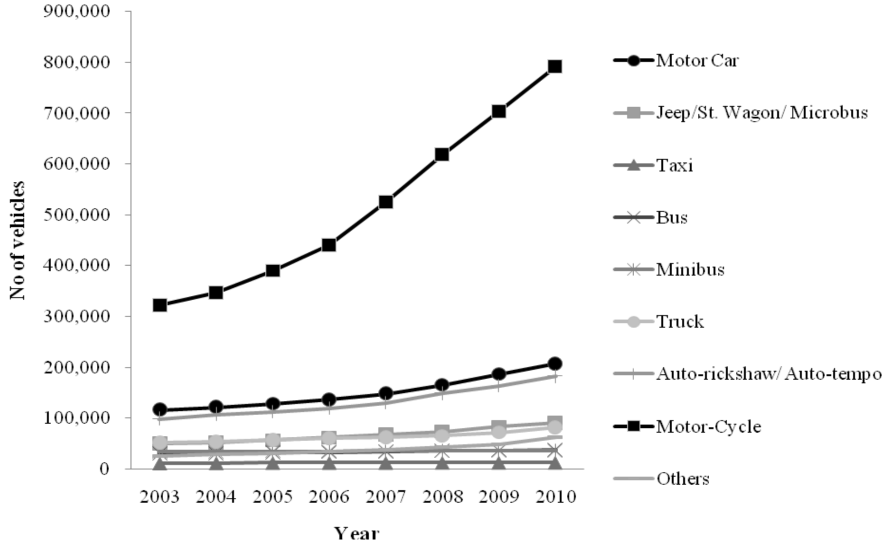

Figure 1: Trends of different types of registered vehicles in Bangladesh

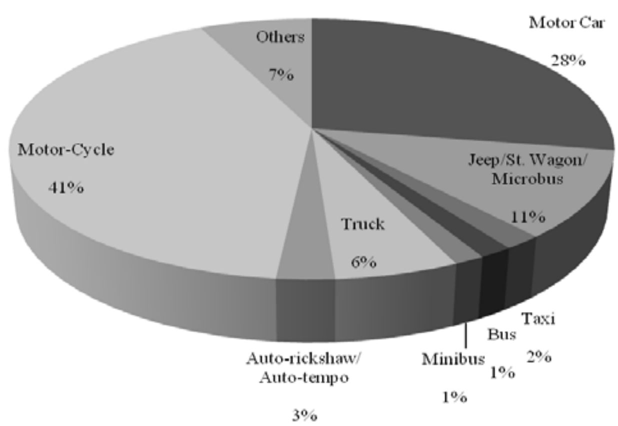

Figure 2: Composition of registered vehicles in Dhaka city

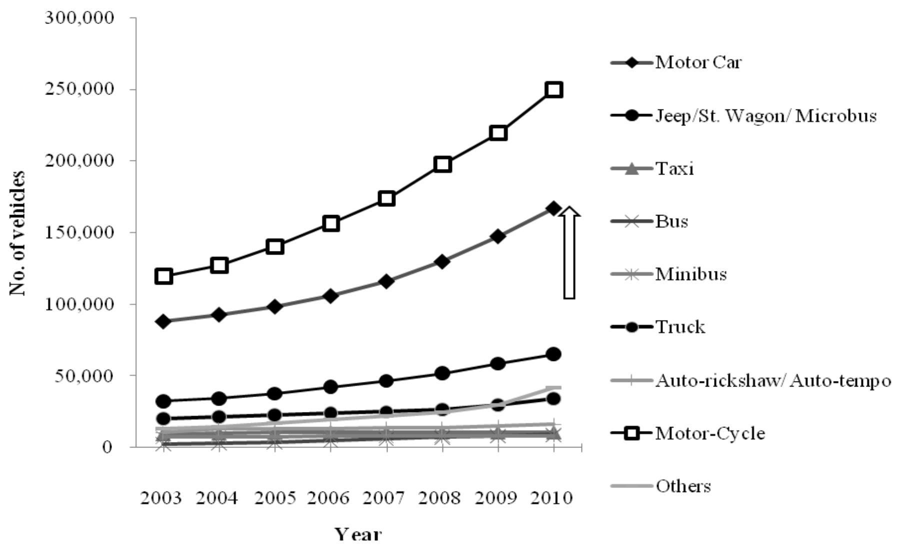

Figure 3: Trends of different types of registered vehicles in Dhaka city

5. INVOLVEMENT OF CARS IN ROAD ACCIDENT IN DHAKA CITY

The detail analyses about the road accidents all over the world as well as Bangladesh are important to understand the nature and pattern of the road safety problem. While every year around 4000 deaths are reported to the police. It is estimated that over 10,000 deaths resulting from road crashes occur annually. In the year 1990, road accident as a cause of death or disability were no means insignificant, lying the ninth place out of a total of over 100 separately identified causes (WHO, 2009). By the year 2030 forecasts suggest that road accident will move from ninth to fifth leading cause of death. In developed countries numbers of cars are very high. Despite the constraints of development the growth of cars and their accidents are significant in Banglaadesh.

The trend of car accidents in Bangladesh shows that grievous injury accident of cars is declining but fatal accident is increasing over the year 2007 to 2009 is shown in Figure 4. It is also noticeable that collision only accident is quite significant along the other types of accident. In Dhaka city almost eleven percent casualty accidents occur due to car crashes in year 2008(Figure 5.). In Figure 6. the highest type of accident is collision only. The trends of grievous accidents are higher than that of fatal accidents. The grievous and fatal accidents are in average thirty five percent and sixteen percent of total respectively. It can be said that due to huge urban congestion, low speed of cars create good numbers of collusion type accidents. But in rural highways, speed of cars affects the severity of accidents and as a result more fatal accidents occur. In Figure 7. car accidents are mainly distributed between hit pedestrian and rear end collision in Dhaka in year 2008. Rests are merely visible to that graph.

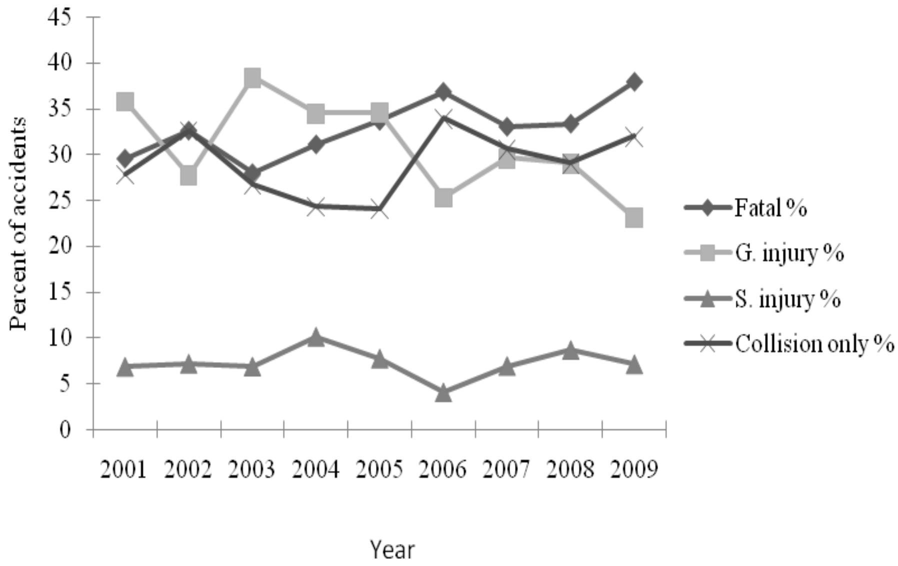

Figure 4. Trends of accident severity of cars in Bangladesh

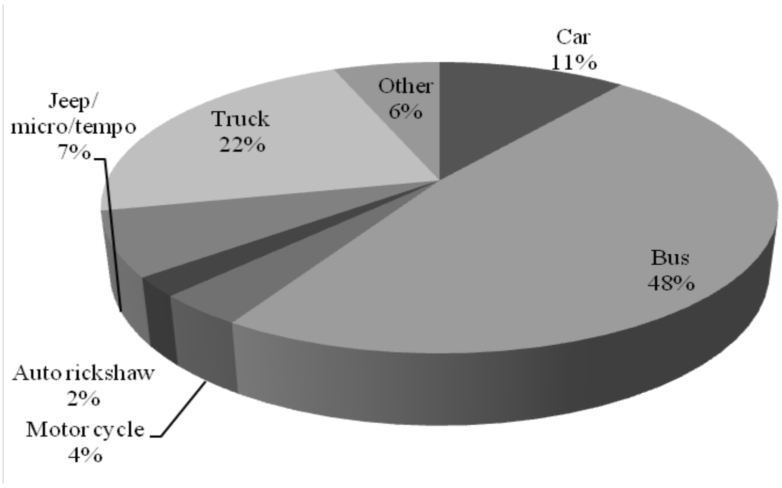

Figure 5. Composition of casualty accident of different vehicles in Dhaka city

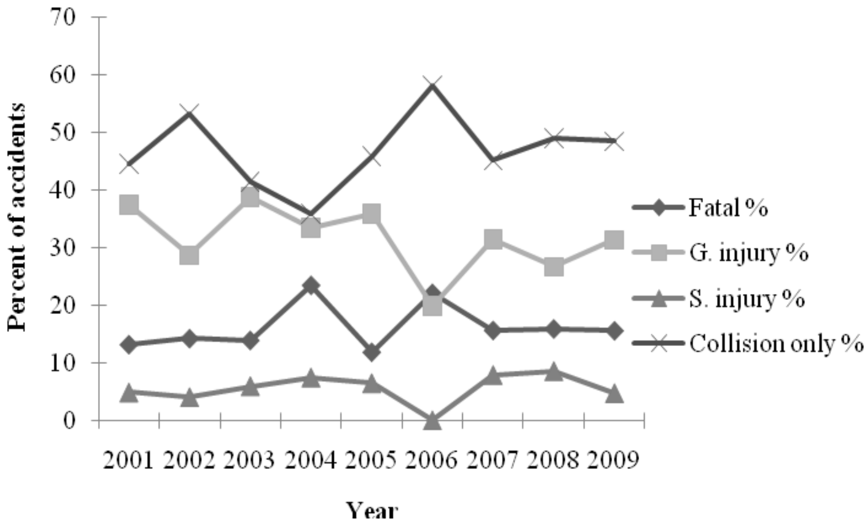

Figure 6. Trends of accident severity of cars

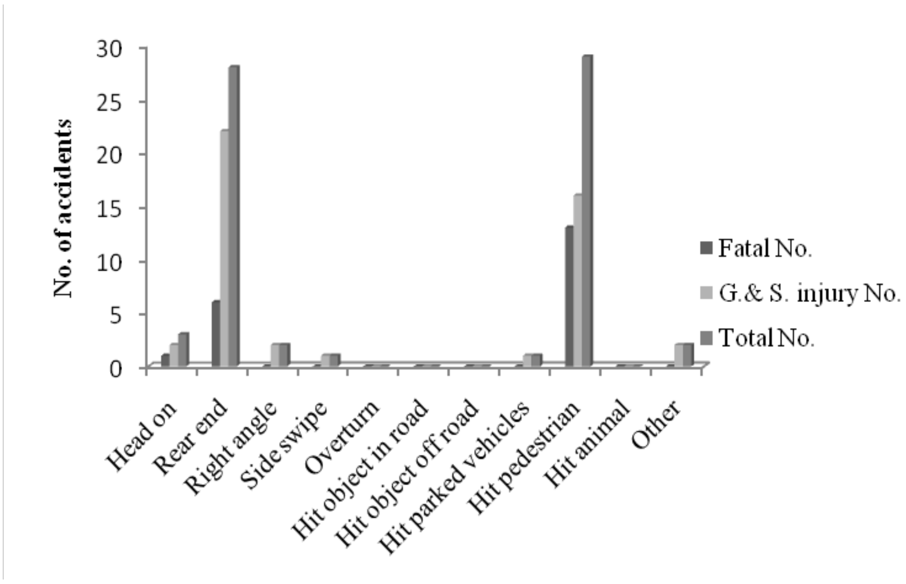

Figure 7.Distribution of collision type of car accidents

6. EFFECTS OF CAR CONGESTION AND ACCIDENT: SUSTAINABLE MEASURES

According to BRTA, everyday they give in average almost 50 licenses to private cars in year 2010. Besides the registered cars, thousands of illegal vehicle operate on the city's roads. Due to the banning of cycle-rickshaw from the most of the major roads of Dhaka city, people shift from the banned mode to other modes. Because of the banning of rickshaws in different streets of the city, transport cost and the suffering of the middle and poor income people have increased to a great extent. Moreover, in almost every part of the city, road space, footpaths, playgrounds, and empty lots, which are occupied by private cars at little or no charge. According to Dhaka City Corporation (DCC), more than 4,000 cars are parked in the Motijheel-Dilkusha area daily. Approximately 400 kms of footpaths are available for pedestrians of which 40% are being occupied illegally by vendors and others (Rahman 2008). To reduce the congestion of Dhaka, two flyovers have been constructed in Mohakhali and Khilgaon. But the prime objectives of the project could not be fulfilled as the congestion problem has become more severe in the areas. Private cars are the major users of the flyovers. The more facilities encouraging private cars are implemented, the more the congestion level will increase. Many cities in the world have constructed multi-level elevated expressways, but it affects the health hazard like cancers rather than reducing congestion. Already Seoul in South Korea have demolished their established elevated expressways . Because of the absence of a quality transport system, people are using private cars, motorized three wheelers and taxicabs more frequently.

The data and analysis showed the intensity of cars and the severity of car accidents throughout the city. It is easily understandable that sustainable countermeasures are required to reduce the effects of cars both in safety and congestion. Some traffic management and road environment measures are given below:

- Ridesharing or car sharing can directly reduce the number of cars from the road. It encourages using the full occupancy of cars. It refers vehicles carry additional passengers. Studies have shown that ride sharing program can reduce daily vehicle commute trips to specific work sites by 5-15%, and up to 20% or more if implemented with parking pricing (Ewing, 1993).
- Congestion pricing reduces the number of vehicles on a highway at peak periods by charging drivers for using the highway during those periods. Congestion pricing could be charged to discourage people from using their own vehicles. This type of pricing has the potential to reduce the need for new highway capacity, improve air quality, relieve peak traffic congestion, increase the use of high-occupancy vehicles, reduce automobile use in highly congested urban environment, raised revenue for much needed transportation improvement and establish a rational pricing system following sound economic principles (Zupan, 1992).
- Parking charge on an hourly basis, or fractions can bring the control and regulation of uncontrolled parking along the city streets. It also collects the fair market price for parking and this revenue can be used for public transport improvement.
- Parking of cars or hawker and shops on the footpath compel pedestrian to share the same road with running vehicles. It is very hazardous as about half of the car casualty accidents are due to hit pedestrians. In this city, many low income people choose the walking as a mode. So, it is very urgent to clear the preoccupied footpath for pedestrians.
- In addition to the installation of road signs, markings, construction of parallel service roads on both side of highway, protective raised walking and crossing facilities etc. should be provided in the locations where pedestrian and NMV activities are high.
- Public transit is very affordable to middle and low income people. It can also carry a good number of people and occupy small space for large number of people. Segregated bus lane or BRT can be introduced as bus would face no congestion. Emphasizing public transport for offices and educational institutions is also necessary to reduce the uses of private cars.
- Licensing private cars should be strict and huge car tax should be implemented to reduce the purchasing cars among the new users. This extra cost will discourage people to buy a new car.
- It is found that collision only and rear end collision of car accident are very high in Dhaka city. These types of accident increase when signal queuing are frequent. To eliminate these types of collision, traffic congestion should be minimized.
- It is true that flyovers can increase the capacity of roadway but this facility also generate the new car users as the empty spaces of flyovers will be occupied. Ultimately it would not be very fruitful to reduce congestion or accidents. Despite supply strategies, demand management should be prioritized.

7. CONCLUSIONS

The growth of motor cycle and cars are very high in Dhaka city. The growth of cars is much more in this city than that of whole Bangladesh over the recent years. Although the numbers of motor cycle are huge, those occupy very few road spaces. In spite of motor cycle, cars consume much more space and its occupancy is not full in most of the case. Eighty percent of total registered motor cars are from Dhaka city. Cars are very much clustered in this city and make it more congested and polluted. It is also noticeable that fatal accident of cars is increasing in the recent years. Due to the long queue of traffic, rear end and collision only accident are predominating. Lack of proper facility for pedestrian hit pedestrian accident is also mentionable. To overcome the increasing problem of congestion some sustainable measures against cars should be undertaken. Car sharing, car pricing, parking & footpath management offer less investment but provide a durable solution. Segregation of non-motorized vehicle (NMV), exclusive bus lane, proper scheduled public transport and information technology are some supply strategies which are very urgent to reduce the involvement of car in congestion and road accident in Dhaka city.

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CAPACITY EVALUATION OF ROUNDABOUT INTERSECTIONS IN KHULNA METROPOLITAN CITY, BANGLADESH

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ABSTRACT

This analysis addresses the most important element of operational performance of capacity analysis of roundabout traffic intersections in Khulna Metropolitan City (KMC), Bangladesh. The capacity analysis was conducted based on the empirical gap- acceptance method that is adopted by aaSIDRA software. The necessary geometric data for the analysis (average entry width, circulatory road width, number of entry lane and circulatory lane, and island diameter), traffic movement data with vehicle characteristics and pedestrian volume were collected from two roundabouts (Mailapota and Shibbari) in Khulna Metropolitan City. The results of the capacity analysis by using aaSIDRA software indicate that both of the roundabouts have their degree of saturation greater than 0.85. This 0.85 value is recommended by the analysis procedure of some model countries such as Australia, Germany, United Kingdom and U.S.A. whose roundabouts are designed to operate at no more than 85 percents of their estimated capacity. Effective capacity versus geometric parameter relationship have been developed in order to find out the causes of over saturation (v/c ratio greater than 0.85) and the results indicates that number of entry lanes, number of circulatory lane, high traffic flow and pedestrian volumes are the major causes of the over saturation.

Keywords*:* *Roundabout, aaSIDRA, effective capacity, over saturation.*

1. INTRODUCTION

Evaluation of intersection capacity is very important since it is directly related to delay, level of service, accident, operation cost and environmental issues. Transport research Laboratory of England first introduced modern roundabout in the early 1960 (Mark, 2003). These facilities were introduced in order to solve the problems of the existing rotaries and traffic circles. Moreover, improvement of safety is the most distinct advantage of roundabouts. France is building almost 1500 roundabouts a year. In the Netherlands, since the late 1980s, approximately 400 roundabouts have been built over a period of only six years (Thaweesak, 1998). For more than three decades modern roundabouts have been used successfully throughout the world as a junction control device (Akcelik, 1997).

In Khulna Metropolitan City, most of the roundabouts are four legs and these have served more than twenty years. But little attention has been paid to the design and capacity evaluation of the roundabouts; nobody knows their capacities or level of services. Now a day, it is very common to see traffic congestion at intersections in KMC at the morning and afternoon peak hours. Hence, the traffic police need to intervene in the situation to regulate the traffic flow by over- riding the traffic control devices. Otherwise, it would be practically impossible to have normal traffic flows, especially at roundabout junctions, which is more dependent on driver behavior and balanced traffic flow between the approaches. This problem will continue and it may worsen in future due to the rapid growth of population and number of vehicles in KMC. Poor road planning and sub-standard geometric conditions of roundabouts have a significant effect on roundabout capacity and traffic congestion (May, 1990). Therefore, it is essential to evaluate the capacity of roundabout for proper traffic operation and to give a clear picture for the planners and traffic engineers involved in highway junction design and traffic operation tasks. Road authorities and other concerned bodies need to conduct a comprehensive capacity and delay study of every roundabout so that they can come up with solutions for the traffic congestions, traffic delays, and level of services, accidents and operating costs. The objective of this study is to evaluate the capacity of roundabout intersections in Khulna Metropolitan City, Bangladesh.

2. METHODOLOGY

Roundabout's geometric and peak hour traffic data were required in order to achieve the objective of this study. The roundabouts were chosen based on the principle of possible representative of the target population of roundabouts in terms of size and numbers. In this study, aaSIDRA software was used for the analysis. Since aaSIDRA does not totally depends on geometric elements, but it is more dependent on traffic rules so that by collecting traffic data and observing some geometric features it is possible to carryout the capacity analysis. In order to collect data eight undergraduate students were trained. The necessary geometric and traffic data were collected at two roundabouts in KMC.

As per the requirement of aaSIDRA and Akcelik's base capacity formula (Akcelik, 2005), the geometric data such as island diameter, circulatory width, number of circulatory lanes, entry lane number, and average lane width at entry were collected. These data were collected by a tape and observed on the roundabout existing sites. Data of vehicle and pedestrian volume were collected at peak hours with the direction of movements. The traffic volume data were collected manually for a period of one hour on working day of sunny weather.

3. RESULTS AND DISCUSSIONS

As per the methodology, the island diameter, circulatory width, number of circulatory lanes, entry lane number, and average lane width at entry were collected. The collected geometric data are summarized in Table 1.

Table 1 Summary of intersection geometry

Sl. No.	Name of the Roundabout	No. of legs	No. of Circulatory Lane (m)	Island Diameter (m)	Circulatory Road width (m)	Inscribed Circle Diameter (m)
1.	Mailapota roundabout	4	2	7	11	18
2.	Shibbari roundabout	4	3	19	17	36

Table 1 shows the summary of intersection geometry at two roundabouts. It is seen that the island diameter of the roundabouts is 7 m and 19 m. When circulatory width is added, the diameter becomes 18 m and 36 m, which can be categorized from mini-roundabouts to multilane roundabouts according to roundabout information guide (FHWA, 2000).

Traffic movements of vehicle and vehicle classification are important parameters for capacity analysis using either aaSIDA or Akcelik's base capacity formula. High pedestrian volume has a significant effect on capacity. The volume of vehicles and pedestrians at peak hour (4:30 PM to 5:30 PM) is summarized in Table 2.

Table 2 Peak hour's volume of vehicles at both of the roundabouts

Roundabout	Heavy Vehicles			Light Vehicles									Total Traffic (Nos.)	Pedestrians volume
	Bus	Truck	Sub Total	Car	Pickup	Micro	Jeep	MV (3W)	NMV (2W)	Motorcycle	Pedal cycle	Sub Total		
1	24	35	59	153	22	32	16	807	2506	941	406	4883	4942	321
2	16	27	43	108	12	34	10	820	1187	272	166	2609	2652	262

[1]Mailapota roundabout, [2]Shibbari roundabout

It is seen in Table 2 that the total number of light vehicles travel was higher than heavy vehicles in both of the roundabouts and the percentage of heavy vehicles at Mailapota roundabout and Shibbari roundabout was about 1% and 2% of total traffic volume, respectively. The traffic movement on the approaches or legs and the traffic volume in passenger car unit (PCU) are necessary for data analysis. The PCU values given in the Geometric Design of Highways (MoC, 2001) are given in Table 3, which were used to convert the number of different type's vehicle in PCU value. The entry traffic flows on different legs of the roundabouts in PCU are shown in

Table 4. It is seen that there is unbalanced traffic flow at legs or approaches in both of the roundabouts. However, it is not recommended to build roundabouts as traffic control devices when there is unbalanced traffic on the legs (FHWA, 2000).

Table 3 PCU of different types of vehicle in Bangladesh (MoC, 2001)

Vehicle categories	PCU
Passenger Car	1.00
Light goods vehicle	1.00
Truck	3.00
Bus	3.00
Auto-rickshaw/Motorcycle	0.75
Rickshaw/Van	2.00
Pedal cycle	0.50

Table 4 Entry Traffic flow on roundabout legs

Roundabout	Leg No.	Name of the Leg	Entry Traffic on Legs (PCU)	Traffic Share (%)
Mailapota Roundabout	1	KDA Avenue	3272	47
	2	Satkhira Road	1654	24
	3	Garib Newaz Road	1118	16
	4	Power House Road	882	13
		Total	6926	100
Shibbari Roundabout	1	KDA Avenue	790	22
	2	Khan A Sabur Road	915	26
	3	Khan A Sabur Road	1227	34
	4	Jalil Sarani	637	18
		Total	3569	100

It has already mentioned that aaSIRDA software was used for the analysis. The aaSIDRA capacity analysis results for both of the roundabouts are summarized in Table 5. The performance was measured with volume to capacity (v/c) ratio or degree of saturation and level of service also applied according to United States Highway Capacity Manual (US HCM) (TRB, 2000).

Table 5: Results of capacity analysis on the roundabouts

Roundabouts	Total Flow (PCU/h)	Effective Capacity (PCU/h)	Degree of Saturation (v/c)	LOS
Mailapota Roundabout	6926	5604	5.513	F
Shibbari Roundabout	3569	6725	3.981	F

Table 5 shows the results of the capacity analysis on the roundabouts. It is seen that the Mailapota roundabout has lower effective capacity compared to the entry flow. The LOS of both of the roundabouts was F. Actually the performance or capacity of roundabout depends on the approaches or legs performance and their v/c ratio is taken from the maximum v/c ratio of the legs. Figure 1 shows the total flow and effective capacity in both of the roundabouts.

Lane-by-lane capacity was also carried out for both of the roundabouts. The capacity at legs, degree of saturation and opposing flow has been summarized as shown in Table 6.

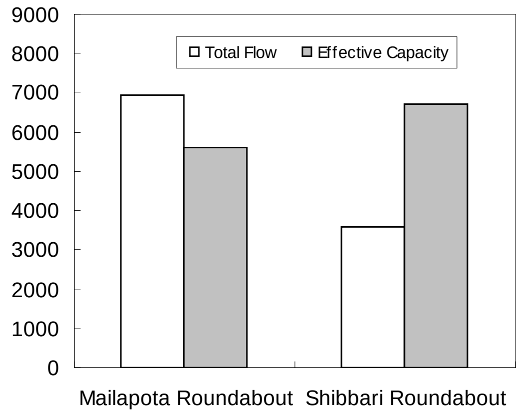

Figure 1 Graphical representation of Total flow and Effective capacity

Table 6: Results of the capacity analysis on the approach or legs

Roundabout	Leg No.	Traffic counts at legs	Opposing circulatory flow	Degree of Saturation (v/c)	Capacity at leg	(v/c) > 0.85
Mailapota Roundabout	1	3272	567	0.899	3639	0.049
	2	1654	2876	5.513	300	4.663
	3	1118	1168	2.6	430	1.750
	4	882	503	0.901	979	0.051
Shibbari Roundabout	1	790	396	0.227	3836	-0.623
	2	915	659	0.724	1455	-0.126
	3	1227	767	0.967	1274	0.117
	4	637	1892	3.981	160	3.131

By observing the (v/c)>0.85 column of Table 6 which is based on Highway Capacity Manual (TRB, 2000), it is easily identify the legs which are in critical condition. It is seen that v/c ratio of all of the four legs at Mailapota roundabout and two legs (leg no. 1 & leg no. 2) at Shibbari roundabout are greater than 0.85. So, these legs are in critical condition. The problem of the approach is identified by using Table 6 which shows v/c>0.85, traffic volume of entry flow at legs and traffic volume of circulatory flow and circulatory lane numbers. Table 7 shows the summary of the conditions of roundabouts problems.

Table 7: Summary of the condition of the roundabouts

Name of the Roundabouts	Leg no.	Problems
Mailapota Roundabout	1	Entry lane number is not adequate.
	2	Entry lane number not adequate and circulatory lane number is also not adequate.
	3	Circulatory lane number is not adequate.
	4	Entry lane number not adequate.
Shibbari Roundabout	3	Entry lane number not adequate. Traffic volume is also very high.
	4	Entry lane number not adequate and circulatory lane number is also not adequate.

3. CONCLUSIONS

The results of the capacity analysis of roundabouts in Khulna Metropolitan City indicate that most of the roundabouts are in serious problems or over saturated. Based on observed field conditions, it is common to notice that the traffic police have to regulate the traffic of these roundabouts in the peak hours. As the study revealed the major problems are related to inadequacy of numbers of entry lanes, numbers of circulatory lanes, high traffic flow and unbalanced traffic on the approaches. Besides, the roundabouts are built when the traffic flow was lower and future traffic volume extension was uncertain. Even if modern roundabouts traffic rules are to be applied to Khulna city roundabouts, some of the important geometric elements don't exist at Mailapota and Shibbari roundabout, such as deflection, proper island splitters. Deflection is the most important geometric element, which forces drivers to reduce their speed and to avoid collision between neighboring leg entering vehicles. The splitter islands on the roundabout approaches provide important indication to the driver as to the angle and radius of approach on entry to the roundabout. All the geometric data used to input parameters for empirical method capacity analysis do not exist at the roundabouts in Khulna city; thus, only analytical method was the option to carryout the capacity analysis with some geometric elements by using aaSIDRA. There are also high traffic flows at some legs of both of the roundabouts that show high percentage of traffic volume share, which is not recommended for roundabouts.

RECOMMENDATIONS

The geometric data of the roundabouts in Khulna city should be revised and built up properly as stated in the design manual of modern roundabout since they are very helpful to have reasonable capacity and traffic safety. The peak hour traffic volumes in some legs of both of the roundabouts were found over saturated. Moreover, the entry traffic flow (6926 PCU) at Mailapota roundabout is higher than the effective capacity (5604 PCU). It is recommended to increase the entry lane numbers of lane width and circulatory lane.

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A STUDY ON PEDESTRIAN CHARACTERISTICS IN KHULNA METROPOLITAN CITY, BANGLADESH

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ABSTRACT

This paper deals with the findings of pedestrian flow characteristics on walkways in Khulna Metropolitan City, Bangladesh. Data were collected at three locations by using a digital camera. The data were analyzed by using the statistical software SPSSv11.5. The mean walking speed of the pedestrian of Khulna city is 52.83 m/min (2.89 ft/sec.). This speed is slower than that of Asian and American counterpart. The collected data were used to develop the speed-flow-density relationship of pedestrian. The free-flow speed of pedestrianis is observed to be 64.14 m/min. which is lower than the Singapore, Britain and United States.Moreover, pedestrian characteristics from various cities in the world are compared. The collected data and established relationships could be used as a basis for the development of more efficient, adequate and safer facilities for the pedestrians.

Keywords: Pedestrian, speed, flow, density, walkways

1. INTRODUCTION

Walking is the most efficient and effective mode of transportation for short trips. Walking is directly involved with using other modes. The importance of pedestrian movements is understood globally and cannot be overemphasized. Khulna Metropolitan City (KMC) is the 3rd largest city of Bangladesh with a population of about 1.4 million and a land area of only 45.65 km^2 (BBS, 2007). The rapid developments of Khulna city as well as the increase of population have put pressure on the pedestrian network. In order to identify the importance of the pedestrian mode, the pedestrian characteristics for various pedestrian facilities need to be investigated. In Khulna, little attention has been given to study on pedestrian behaviour and pedestrian flow characteristics. A review of different models for pedestrian facilities around the world was conducted. It was concluded that there was a fundamental need for studying the characteristics of Khulna City's pedestrians and developing a pedestrian model for road network improvement purposes.

The research works on pedestrians flow characteristics started about four decades ago. Oeding (1963) indicated that the speed/flow relationships for different types of pedestrians are different. Fruin (1971) also reported that different speed/flow relationships were established for different types of pedestrian facilities. A comprehensive review of Asian pedestrian characteristics can be found in Morrall et al. (1991) in which a comparison of the pedestrian characteristics in Canadian and Asian cities was made. Lam et al. (1995) studied the pedestrian flow characteristics in Hong Kong and investigated the walking speed/pedestrian flow relationships for indoor and outdoor walkways, signalized crosswalks as well as stairways in Mass Transit Railway (MTR). The speed-density-flow models developed for Hong Kong are similar to those developed for Singapore.

In order to develop pedestrian planning standards for Khulna city, it is required to conduct study on local pedestrian characteristics. The walking speed of pedestrians is the key factor for the design of pedestrian facilities. The objective of this paper is to develop the relationship between speed, flow and density for the pedestrians at walkways of urban areas in Khulna city. This result may be useful in the planning and design of pedestrian networks in Khulna and can be applied to other cities in Bangladesh.

2. DATA COLLECTION AND METHODOLOGY

This study was conducted in the Khulna Metropolitan City (KMC), Bangladesh. To conduct the speed studies in the concentrated areas, three walkways along the main streets (i.e. Day Night College Road, K.D. Ghosh Road and Khan-A-Sabur Road) were selected. Pedestrians were manually timed over a measured test length and speeds were then measured.

A photographic technique was used to collect data for the study on speed, flow, and density relationships. A digital video camera was used to record the situation at the observation sites. From the recordings, suitable time intervals were chosen to measure the walking speed, density and flow. The time taken by a pedestrian to traverse the test length was measured from the recording. The density was obtained by counting the number of pedestrians within the boundaries of the observation site. The flow was obtained by counting the number of pedestrians passing the centre line of the observation site within the time interval.

3. RESULTS AND DISCUSSIONS

As mentioned before, there were total three observation sites, one in the Day Night College Road (DNCR) at Daulatpur and other two in the K.D. Ghosh Road (KDGR) and Khan-A-Sabur Road (KSR) of Khulna Sadar. The details of each location are shown in Table 1.

Table 1: Detail of Study Locations

Location ID	Observation Sites	Width (m)	Length (m)
I	Day Night College Road (DNCR) (Walkway at the side of Century Super Market)	1.37	6.10
II	K.D. Ghosh Road (KDGR) (Walkway beside Hadit Park)	1.22	5.49
III	Khan-A-Sabur Road (KSR) (Walkway at the side of Jalil Tower)	1.27	3.56

The observed walking speed for the various types of pedestrian facilities in walkways were obtained from the video recording surveys and tabulated in Table 2.

Table 2: Results of the Walking Speed Study

Characteristics		Pedestrians				
		Men	Women	Combined	Young	Elderly
Mean walking speed (m/min)		53.52	50.77	52.83	51.07	47.67
Standard deviation		5.57	5.83	5.75	4.63	5.25
Range	High	68.90	63.00	68.90	61.10	56.60
	Low	40.10	41.40	40.10	43.20	39.00
Sample size		249	83	332	102	81

It is seen in Table 2 that the mean free-flow walking speed of these pedestrians was found to be 52.83 m/min. For a further breakdown by gender, it was found that the Khulna city's males generally walked faster than the females as their mean free-flow walking speeds are 53.52 m/min 50.77 m/min for males and females, respectively. The observed mean free-flow walking speed of Khulna City's pedestrians is comparatively slower than the Asian and American counterpart. The mean walking speeds for young pedestrians were found to be 51.07 m/min. The elderly, however, were found to have a lower walking speed – 47.67m/min. It is noted here that the term elderly is used subjectively. Any pedestrian who appeared to be over 60 years old was termed elderly. Table 3 shows the comparison of pedestrian walking speeds. It is noticed that the Asian and American pedestrian was found to have free-flow walking speed ranging from 73 m/min to 74 m/min and 79 m/min to 88 m/min, respectively.

Table 3: Comparison of Pedestrian Walking Speeds

Speed	Asia		United States		
	Japan (Murata, 1978)	Singapore (Tanaboiboon et al, 1986)	Navin & Wheeler (1969)	Fruin (1971)	Hoel (1968)
Mean Walking Speed (m/min)	73.0	74.0	79.0	81.0	88.0

The pedestrian data were analyzed by using the statistical software SPSSv11.5. The values of pedestrian flows, pedestrian speeds, pedestrian density and pedestrian area module were computed at each study location. Curves were plotted between speed and density, speed and flow, flow and density and flow and area module. These are presented in Figure 1 to Figure 3. For all locations, the analysis was done for one direction. The scattered plot of data points suggested a straight line relation between pedestrian speed and density; quadratic relationship between pedestrian flow and density, and pedestrian speed and flow and polynomial relationship between pedestrian flow and area module.

The general relationships used for the analysis are developed based on single-regime approach and are described as follows:

Pedestrian speed (μ) and density (k): $\mu = a - b \times k$ (1)

Pedestrian flow (q) and density (k): $q = a \times k - b \times k^2$ (2)

Pedestrian speed (μ) and flow (q): $q = \mu(a - \mu)/b$ (3)

Pedestrian flow (q) and module (M): $q = \dfrac{a}{M} - \dfrac{b}{M^2}$ (4)

Where, speed (μ) in m/min, density (k) in ped/m^2, flow (q) in ped/m/min and Area module (M) in m^2/ped

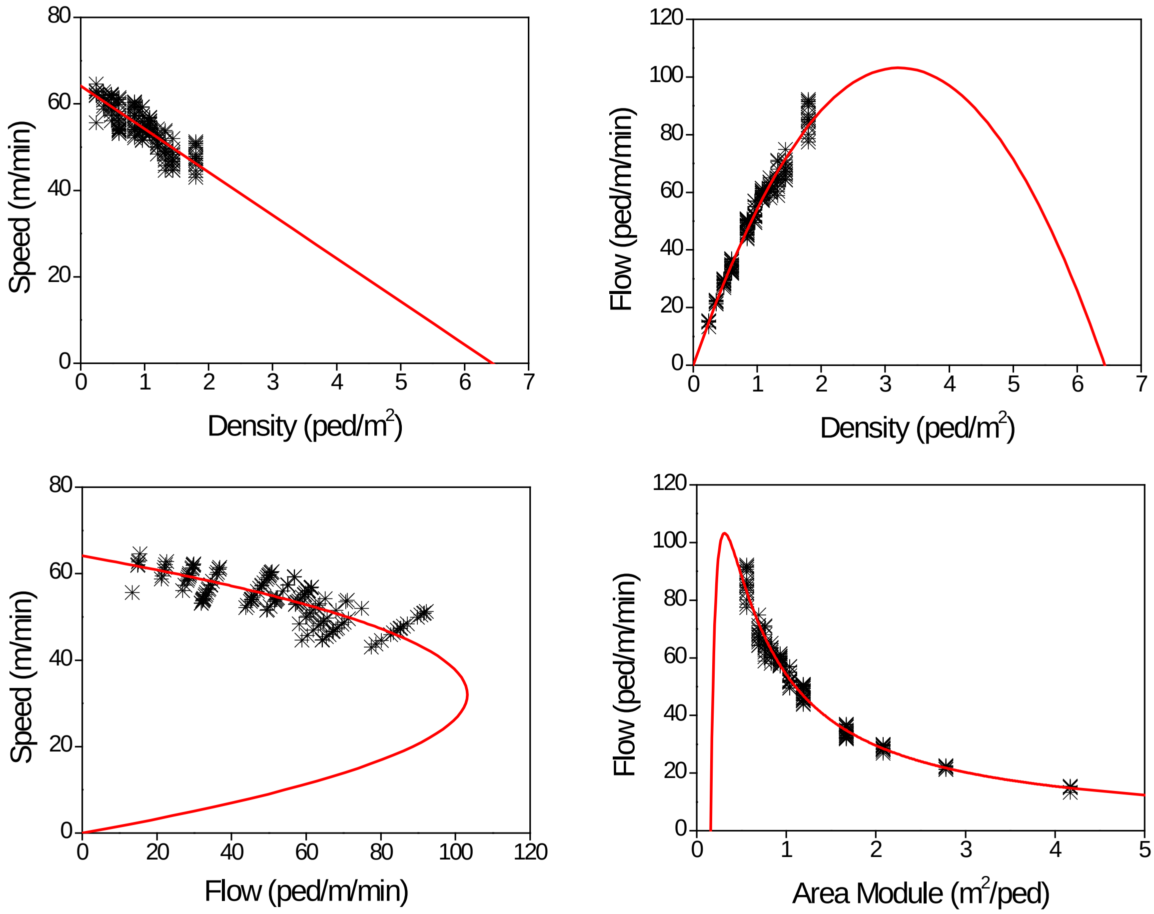

Figure 1: Pedestrian flow characteristics in Day Night College Road

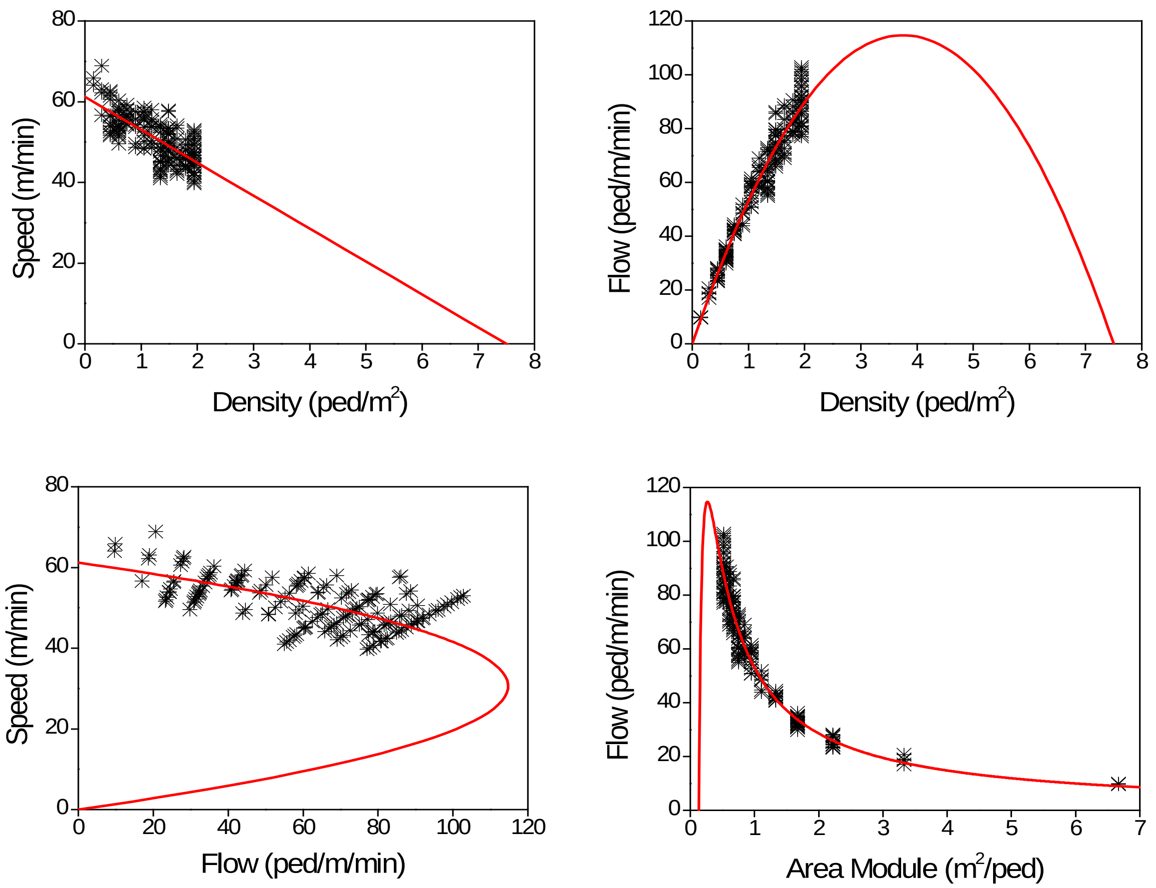

Figure 2: Pedestrian flow characteristics in K.D. Ghosh Road

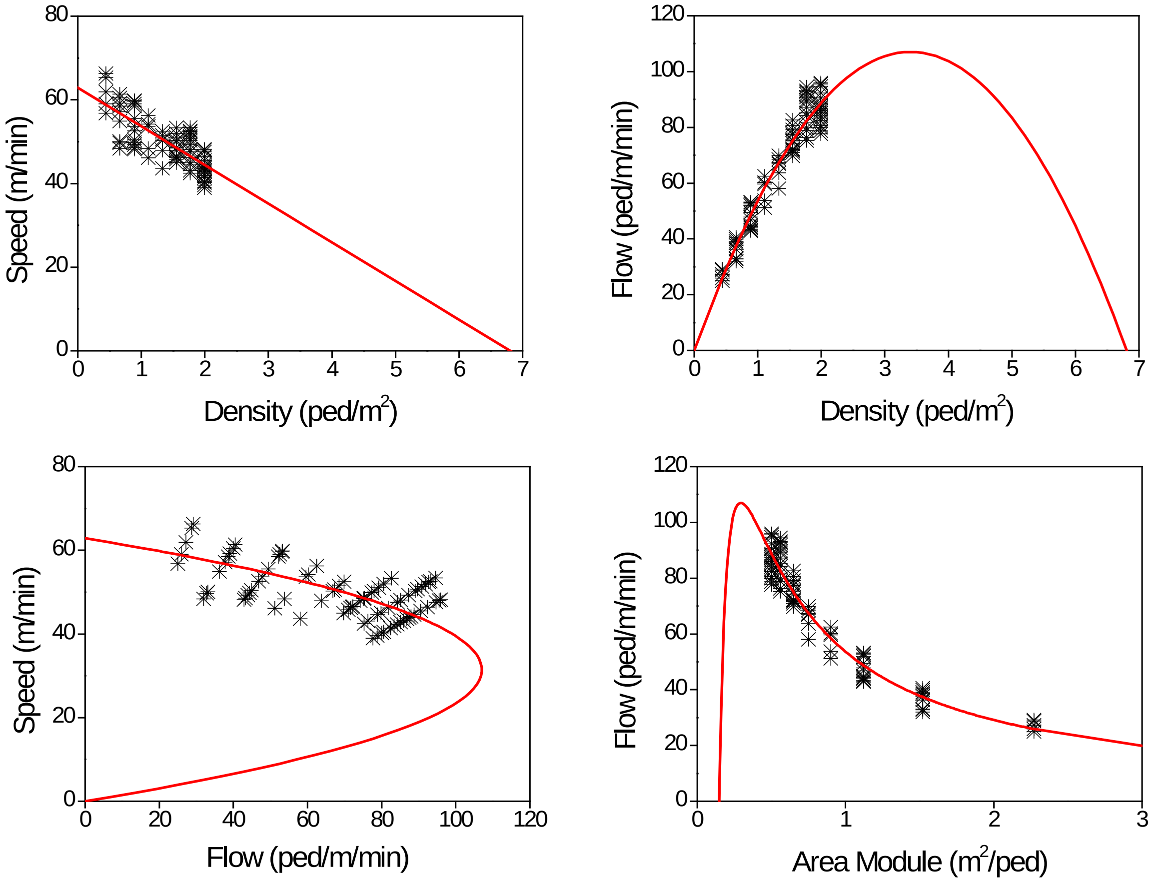

Figure 3: Pedestrian flow characteristics in Khan-A-Sabur Road

The relationships obtained from the analysis of data are presented in Table 4. The important flow characteristics estimated from the above relationships are given in Table 5. It is seen in Table 4 that the correlation coefficient R^2 varies between 0.33 and 0.97. These represent from fair to good fits to the data. It also suggests that instead of single-regime models, two to three-regime models might give better R^2 value. In the absence of extensive data these are not attempted in the present study.

Table 4: Relationship Developed between Different Pedestrian Flow Characteristics

Location ID	Site Location	Relationship	Model Equation	R^2 value
I	Day Night College Road (Walkway at the side of Century Super Market)	Speed-density	$\mu = 64.14 - 9.97k$	0.72
		Flow-density	$q = 64.14k - 9.97k^2$	0.97
		Flow-speed	$q = \mu(64.14 - \mu)/9.97$	0.58
		Flow-space	$q = 64.14/M - 9.97/M^2$	0.75
II	K.D. Ghosh Road (Walkway beside Hadit Park)	Speed-density	$\mu = 61.19 - 8.16k$	0.54
		Flow-density	$q = 61.19k - 8.16k^2$	0.94
		Flow-speed	$q = \mu(61.19 - \mu)/8.16$	0.33
		Flow-space	$q = 61.19/M - 8.16/M^2$	0.61
III	Khan-A-Sabur Road (Walkway at the side of Jalil Tower)	Speed-density	$\mu = 62.89 - 9.24k$	0.62
		Flow-density	$q = 62.89k - 9.24k^2$	0.94
		Flow-speed	$q = \mu(62.89 - \mu)/9.24$	0.40
		Flow-space	$q = 62.89/M - 9.24/M^2$	0.84

The mathematical relationships obtained in the present study are similar to those reported by Tanaboriboon et al. (1986) for the United States. The free-flow pedestrian speeds are found to be more than 60 m/min at all of the three locations. These speeds are highest at Location I (64.14 m/min). This location has highest width of the carriageway (1.37m) and the pedestrian face little frictions at this location. In case of Location II and III, the friction due to parked vehicles is present. Due to high pedestrian flow, many pedestrians use carriageway. The friction imposed by motorized vehicles is higher at Location III as compared to the Location I and II. This has resulted in a reduction in speed at the location II (61.19 m/min) and III (62.89 m/min) compared to Location I.

Table 5: Pedestrian Flow Characteristics at Different Study Locations

Location ID	Free-flow speed (μ_f), m/min	Jam density (k_j), ped/m^2	Maximum flow rate (q_{max}), ped/m/min	Area module (M) (m^2/ped.)	
				At q_{max}	Minimum
I	64.14	6.43	102.87	0.31	0.16
II	61.19	7.50	114.71	0.27	0.13
III	62.89	6.81	107.00	0.30	0.15

It is seen in Table 5 that the maximum density was observed at Location II, 7.50 ped/m^2 or 50 pedestrian in an area of 7 m^2. The minimum density was observed as 6.43 ped/m^2. It may be due to the level of frictions on roads and the roadway width. The higher level of friction and lesser roadway width make the pedestrians to use restricted road space, thus resulting in higher density. The maximum and minimum flow rates were observed as 114.71 ped./m/min or 6883 ped./h and 102.87 ped./m/min or 6172 ped./h. It is highest at location II and lowest at location I. Flow rate is lowest at location I due to open area and pedestrian freedom to use the space. The minimum area module was observed between 0.13 to 0.16 m^2/ped. and the area module at maximum flow rate is found between 0.27 and 0.31 m^2/ped. The comparison of different flow characteristics is shown in Table 6.

Table 6: Comparison of Pedestrian Flow Characteistics From Different Studies

Source	Country	Free-flow speed μ_f (m/min)	Traffic jam density k_j (ped./m^2)	Maximum flow rate (q_{max}), ped/m/min
Older (1968)	Britain	78.64	3.89	78
Fruin (1971)	United States	81.40	3.99	81
Tanaboriboon et al. (1986)	Singapore	73.90	4.83	89

It is seen in Table 6 that the free-flow speed computed in this study are lower than those observed in Britain, United States and Singapore. The maximum density (7.50 ped./m^2)observed in this study is higher than the observed density in Britain, United States and Singapore. The maximum flow rate observed in this study (114.71 ped./m/min) is higher than that of Britain (78 ped./m/min), United States (81 ped./m/min) and Singapore (89 ped./m/min). Because, Bangladeshi pedestrians require less personal space than others study.

4. CONCLUSIONS

This paper aims to investigate the pedestrian flow characteristics in the walkways of Khulna city. The variations of pedestrians mean walking speed with respect to age and gender were also observed. The results indicate that the pedestrian of Khulna city has a slower walking speed than the American and other Asian cities. However, the maximum flow rate obtained in this study is higher than that obtained in the Asian and Western counties. This study also shows that the characteristics of the location have effect on the pedestrian flow characteristics.

The relationships developed between different flow parameters i.e. speed, flow, density and area module are observed to be satisfactory to good. There is need to re-examine these relationships based on two-or three-regime models. The free-flow speeds of this study are found lower than the Asian and Western countries. The observed free-flow speed and densities are found proportional to each other. The increase in road friction also increases the jam density. Thus, the findings of this study may be useful to the planners and designers to design efficient, adequate and safer pedestrian facilities.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CURRENT CAR PARKING SITUATION IN KHULNA METOPOLITAN CITY: A CASE STUDY

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ABSTRACT

The increasing number of vehicles generates various parking problems such as insufficiency of parking spaces, illegal parking etc. This study explores the parking characteristics as well as the current car parking situation in the Central Business District (CBD) area of Khulna Metropolitan City. Data were collected by using license plate survey at two locations of on-street parking lots. The inventory on the parking supply revealed that the available parking spaces are sufficient. The degree of utilization of Location II is more than the Location I.In both of the locations, the supply exceeds the demand.This may be useful for the city planners.

Keywords: License Plate survey, parking accumulation, on-street parking, parking turnover, parking demand and supply

1. INTRODUCTION

Parking is a critical component of transportation policy especially for the large cities. As of today the commercial areas in every city are faced problem of parking congestion, accidents and environment pollution Previous studies (Kimber, 1984 & Hobbs, 1979) have shown that the capacity of the road network can be reduced considerably if parking facilities and locations are not selected and controlled properly. Planned parking facilities are required to improve roadway capacity and safety. It has been noted that on-street parking comprises roughly 60 to 90 percent of the requirements in small town and around 30 percent in urban areas (Afroze, 1991).

Khulna Metropolitan City (KMC) is growing fast with increasing population in Bangladesh. The present transportation scenario becomes hazardous due to heterogeneous characteristics and inefficient road network system. Due to the increasing population, the need for automobile oriented transport facilities is increasing excessively. In the year 2009, 4453 vehicles were registered in Khulna circle among those 57 were cars. The registered vehicles in the year 2009 are about 33 percent more than the year 2006. (www.brta.gov.bd, 2009). The registered vehicles are about 25 to 50 percent of the actual vehicles plying the roadways due to limited enforcement of registration regulations (ADB, 2006). It is roughly estimated that out of 8760 hours of a yea, the car runs on an average for only 400 hours which is only 4.5% of the available time (Kadiyali, 2004). So, parking is a serious concern and requires a carefully considered investigation. The traffic movement in Central Business District (CBD) is one of the important factors for traffic congestion because maximum number of vehicles arrives for different purpose in the city centre. In Khulna Metropolitan City (KMC), some roads are occupied by parked vehicles. As a result, the authority of KCC has to provide effective measures to improve the parking condition. As such, this study was carried out to investigate the existing on-street car parking situation as well as the parking characteristics in the city centre of Khulna Metropolitan City.

2. STUDY AREA AND METHODOLOGY

Surveys were carried out to collect the required data for the analysis of this study. Theses surveys were conducted to assess the parking demand, parking characteristics, parking accumulation and parking duration in the study area. Theses surveys were inventory of parking facilities, on-street parking observational survey and license plate survey.

2.1 Study Area

Khulna metropolitan city is the 3^{rd} largest city of Bangladesh which is located on the bank of the river Bhairab and at the southwest potion of the country. The city is confined between latitudes 22.49^0 North and longitudes 89.34^0 East and it covers an area of about 45.65 square kilometres with a population of about 1.5 million (Alamgir et al. 2005). Most of the part of this area are urbanized and well developed in infrastructures. Both motorized and non-motorized vehicles run on the busy streets at the same time, resulting in deteriorating the traffic congestion. The study areas for on-street parking were Khan-A-Sabur Road (In front of Hard Metal Gallery) and around Khulna new market. These are located in the main business point of the city as shown in Figure 1.

New Market (Near Gate No. 2) Khan-A-Sabur Road (In front of Hard Metal Gallery)

Figure 1 Location of study area

2.2 Data Collection

A series of survey were carried out to collect the required data for the analysis of this study. These surveys were carried out to assess the parking demand, parking characteristics, parking accumulation and parking duration within the study area. These surveys were inventory of parking facilities, on-street parking observational survey and license number plate survey.

2.2.1 Inventories of parking facilities

Inventories of on-street and off-street parking facilities were carried out to determine the availability of parking spaces in the Central Business Area (Khan-A-Sabur Road and around the New Market). On-street parking inventories were carried for marked spaces. Marked curb spaces were manually counted by a group of surveyors. The number of spaces was tabulated according to site. Curb parking inventory includes the location of spaces, parking orientation and the characteristics of the adjacent streets. Observation on the length of existing parking stalls was also made. Each marked stall was measured directly.

2.2.2 License Plate Survey

The main purpose of this survey is to gather information on the degree of utilization of parking facilities and demand of parking in the study locations. On-street parking survey was carried out on week day in sunny weather from 8:00 A.M. until 7:00 P.M. Two surveyors were assigned to collect the data for each location. The license number plates of parked vehicles, their location, time of arrival, and time of departure were checked and recorded by the surveyors assigned to monitor the vehicles in their respective location. The surveyors were revisited each space at every 30 minutes interval. Three digits of the plate numbers were recorded.

3. RESULTS & DISCUSSIONS - PARKING CHARACTERISTICS

The parking characteristics intend to determine the degree of parking utilization and the parking facilities. Parking inventory for on-street parking facilities is covered as well. Furthermore, the analysis of the utilization of parking facilities in both the locations is also presented to elaborate the present parking demand.

3.1 On-Street Parking Ordinances

On-street parking is commonly noticed in the central business area of Khulna Metropolitan City. Actually, there is no specific on-street parking ordinance enacted by Khulna City Corporation (KCC). The parking demands rarely reach at a problematic level. Consequently, the KCC does not make on-street parking ordinance.

3.2 Parking Inventory

The parking space is specified in terms of length, width, angle and use. The average length and width of parking stall at Location II is about 5.50 meters and 2.50 meters, respectively. But in case of Location I, the parking stall is unmarked. In both of the locations, the parking stall pattern is angular. A total of 34 and 14 parking stalls were recorded in Location I and Location II, respectively. It is mentioned that at present parking fee is not charged for both of the locations.

3.3 Parking Utilization

Parking utilization describes the degree of facility's usage at the existing conditions. The general utilization indicators are accumulation of vehicle, occupancy ratio, turnover rate, and duration. These are discussed in the following sections.

3.3.1 Parking Accumulation

This index shows the pattern and timing of vehicles on how they utilized the space. The pattern of accumulations is summarized in Table 1.

Table 1 Peak parking accumulation within the study area

Location ID	Location	Available Spaces (Nos.)	Peak Parking Accumulation (Nos.)	V/C Ratio
I	Around New Market	34	274	0.12
II	Khan-A-Sabur Road (Near Hard Metal Gallery)	14	88	0.16

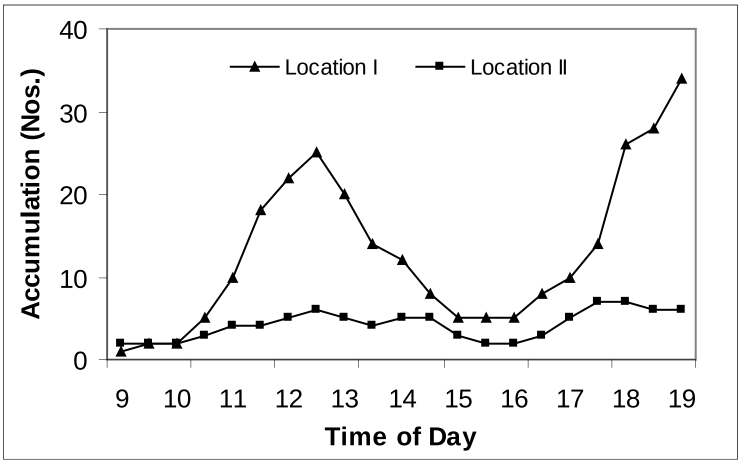

Figure 2 Hourly variations of the parked vehicles

The hourly variation in terms of the vehicles parked on both of the locations for on-street parking is as shown in Figure 2. It is seen that the accumulation reaches maximum by 1230hr and declines gradually and again becomes maximum at night by 1900hr for Location I. But in case of Location II, the accumulation reach maximum by 1230hr and decreases gradually and becomes maximum in the afternoon by 1730hr. The parking generators of these locations are mostly private and government offices. These offices generated trips for parking after the office hour (1700hr) and the visitors and shoppers usually visited these locations from 1700hr to 1900hr. Because of this, maximum accumulation occurs at 1900hr and 1800hr for Location I and Location II, respectively. It can be concluded that on-street parking in the above two locations is widely practiced.

3.3.2 Occupancy Ratio

Occupancy ratio or vehicle-capacity (v/c) ratio is another measure of parking utilization. A ratio of greater than one implies demand exceeds the available space supplied. It determines the degree of utilization and indicates the existing illegal parking. Table 2 shows the occupancy ratios with respect to time of observation. It is seen that the occupancy ratio is highest at 1900hr and 1800hr for Location I and Location II, respectively. Since the occupancy ratio is a function of the vehicles observed in the area so it follows the same pattern as that of parking accumulation. The average overall occupancy rate of on-street parking is 0.39 and 0.30 for Location I and Location II, respectively.

Table 2 Occupancy ratios of the observed locations

Location ID	Time of the day											
	9	10	11	12	13	14	15	16	17	18	19	Avg.
I	0.03	0.06	0.29	0.65	0.53	0.38	0.15	0.15	0.29	0.76	1.00	0.39
II	0.14	0.14	0.29	0.36	0.36	0.36	0.21	0.14	0.36	0.50	0.43	0.30

3.3.3 Parking Turnover Rate

Another measurement of parking facilities utilization is the parking turnover rate. This measurement reveals the number of vehicles utilizing the same stall over a given time period. Papacostas and Prevedours (1993) mentioned that a turnover rate of four or more during an 8-hour period is considered high. Table 3 shows the parking turnover rate of on-street parking. It is seen that the average turnover rate of on-street parking at location I and location II is 8.06 and 6.29 vehicles per stall per day, respectively. Both of the locations fall under high turnover rate. The overall turnover rate for on-street parking is 7.54 vehicles per stall per day.

Table 3 Parking turnover of on-street parking

Location ID	Available spaces (Nos.)	Observed vehicles (Nos.)	Turnover (vehicle/stall/day)
I	34	274	8.06
II	14	88	6.29
Total	48	362	7.54

3.3.4 Parking Duration

Parking duration is the time spends in the parking space. Parking duration measures the effective utilization of parking facilities needed for an effective planning and design of parking facilities. It depends on the purpose of trip and type of facilities. The on-street parking has an average duration of 1.01 and 1.10 hours for Location I and Location II, respectively as shown in Table 4.

Table 4 Average duration of on-street parking

Location ID	Available Spaces (Nos.)	Observed Vehicle (Nos.)	Space Hour used (Hr.)	Average duration (Hr./Veh.)
I	34	274	276.4	1.01
II	14	88	96.5	1.10

The results show that Location II has the higher average duration of parking than Location I. It can be concluded that motorists want to use the parking facilities in Location II. Because, they can stay more time to perform their business.

3.4 Parking Demand and Supply

Parking demand refers to the number of vehicles whose driver desired to park at specific location of the study area. It is usually expressed in the number of vehicles. One of the important factors that influence the parking demand is the drivers' destination. Parking generators are assumed to be nearest or within convenient walking distance from the parking space utilized. The summary of parking demands is presented in Table 5. The dynamic capacity of the on-street parking was estimated by using the Equation (1).

$$P_d = \left(\sum_n N \times T / D\right) * F$$

Where, P_d = Dynamic parking supply; N = Number of spaces of a given type and time restriction; T = Time that N spaces of a given type and time restriction are available during the study period (hr.), D = Average parking duration (hr./veh.); and F = Turnover factors. The factor has a value ranging from 0.85 to 0.95. For this study, it is assumed that the average effect of turnover would be 10%, therefore F should be 0.90. The summary of parking demand and supply is presented in Table 5. In case of Location I, out of 34 available spaces, 274 vehicles were observed for a period of 11 hours.

Table 5 Summary of Parking Demand and Supply

Location ID	Available Spaces (Nos.)	Average duration (Hr/Veh)	Parking Demand (Veh)	Parking Supply (Veh)	Surplus (Veh)	Deficiency (Veh)
I	34	1.01	274	333	59	-
II	14	1.10	88	126	38	-

In case of Location II, a total of 88 vehicles were parked in 14 available spaces in the study area. This does not mean that theses vehicles would park at the same time, rather, in over 11 hours all these vehicles parked. Table 5 also shows the comparison of surpluses for on street parking. Both of the parking lots show surpluses. This is an indication of being underutilized.

4. CONCLUSIONS

The parking surveys carried out in this study show that the dimension of parking stalls in Khulna city is about 5.50 m by 2.50 m for angular parking which is different from other Asian countries. The utilization rate of parking in Location I is higher than Location II. Moreover, the turnover rate in Location I is higher than the Location II. Generally, parking supply exceeds the demand. But in this study the parking demand does not exceed the capacity. The results of this study may be useful as a guideline for the city planners and traffic authorities to develop the parking planning system in Khulna city. A financial analysis may be conducted to evaluate the future parking demand.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

TRANSPORTATION SYSTEM MANAGEMENT: AN ASSESSMENT FOR IMPLEMENTATION OF GENERAL TOOLS IN DHAKA CITY

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ABSTRACT

Transportation System Management (TSM) is a package of short term measures to make the most productive and cost-effective use of existing transportation facilities, services and modes. For the implementation of such well recognized, cost-effective management tools in a city area demand assessment of the potentiality and the fulfillment of implementation pre-requirements. In this study, an attempt has been made to assess the possibilities and inherent weakness for implementation of different transportation system management tools in Dhaka city. The study revealed that there are only about 4 km one-way road which is only 0.31 percent of total road network of Dhaka City Corporation (DCC). The city has not any tidal flow operational road. About 86 percent of major X-junction are 4 phases and about 90 percent of T-junction are 3 phases. Among the newly signalized 59 intersections, 47 (80%) intersections phases are equal to its approaches. There is no coordinated signal system in the intersection of the city. There is very little options to introduce such tools for the huge lack of implementation requirements in the present development condition. Corresponding data has been collected from the field observation, road inventory and landuse survey and tried to justify with the prerequisite for the implementation of the tools. At the very outset, the prevailing transport and travel scenario and existing road network which is foundation of the implementation of TSM has discussed.

Keywords: *Transportation System Management, One Way Operation, Tidal Flow Operation, Signal Co-ordination,*

1. INTRODUCTION

Dhaka is one of the fast growing metropolitan cities with a highly dense and increasing population in the world. Haphazard urban expansion with a minimum attention to the living environment has been the most common scenario here and existing transportation system has become hazardous for the entire city system due to its inherent transport as well as road network deficiencies. Although, the city mainly depends on road-based transportation system, the amount of road network is far apart from the minimum requirements. Improving roadway capacity through application of traffic management tools is a crucial issue to minimize congestion and to increase mobility and accessibility. At present, in Dhaka city, there are only about 4 km one-way road which is only 0.31 percent of total road network of Dhaka City Corporation (DCC) (Mahmud, 2009). The city has not any tidal flow operational road. About 86 percent of major X-junction are 4 phases and about 90 percent of T-junction are 3 phases. Among the newly signalized 59 intersections, 47 (80%) intersections phases are equal to its approaches. There is no coordinated signal system in the intersection of the city. For the lack of interconnection between the side roads or for the absence of service road along the major road there is no closing of side-street at all. Providing exclusive bus-lane for the implementation of BRT is an apprehension issue in Dhaka city. For the implementation of different such tools in the present development condition of the city, the potentiality or the implementation requirements need to be assessed to check whether they are warranted or not in the context of Dhaka, demand a comprehensive assessment. In this study, an attempt has been made to assess the possibilities and inherent weakness for implementation of different transportation system management tools in Dhaka city. Correspondingly data has been collected from the field observation, road inventory and landuse survey and tried to justify with the prerequisite for the implementation of the tools. At the very outset, the prevailing transport and travel scenario and existing road network has been pointed out.

2. PREVAILING TRANSPORT AND TRAVELING SCENARIO IN DHAKA METROPOLITAN CITY

An assessment of the existing urban transport system of Dhaka revealed that it suffers from major constraints such as – fragmentation of organizational responsibility; inefficient regulatory frameworks; insufficient financial resources; poor allocation of road space; presence of too many low capacity and slow vehicles on major roads; poor traffic control, management and enforcement; underdeveloped public transport system; absence of adequate pedestrian facilities; poor linkage between land use planning and transport development and finally, lack of emphasis on environmentally sound and sustainable transport development.

Various surveys (STP 2004, DITS 1994, JBIC 2000 etc.) undertaken and their analysis revealed the following information about the city:

- The average household income in 2004 was Tk15,000 ($253) per month while 3½% of households was in the “high” income group exceeding Tk55,000 ($920) per month (STP 2004);
- The average number of persons per households in 2004 was 4.12 (STP 2004);
- Among households 7% either have or have access to a car, 4% owns motorcycle, 3% cycle rickshaw, 5% bicycle and 2% auto-rickshaw (STP 2004).
- Almost 84% of household do not have any sort of transport vehicles and depend on public transport and only 16% have some sort of vehicles (STP 2004).
- Among the important purposes (i) home-work 31%, (ii) home-education 25%, (iii) home-shopping and others 36%, and (iv) non-home based trips 8% (STP 2004).
- On average, each household undertook 8½ trips per day by all modes (STP 2004);
- At present, the number of trips that are generated per day is 21.98 million and after 20 years, the trip generation per day that is estimated is 159.63 million in 2024 (STP 2004).
- The average trip length was found to be 5.37 kilometers (STP 2004).
- Buses comprise a small proportion of vehicle numbers (11.5%) but carry about 77% of people (STP 2004);
- Rickshaws comprise 28% of all vehicles moving in the city and forms primary travel mode for 34% of all person trips (STP 2004);
- Proportion of trips made by walking is substantial, though according to STP around 22%, DITS estimated that 62%; and
- Road space occupied by rickshaws is 73%, and by cars, buses and tempo is 19.7%, 4.4%, and 0.4%, respectively (DITS, 1994).
- Among the modes, car occupies the highest space for carrying a person which is 75.8%. Rickshaw occupies 21.9%, Baby taxi 17.9%, Bus 8.7% and Tempo 5% (STP 2004).
- Auto-rickshaw are a low proportion of vehicles comprising less than 10% of travel;

3. EXISTING ROAD NETWORK OF DHAKA METROPOLITAN CITY

An important part of every town is its transport system and particularly its road system for affecting smooth and efficient movement of people and commodities. For good accessibility, the circulation system of a city should meet certain basic requirements. Firstly, the vehicle users should be able to move from one part of the city to the other easily, safely and efficiently. Secondly, the roads should be arranged in such a way so that they provide adequate access to every building for vehicles and pedestrians. To meet all these requirements a system of hierarchies in the road network is required, and this hierarchy results in a system which is composed of various types of roads designed for different types of movements according to the function, character and volume of traffic. A well-articulated road network system usually occupies 20 to 40 percent of urban area depending upon the size, function and character of the city (Chowdhury, 2001). There are only 9 percent of the total area are road space where’s pavement space only 6 percent in the DCC area of the 1286 km of road comprising 61 km primary, 108 km secondary, 221 km connector, 573 km local and rest narrow road as shown in Table 1).

Table 1: Different Types of Road Length, Pavement Area and Road Area of DCC

Classes of road	Length (km)	Percent	Pavement area (sq. km)	Percent	Percent of total land area	Road area (sq. km)	Percent	Percent of total land area
Primary	61.45	4.78	1.46	16.47	1.08	1.89	15.67	1.41
Secondary	108.20	8.41	1.86	21.05	1.39	2.41	19.94	1.80
Connector	221.35	17.21	1.68	19.04	1.25	2.47	20.42	1.84
Local	573.75	44.61	2.93	33.17	2.18	4.25	35.19	3.17
Narrow	321.27	24.98	0.91	10.28	0.68	1.06	8.78	0.79
Total	1286.02	100.00	8.84	100.00	6.59	12.09	100.00	9.01

Source: RMMS, 2004

In further analysis, it is found that there are only 107 kilometers road which width more than 24 meters in all over the city of Dhaka among the total 1286 km road. Indeed, there are only 45 km of road which pavement width more than 24 meter. Indeed, among the total road of the Dhaka City Corporation area, 821.61 km (64%) road width is more or equal to 4.75 meter but according to pavement width, that's are only 618.14 km (48%). So, although 203 km (16%) road width is more or equal to 4.75 meter but their pavement width is less than 4.75 meter, i.e. emergency vehicle could not be entered that pavement width road. So, this 203 km road could be made accessible by increasing the width of pavement of the road. But, other 464 km (36%) road pavement width could not be increased without demolishing existing road side development as road entire road width is less than that the desired level of accessibility but that's are almost impossible as this problem is started from very beginning of the planning of the city and both side roads is highly buildup by multi-storied building and huge densified with about 0.1 million people per sq. kilometer. Indeed, a city could not sustain with this huge so thin vein which is totally blocked or plugged or out of use.

In further analysis, it is found that there are only 2.15 km of road is available for 10,000 of population and pavement space available only 0.015 sq. Km (STP, 2004). The availability of major roads in terms of either km per thousand populations or km per square kilometer of area is too low as compared to the other cities of developing countries. According to the World Bank statistics in the report on road per capita in 9 cities of developing countries and 26 cities of developed countries is 0.5 and 4.5 meter respectively (Ingram and Liu, 1998). Whereas, in Dhaka City Corporation area, there are per capita road is only 0.0213 meters. This scarcity of the road length or area will be sustaining until the city remain. This is one of the fundamental inherent weaknesses of the city of Dhaka for their yield land use and transport panning.

From the above discussion and facts, it is found that the existing road in the entire city road network is not quite enough and is the one-forth of the minimum requirement of a modern city. Even, after the implementation of newly developed Strategic Transport Plan (STP) , total road network would be around 1413.67 km and road and pavement area would be 15 (11%) and 11 (8%) sq.km respectively. Besides this, the roads which are existing are not proper functioned and are not fully operational with full capacity for the causes of huge alignment, layout or orientation problems of the entire road network.

4. TRANSPORTATION SYSTEM MANAGEMENT

As the traffic on the existing road system in cities grows, congestion becomes a serious problem. Medium and long-term solution like widening roads, providing elevated fly-overs, constructing bypasses, and urban expressways are costly. Simple and inexpensive solutions can tide over the crisis for same time. Transportation System Management (TSM) is a package of short term measures to make the most productive and cost-effective use of existing transportation facilities, services and modes. TSM also embraces Travel Demand Management (TDM) and Traffic Supply Management (TSM) (Kadiyali, 2007).

4.1 Travel Demand Management

TDM techniques are aimed at reducing the traffic flows, especially during the peak hour. Some of the techniques commonly adopted are car pooling and other ride-sharing programmes, peripheral parking schemes, chartered buses (Institutional buses) to serve areas of trip origins to common work place, staggering of office hours and flexible time of work, Internal shuttle service in the CBD, parking restraint , road pricing , entry fee , priority for buses in traffic, restrictions on entry of trucks during day-time.

4.2 Traffic Supply Management

The fundamental approach in traffic management measures is to retain as much as possible existing pattern of streets but to alter the pattern of traffic movement on these, so that the most efficient use is made of the system. Some of the well-known traffic management measures are one-way streets, tidal-flow operations, restrictions on turning movements, reduction of signal phases, installation coordinated signals, restrictions on loading and waiting, exclusive bus-lanes, and closing side-streets (Kadiyali, 2007).

5. TSM: ASSESSMENT FOR IMPLEMENTATION IN DHAKA CITY

Most of the transport expert in the developing world, the term traffic management is frequently used to refer to and understood as parking control, control of side friction of the roads which are usually demand management approaches. So far, these factors have been given more priority and considerable efforts have been taken as well. On the contrary, supply management approaches are self enforcing, effective and low-cost measures that are not considered for eliminating the congestion problems in Dhaka city. Therefore, this study is undertaken on this management approaches with a view to investigate the applicability of these management regulations i.e. supply management. For the possibility assessment of the implementation of well recognized, cost effective above mentioned general transportation system management tool, effectiveness and pre-requisite of the implementation has been gathered from the differnt literature and manual. Correspondingly data has been collected from the field observation, road inventory and landuse survey and tried to justify with the prerequisite for the implementation of the tools. The observations are described in below:

5.1 One-way Streets

5.1.1 Purpose

As the name itself implies, one-way streets are those where traffic movement is permitted in only one direction. As a traffic management measure intended to improve traffic flow, increase the capacity and reduce the delays, one-way streets are known to yield beneficial results. They afford the most immediate and the least expensive method of alleviating the traffic conditions in a busy area (Kadiyali, 2007).

5.1.2 Advantages of one-way streets

The major advantages of one-way operations are reduction in the points of conflict, increased capacity, increased speed, facilitating the operation of a progressive signal system, improvement in parking facilities, elimination of dazzle and head-on collision as well as reduce of accident severity.

5.1.3 Implementation Requirements:

Most implementation requirements of the one way operation are suitable alternative roads (preferably grid-iron type or road network), access the impact on: Transit operations, Freight movement and Road side business and major traffic generators (Kadiyali, 2007).

5.1.4 Prospects in Dhaka city

In Dhaka city, there are only about 4 km one way road which is only 0.31 percent of total road network of DCC. Now question is, is there any option to introduce one way operational tool to other roads. According to the literature review which is pointed out above, for the implementation of one way operation of a city, first and foremost requirement is alternative or/and analogous couple/twin road for opposite directional vehicle. That is road network pattern should be developed in such a way that parallel and twin road must available for both directional vehicles. From the overall observation of Dhaka city road network, there are a very few parallel alternative or identical twin road where one way operational can be implemented. Indeed, the city road network is not developed considering this idea for future implementation of such tool to manage the demand of traffic. Even, the roads where one way operation is now being implemented, most of them are superimposed on road by executive force, not by traditional manner. Only, Indira and Khamar Bari road of Farm gate is parallel road which is fit for traditional one way operation. Besides this, Tikatoli Hatkhola road form as loop and Sonargaon road in front of Eastern plaza is a only single road which is fully unfit for one way operation. Now, if such question arise, is there any further option to introduce one way operation in the present road network of Dhaka city, obviously the answer is quite impossible, because of the pattern of road network is not is support the criteria of one way operation. If it is observed the Mirpur road, there is no matching road to go to Gabtoli from Azimpur. Similarly, there is no similar alternative road to move to Uttara from Shahabag except New Airport Road. The picture is also same fashion for the case of the Rokeya Sharani and DIT road. From the observation it is also found that, there is no twin or couple road any of the east-west connecting major road like Fazle Rabbi

Road, Topkhana Road, Inner Circular Road, Outer circular Road, Elephant Road, Pantha Path etc. which is fit for reverse directional vehicle. So, it is concluded that there is very few option to introduce the one way operation of the entire city road network of the mega city Dhaka and also there is not any cost effective solution to make possible for introducing such tool. This is a built in problem for the mega city of Dhaka and the city is detriment such a cost effective modern traffic management tool everlasting.

5.2 Tidal flow operation

One of the familiar characteristics of traffic flow on any street leading to the city centre is the imbalance in directional distribution of traffic during the peak hours. For instance, the morning peak results in a heavy preponderance of flow towards the city centre, whereas the evening peak brings in heavier flow away from the city centre. In either case, the street space provided for the opposing traffic will be found to be in excess. This phenomenon is commonly termed as "tidal flow"(Kadiyali, 2007).

5.2.1 Methods

The principle of tidal flow operation can be translated into practice in two ways:

(i) The first is to apportion a greater number of lanes in a multilane street to the in-bound traffic during the morning peak and similarly a great number of lanes to the out-bound traffic during the evening peak.

(ii) The second requires the existence of two separate streets parallel to each other and close to each other, so that the wider of the two can be set apart for the heavier traffic both during the morning peak and the evening peak. In this case, the two streets will operate as one-way streets.

5.2.2 Introduction Requirements

When considering the introduction of a proposed tidal-flow scheme it is important that the some conditions are met i.e. occurrences of flow imbalance should be periodic, distinct and predictable, the difference between the flows in the two directions is substantial or at least sufficient to justify an extra lane in the direction of major flow, for maximum efficiency of operation the number of traffic lanes allocated to each direction of travel should correspond as closely as possible to the ratio of the flows at the peak periods, route should be undivided, there must be adequate capacity at the end points of the reversible lane system to ensure easy transition of vehicles between the normal and reversible lane conditions (Kadiyali, 2007).

5.2.3 Prospects in Dhaka city

As described above, tidal flow operation is one of the most potential and cost effective tools to increase the capacity of a road as well as to reduce the congestion but there has some pre-requisite to introduce this tools. Generally, tidal flow operation is justified where 65 per cent or more of traffic moves in one direction during peak periods ((Kadiyali, 2007). It is also necessary that the remaining lanes for the lighter flow are adequate for that traffic. With a three-lane street, two lanes can be reserved for heavier flow and one lane for the smaller flow. In most of the modern city, large proportions of inbound traffic inter into the city in morning and vise versa in evening. To found this pattern of traffic flow, residential area and office area, industrial area, market area i.e. residential place and work place must be fully separated. People come to their work place from the peripheral residential area in morning and return to the resident in evening. It is evaluated that hat in Dhaka Metropolitan city; most of the lands are mixed use (Mahmud, 2009. There has not any complete residential area, pure industrial area, even pure commercial area. It is also described that the planned residential area, commercial are has been converted to also mixed area like Dhanmondi, Banani, Gulshan. Dhanmondi area was planned as a residential area and developed in such way. But, at present, for the lack of controlled, huge number of mixed used (8%), institution (5%), commercial market and industry (7%), is developed and turn as a fully mixed up area. Motijheel Centre Business District (CBD) area is developed as fully commercial area. But present situation is that commercial building is lower than the residential building, which is accounted for 38 percent and 40 percent, respectively as shown in Table 2 (DCC, 2006). Therefore, most of the developed land of this city (pre-planned, post planned, unplanned) are mixed landuse, there is no possibility to grow inbound and outbound peak flow in opposite direction in different times. That is, no possibility to introduce tidal flow operation in this road network of the Dhaka city too.

Table 2: Number of Building by Different Types of Use in Dhanmondi (Ward 49) and Motijheel (Ward 32)

Area	**Ward No. 49 Dhanmondi**		**Ward No. 32 Motijheel CBD**	
Building use type	Number	Percent	Number	Percent
Commercial/ Industry	156	7%	585	38%
Residential	1523	68%	619	40%
Education	123	5%	12	1%
Mixed	191	8%	238	15%
Medical /Health	53	2%	0	0%
Govt. Organization /Social Centre	85	4%	51	3%
Religious	13	1%	11	1%
Unknown/Under Construction	95	4%	25	2%
Club	7	0%	5	0%
Open Area	6	0%	0	0%
Total	2252	100%	1546	100%

5.3 Restrictions of Turning Movements

5.3.1 The problem posed by turning traffic

At a junction, the turning traffic includes left-turners and right-turners. Left-turning traffic does not usually obstruct traffic flows through the junctions, but right-turning traffic can cause serious loss of capacity. At times, right-turning traffic can lock the flow and bring the entire flow to a halt. One way of dealing with heavy right-turning traffic is to incorporate a separate right-turning phase in the signal scheme, or to introduce an early cut-off or late start arrangement. These schemes have their limitations and result in a long signal cycle. Another solution is to ban the turning movement altogether (Kadiyali, 2007).

Prohibited Right-Turning Movement: Prohibition of right-turning movement can be established only if the existing street system is capable of accommodating an alternative routing.

Prohibited Left Turning Movements: Left-turning movement is not obstructive to traffic and it is rare they are prohibited. However, such prohibition may be needed to provide a safe crossing for pedestrian traffic especially when the pedestrian traffic across the minor road is heavy.

5.3.2 Pre-requisite conditions for Prohibiting Turns

Some of the major pre-requisite for prohibiting turns are availability to suitable alternative diversion routes. There is also need to examine to possible impact of traffic diversion i.e. if the regulation, simply shift the problem to another junction, extra travel must be investigated and difficulties with the right turning mass transit. In addtion, the scheme should be coupled with continuous monitoring and enforcement (Kadiyali, 2007)

5.3.3 Prospects in Dhaka city

From the field observation and the project report on the intersection improvement, in Dhaka city, there are only two intersections viz. Banglamotor and Russel square intersection where's' exclusively right-turning has been banded. In Newmarket, Kawranbazar intersections, right-turn is being partially banded in several times. Besides this, for the lack of well orientation of road network as well as for the lack of alternative road, there have very few options to restrict right-turn of an intersection by accommodating the vehicle in alternative road in Dhaka city. Very few options which were available that's being unusable for the restriction of the road use by Dhaka University, BUET etc. Prohibition of right-turning movement is known to increase the saturation flow and the capacity of the junction but the city is deprived of this effective and efficient management system.

Left-turning movement prohibition is needed to provide a safe crossing for pedestrian traffic especially when the pedestrian traffic across the minor road is heavy. In Dhaka metropolitan city, around 60 percent of all trips are pedestrian (DITS, 1994) and pedestrian are the most vulnerable road users groups. Among the total road fatalities, 86 percent are pedestrian (Hoque et al., 2008). So, for safe pedestrian crossing, banning of left turn would be very effective for the Dhaka city, but for the lack of alternative road and lack of continuous monitoring and enforcement, there has very few options to introduce this tool.

5.4 Closing Side-streets/Access Control

5.4.1 Purpose

A main street may have a number of side-streets where the traffic may be very light. In such situations, it may be possible to close some of these side-streets without affecting adversely the traffic, and yet reap a number of benefits.

5.4.2 Advantages

Some of the advantages of closing side-streets are: Since interference from the traffic from side streets is eliminated, the speed increases and journey time reduces, for the same season as above, the accidents get reduced, if the side streets are too many and at close intervals, it is difficult to formulate a scheme for the progressive system of signals. A spacing of 275 m between signalized intersections is desirable and this necessitates the closure of all intermediate side streets. The side-streets which are closed can be utilized for parking of vehicles, if there is an acute shortage of parking space in the area. The side-streets, closed for traffic from the main streets, can he easily converted to a pedestrian precinct, thereby enhancing the safety, comfort and convenience of pedestrians (Kadiyali, 2007).

5.4.3 Prerequisites

The prerequisite of access control are alternative oad to diverse the traffic, internale connective road like service road, Before implementing this management scheme the relative advantages and disadvantages associated with closing side roads conditions are need to be studied, it must be noted that displaced through traffic will reappear elsewhere on the network and care must be taken to ensure that the result of street closure is not simply to move the problems to other locations (Kadiyali, 2007).

5.4.4 Prospects in Dhaka city

One of the major problems of Dhaka city roads are the uncontrolled access which reduce the mobility of the road as well as entire capacity of the road is that too much access roads (around 10 side road per km in the main road) which are directly connected with the major road as shown in Table 3. No service road has been provided either side of the road except some part of Uttara. So the access or connector roads get direct access to the road and reduce the desired mobility of the city road.

For reducing the interruption of traffic flow or eliminating the interference from the traffic from side streets, closing of side road is a very effective and demandable solution. Closing some of these side-streets without affecting adversely the traffic requires interconnection between two side roads or service road facilities for diverging the vehicles. But, from the field observation it is observed that in Dhaka city, the side access road has not sufficient interconnection by which vehicle can diverge from one road to another and finally come to the main road from a particular point. Besides this, there has not sufficient service road by the side of main road. Most of the case, side roads are perpendicularly connected with the main road and closes as a dead end.

Even there has not sufficient space in the side of main road by which two accesses can merge in a single road and will act as an access road. The root of this problem is the lack of planning and guidance of development. At the time of planning road, there has not provide the layout of access road or even did not locate the access merging point of the main street. So, frequent accesses are joined with the main road from different directions, angles and the adjacent spaces are highly developed. So, it is almost impossible to close side streets as it is form from the very beginning of the planning.

5.5 Reduction of Signal Phases

5.5.1 Purpose and Advantages

Intersection is the most complicated and complex part of the entire road network. The productivity/ capacity of a road is proportionally depends on the productivity of the approach of intersections of that road. Therefore, intersection dictates the overall performance of the road network. The road network or approach road will be more productive when its intersection signal phase will be minimum (Kadiyali, 2007).

Table 3: Access Density of Major Roads in Dhaka City (Source: Field Survey, 2007)

DIT Road						
From	**To**	**Length (m)**	**Access Road**		**Access per KM**	
			East side	**West Side**	**East Side**	**West Side**
Mouchack	Rampura Bridge	2739	26	20	9.49	7.30
Rampur Bridge	Natun Bazar	3362	36	33	10.71	9.82
Natun Bazar int.	Kuril Bisaw Road Intersection	2719	27	19	9.93	6.99
Mouchack	**Kuril Bisaw Road Intersection**	**8819**	**89**	**72**	**10.09**	**8.16**
Airport Road						
From	**To**	**Length (m)**	**Access Road**		**Access per KM**	
			East side	**West Side**	**East side**	**West Side**
Shabagh	Firmgate	2340	12	16	5.13	6.84
Firmagate	Mohakhali	2817	18	17	6.39	6.03
Mohakhali	Kakoli Intersection	1812	7	2	3.86	1.10
Kakoli Intersection	Kuril Bishaw Road Intersection	4041	14	9	3.46	2.23
Kuril Bishaw Road Intersection	Airport Intersection	3574	7	9	1.96	2.52
Airport Intersection	Abdullahpur	3628	13	14	3.58	3.86
Shabagh	**Abdullahpur**	**18212**	**71.00**	**67.00**	**3.90**	**3.68**
Mirpur road						
From	**To**	**Length (m)**	**Access Road**		**Access per KM**	
			East side	**West Side**	**East side**	**West Side**
Newmarket	Kalabagan Over bridge	1879	17	18	9.05	9.58
Kalabagan Over bridge	Manik Mia Avenue	1230	10	9	8.13	7.32
Manik Mia Avenue	Shamoli Over bridge	2025	13	17	6.42	8.40
Shamoli Over bridge	Technical More	1655	13	12	7.85	7.25
Technical More	Mirpur Bridge	1719	14	14	8.14	8.14
Newmarket	**Mirpur Bridge**	8508	67.00	70.00	7.87	8.23
Rokeya Sharani						
From	**To**	**Length (m)**	**Access Road**		**Access per KM**	
			East side	**West Side**	**East side**	**West Side**
Agargaon road	Mirpur 10 RA	3579	20	17	5.59	4.75
Kathal bagan	Agargaon road	2060	8	8	3.88	3.88
Kathal Bagan	**Mirpur 10 RA**	5639	28	25	4.97	4.43

5.5.2 Prerequisites

The availability of altenative road to diverse the traffic and link road for connectivity are the prerequisities for reduction of signal phases.

5.5.3 Prospects in Dhaka city

To observe the possibility of the reduction of signal phase, number of intersection phase and corner widening data has been collected from the field survey (Field Survey, 2007). The survey results are shown in Table 4.

Table 4: Major Intersection, Phase, Operational Type and Corner Widening in Dhaka City

Road	Major Intersections	Type	Phase	Operational Type	Corner Widening
Mirpur Road	Azimpur Intersection	X-junction	4	Signalized	nil
	New Market	X-junction	4	Signalized	3
	Elephant Road	T-junction	2	Signalized	2
	City College (Rd. No 2)	T-junction	2	Unsignalized	Nil
	Russel Square	T-junction	2	Signalized	
	Manik Mia Avenue	T-junction	3	Signalized	2
	Asad Gate	T-junction	3	Unsignalized	1
	Lake Road	X-junction	4	Unsignalized	
	Agargaon Road	T-junction	3	Unsignalized	2
	Technical Mor	T-junction	3	Signalized	
	Beribadh Road	T-junction	3	Unsignalized	nil
Pragati Sharani	Mouchack Intersection	T-Junction	3	Signalized	2
	Saidabad DIT Road Int.	T-Junction	3	Signalized	1
	Natun Bazar Intersection.	T-Junction	3	Unsignalized	2
	Bashundhara Junction	T-Junction	3	Unsignalized	0
Airport Road	Shabagh Intersectin	X-junction	4	Signalized	4
	Hotel Sheraton	T-juncton	3	Signalized	2
	Bangla Motor	X-junction	2	Signalized	2
	Kawranbazar	X-junction	4	Signalized	Roundabout
	Rangs Bijoy Sharani	T-juncton	3	Signalized	2
	Jahangir Gate int.	T-junction	3	Signalized	0
	Kuril Bishaw road int.	T-juncton	3	Signalized	2
S.C.M. Ali & S. Tazuddin Road	Matsha Bhaban	X-junction	4	Signalized	2
	Kakrail Mosque	Y-junction	3	Signalized	3
	Maghbazar	X-junction	4	Signalized	0
	Pantha Path	T-juncton	3	Signalized	2
	Mohakhali	Y-juncton	3	Signalized	1
Kakrail Road	Santinagar	T-juncton	3	Signalized	2
	Kakrail	X-juncton	4	Signalized	2
	Malibagh	T-juncton	3	Signalized	2
Some other major Intersection	Palashi	X-junction	5	Signalized	0
	Paltan	X-junction	4	Signalized	2
	Dainik Bangla	X-junction	4	Signalized	2
	Kataban	X-junction	4	Signalized	0

Source: Field Survey, 2007

From Table 4, it is seen that about 86 percent of X-junction are 4 phases and about 90 percent of T-junction are 3 phases. Only the Banglamotor X-intersection and New elephant road to Mirpur road T- intersections’ 2 and 1 phases respectively could be reduced for the supportive road of Panthopath road and Sonargaon road. Besides this, some intersections like Shahabag intersection phase can be reduce, but the alternative Nilkhet road is closed for public transport for the causes of Dhaka University area.

In recent year, new traffic signal has been installed in 59 major intersections in Dhaka city by the Dhaka Transport Coordination Board (DTCB). Among the newly signalized intersection, 31 intersections are X-intersection, 26 are T-intersection and 2 intersections are multi-leg intersection. Among the 31 intersections are X-intersection, 22 (71 %) are 4 phases and 8 (26%) are 3 phases and only one intersection are 2 phases. On the other hand, among the 26 T-intersections, 23 intersections are 3 phases and other 3 are 2 phases. Both of the multi-leg intersections are 5 phases. However, among the total 59 intersections, 47 (80%) intersections phases are equal to its approaches. Indeed, for the causes of 4 phases every X-junctions and 3 phases T-junctions, the each approach road loses their capacity ¾ times and 2/3 times respectively. For the lack of alternative road, or link road or for the lack of east-west connectivity as well as for the lack of proportionality between north-south and east-west connective road, there is very few opportunity of reducing signal phases of the major busy intersections. Therefore, this lose will continue until the alternative road is constructed to diverse right turn traffic into other road. However, it is not only very difficult but also almost impossible for such an over populated unplanned built-up city. So, most of the cases, for the lack of alternative road even secondary road, there is not possible to reduce the signal phase by diverting the traffic into other roads. Indeed, the city is not built considering such opportunity and there has not provided any option to fulfill the requirements by developing new infrastructure or road in the city areas. So, these unproductive intersections would be sustained until the city life.

5.6 Signal Coordinated

5.6.1 Purpose

For uninterrupted movement of traffic or to pass the maximum amount traffic without enforced halts or to have minimum overall delay to traffic streams both in main and side road and to prevent the queue of vehicles at one intersection from extending and reaching the next intersection, coordination between the signal is one of the fundamental tools which is widely used in various cities in the world.

5.6.2 Prerequisites

Most of the system operates efficiency where the blocks are of equal length and traffic motion is free and smooth in the link road between the two intersections (Kadiyali, 2007).

5.6.3 Prospects in Dhaka city

For the causes of uncoordinated signal design, a significant time of delay has been occurred in different intersection of different route in the city. To show the operational impact of uncoordinated signal, delay time at peck period has been counted in the major intersection in different routes of the city.

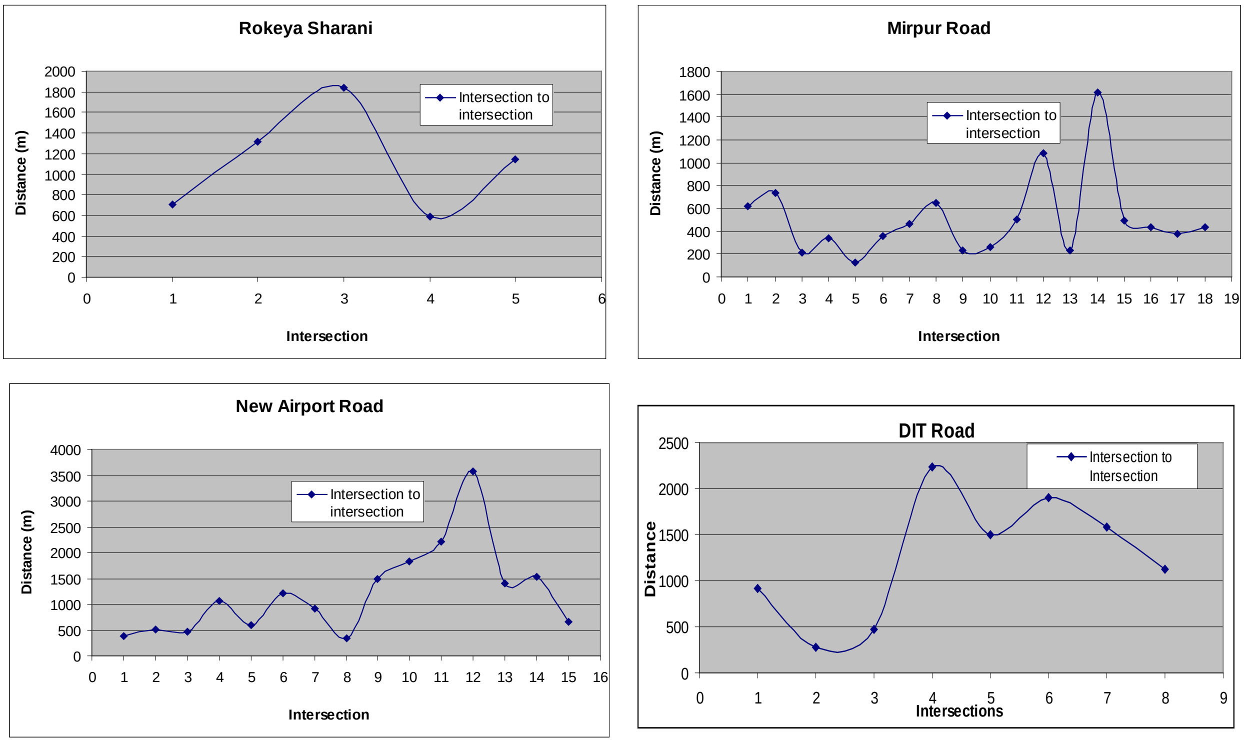

Figure 2: Distance between Intersection to Intersection in Different Roads

As discussed earlier, this delay could be minimized by co-ordinated signal system. But, for the prohibition of co-ordinated signal system, there must fulfill some prerequisite condition. To evaluate the possibility of the installation of coordinated signal under considering this requirement, intersection to intersection data has been collected using Google map and field survey, 2007. As the road network in the city is not any particular pattern and the difference between the two major roads is not chronological, the distance between the two intersections is varying on after another. So, it is difficult to provide coordinated signal in the intersection of the city. Figure 2 presents the intersection to intersection distance in major road in Dhaka metropolitan city.

Smooth discipline traffic flow is also one of the fundamental requirements for signal co-ordination. But, unfortunately for the causes of huge pedestrian flow on main road, random crossing, frequent access, mix traffic particularly rickshaw with the main stream flow interrupt the traffic in all of the sections of the main road. However, if the geometric configuration allows implementing coordinated signal, it will impossible to operate properly for the causes of such miasmas indiscipline traffic operational characteristics.

5.7 Exclusive Bus Lanes

5.7.1 Purpose and Requirements

A recent innovation in traffic management practice in some of the cities is to reserve a lane of the carriageway exclusively for but traffic. This is possibly only in situations where the carriageway is of adequate width and a lane can be easily spared for the buses. This implies that there should be at least three lanes in each direction. For reasons of convenience of alighting and embarking passengers at the curb, the exclusive lane has to be adjacent to the curb.

Exclusive bus lanes running against heavy one-way flow (Contra-flow lanes) are also very common. One experience suggests that such an arrangement nearly halves the journey time. To be successful, the bus-lanes should be created for a good length of the road instead of in small bits (Kadiyali, 2007).

5.7.2 Prospects in Dhaka city

Bus priority measures are a cheap and easy way to provide some aid to bus services. The journey time can be considerably reduced and bus journey can be made more attractive. Regularity of the buses can be improved. Viewed in the present context of traffic dilemma caused by the increase in car ownership and the need to patronize public transport, this is a welcome strategy. To be successful, the bus-lanes should be created for a good length of the road instead of small portion of the route. An effective enforcement policy is a prime requisite for safety. Satisfactory signing and marking of the lanes and adequate publicity are also needed. Recessed bus bays must be provided wherever reserved bus lanes are to be introduced.

In some of the advanced countries, as a further measure of patronage of the buses, traffic signals have been installed with special bus detectors at street intersections. Thereby it is possible to give preferential treatment for buses at traffic signals, minimizing delays (Kadiyali, 2007). But, unfortunately, from the road network data analysis coupled with road inventory survey as well as field observation it is revealed that in Dhaka metropolitan city, there is a minimum scope to provide exclusive bus lane in present road network.

In addition, separate bus lane is the first and foremost requirement for implementation of Bus Rapid Transit (BRT) which required sufficient road width in the route. But, after that, it is very difficult to provide separate lane for BRT in different segment of road network. Mainly in old Dhaka road segment like North-South road and English road required huge space acquisition for road widening for successful implementation of BRT. Unfortunately, these roads are in the old Dhaka and roadside as well as entire areas are highly haphazardly densified and widening of road is extremely difficult.

5.8 Restrictions on Loading and Waiting

5.8.1 Purpose and Requirements

Restriction on loading and waiting from the main stream requires sufficient bus-bays. Properly designed bus-bays recessed into the curb facilitate loading and unloading of passengers without the vehicles blocking the stream of traffic on the carriageway.

5.8.2 Prospects in Dhaka city

It is real to fact that in Dhaka city, there has not single bus bay in the core area. Only few bays are available in the cantonment road and airport road after Mohakhali. Even, few passenger shelter which are constructed by DCC, almost all of the passenger shelter just on footpath without any bus waiting or stopping facilities. Therefore, all of the passenger vehicles even trucks are loading or unloading standing on the main carriageway, which reducing almost one lane of the road way capacity. Even, all the ticket counters of the newly introducing non-stop modern bus service of Dhaka's road are on the footpath and alighting and boarding of passenger done standing on the main road. Real fact is that, at present condition, it is nearly impossible to construct bus bay on road side, as most of the side of the road are fully buildup without sufficient setback.

Another fact is that, most of the stoppages of the vehicle are on the intersection or just approach of the intersection. Where as the stoppage should not be located too close to the intersections. A minimum distance of 75 m from the intersections in desirable for urban situation and for rural locations a distance of 300 m is desirable (Kadiyali, 2007). But, it is open to all that most of the bus stoppage are just on the junction clear way, even in many case the than within the junction influencing area. At present haphazard densified condition, there has not any scope without major demolishing to provide well designed loading and unloading facility and ensure of proper use of that facility without massive strict enforcement. However, this problem is also a built in problem, which loses about one third of the capacity of the main road of Dhaka City.

5.9 Increasing Roadway Space though Demand Management

5.9.1 Requirements and Prospects in Dhaka city

According Bangladesh Road Transport Authority (BRTA, 2008) the total number of motorized vehicles in the city now stands at a staggering 1.05 million. From the vehicle register database of BRTA it is found that in Dhaka city there are 412,540 registered vehicles excluding rickshaw in 2006. There is no exact statistics in DCC about the number of rickshaw. One says that the registered rickshaw is about 80 thousands but it is argued that almost 0.4 million rickshaws are operating in Dhaka Metropolitan city.

The length of roads, pavement areas and road areas in considering all roads and accessible roads separately are available in the city for the case of entire, 70 percent and 50 percent registered motorized vehicles on road are provided in the Table 5 (Mahmud, 2009).

From Table 6, it is found that there are only 5.33 meters of road is available if all the registered motorized vehicles are coming on road at a time and if at least 50 percent of vehicles come into the road then they will get only 10.66 meters of road. In the case of accessible road, these length only 4.86 and 9.72 meters respectively and this are exactly operational. In terms of pavement space, the available areas are 17.9 sq. m, 25.57 sq.m and 35.8 sq. m. for the case of all, 70 percent and 50 percent on road registered motorized vehicles. That is, if it is consider that 50 percent of registered vehicles are non-operational or out of Dhaka or off street parking and other 50 percent of vehicles on road, then each vehicle will get only 9.72 meter road length and 35.8 sq.m pavement area on the city road network as shown in (Table 5). Whereas only for resting situation, on an average 6 to 7 meter road length and 12-18 sq. m. roads space is required (Kadiyali, 2007) and for free flow condition there will required more length and space. In this situation, congestion is must and that is a real fact in the daily life on our city road.

Table 5: The Length of Roads, Pavement Areas and Road Areas are Available for Motorized Vehicles

All Road				**Accessible Road**			
Vehicle	**Length per vehicle (m)**	**Pavement area per vehicle (sq. m)**	**Road area per vehicle (sq. m.)**	**Vehicle**	**Length per vehicle (m)**	**Pavement area per vehicle (sq. m)**	**Road area per vehicle (sq. m.)**
AllVehcie	5.33	21.44	29.31	All Vehicle	4.86	17.90	25.50
30% less Vehicle	7.62	30.63	41.87	30% less Vehicle	6.95	25.57	36.43
50% less Vehicle	10.67	42.88	58.62	50% less Vehicle	9.72	35.79	51.00

Source: RMMS2004, BRTA 2008, Mahmud 2009

On the other hand, every month around 3,000 new vehicles hit the road contributing to increasing burden on road and severe traffic congestion. This is also badly hampering the average speed as well as traffic operational condition of the vehicles.

6. CONCLUSIONS

At present, in Dhaka city, there are only about 4 km one-way road which is only 0.31 percent of total road network of DCC. There are very little options to introduce the one way operation in other road for the lack of alternative or twin road in the city. The city has not any tidal flow operational road and it is almost impossible to introduce such tool for the mixed land use of the entire city. For the lack of well orientation of road network as well as alternative road, there have very few options to restrict right-turn of an intersection by accommodating the vehicle in alternative road in Dhaka city. About 86 percent of major X-junction are 4 phases and about 90 percent of T-junction are 3 phases. Among the newly signalized 59 intersections, 47 (80%) intersections phases are equal to its approaches. For the lack of alternative road, or link road or for the lack of east-west connectivity there is very few opportunity of reducing signal phases of the major busy intersections. It is also difficult to provide coordinated signal in the intersection of the city. For the lack of interconnection between the side roads or for the absence of service road it is also difficult to close the side street. At present condition, it is nearly impossible to construct bus bay on road side, as most of the side of the road are fully buildup without sufficient setback. Providing exclusive bus-lane for the implementation of BRT is an apprehension issue in Dhaka city. There is also minimum scope to provide exclusive bus lane in present road network. The underlying reasons behind this failure are found in this study as the inherent weakness in the land use and transportation planning and the consequent haphazard uncontrolled development. This is a built in problem for the mega city of Dhaka and the city is detriment such a cost effective modern traffic management tool everlasting.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DEFINE THE PROBLEMS ASSOCIATE WITH AN EXISTING INTERSECTION & DESIGN IT TO AN IDEAL INTERSECTION

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ABSTRACT

The study to evaluate the condition of our study intersection (Polashi Intersection) data were collected manually to volume study & speed study.The condition of the intersection was evaluated by site inspection. From the data & inspection it can be said that mainly congestion occurs due to lack of maintanence & some faulty constructions.Signal light,signs,road marking,drainage systems either faulty or not working,From the study it is noticed that if we do any correction in exixting design or desing the interesction as a another type of intersection then congestion can be removed to far extend.

Keywords: *Intersection ,congestion ,existing problems, modification ,solution.*

1. INTRODUCTION

An intersection is the area where two or more streets join or crossat-grade. The intersection includes the areas needed for all modes oftravel: pedestrian, bicycle, motor vehicle, and transit. Thus, theintersection includes not only the pavement area, but typically theadjacent sidewalks and pedestrian curb cut ramps. The intersection is defined as encompassing all alterations (for example, turning lanes) to theotherwise typical cross-sections of the intersecting streets.

2. METHODOLOGY

2.1 Study Area:

The Polashi Intersection is located at the junction of five roads .One is directed towards BUET,one towards Dhaka University,one towards Lalbagh,one towards Nilkhet & one towards Azimpur.The intersection is a basic 5-way & simple intersection with free movement in any direction.It is complicated by the fact that all the street segments curve as they approach the intersection.

QuickTime™ and a
decompressor
are needed to see this picture.

Figure 1: Polashi Intersection (Intersection of consideration)

2.2 Process of our Intersection analysis

volume study &
speed study
visiting site to identify problems

***2.3* Method of counting**

manual counting method
Direct method.

Data is collected by using hand tally.

2.3.1 Advantages of direct method

Besides traffic volume, vehicle classification and turning point can be obtained data can be used immediately after collection.

2.3.2 Counting periods

short duration count at peak period is conducted for operational studies including those for roadway improvement. So, we are taking 1 hour count period for this study.

2.4 Factors affecting traffic volume

Different types of vehicle within the traffic stream make it difficult to express flow in terms of vehicle. Flow value is calculated by converting all the vehicles to a common type usually the passenger car. It is called the passenger car unit/equivalent (PCU/PCE)

3.RESULTS:

3.1 Volume Study

Traffic Counting Summary Sheet
(Directional hourly traffic volume)

Direction: Bakshibazar to Palashi   Day: Thursday   weather: sunny   Date: 10-16-11

Time Pack	Time	1	2	3	4	5	Total Traffic/ Time pack
		Car, Jeep, Taxi, Micro	NMT	Other Trucks, Pickups	3 Wheelers	Motorcycle	
1	15:30 to 15:35	4	0	5	2	1	**12**
2	15:35 to 15:40	8	2	4	4	1	**19**
3	15:40 to 15:45	1	0	6	10	4	**21**
4	15:45 to 15:50	7	4	2	2	5	**20**
5	15:50 to 15:55	4	0	2	0	2	**8**
6	15:55 to 16:00	6	0	7	4	4	**21**
7	16:00 to 16:05	1	2	5	3	7	**18**
8	16:05 to 16:10	4	4	6	3	0	**17**
9	16:10 to 16:15	1	4	11	0	0	**16**
10	16:15 to 16:20	7	0	13	2	2	**24**
11	16:20 to 16:25	7	2	9	6	0	**24**
12	16:25 to 16:30	2	1	7	0	4	**14**
Total Vehicle/ Type		**52**	**19**	**77**	**36**	**30**	

Direction: Palashi to Bakshibazar   Day: Thursday   weather: sunny   Date: 10-16-11

Time Pack	Time	1	2	3	4	5	Total Traffic/ Time pack
		Car, Jeep, Taxi, Micro	NMT	Other Trucks, Pickups	3 Wheelers	Motorcycle	
1	15:30 to 15:35	22	115	0	2	9	**148**
2	15:35 to 15:40	21	111	3	3	11	**149**
3	15:40 to 15:45	16	125	3	3	9	**156**
4	15:45 to 15:50	18	128	2	2	8	**158**
5	15:50 to 15:55	19	122	1	4	11	**157**
6	15:55 to 16:00	27	110	3	3	12	**155**
7	16:00 to 16:05	22	108	2	3	12	**147**
8	16:05 to 16:10	26	117	2	2	13	**160**
9	16:10 to 16:15	12	127	0	1	18	**158**
10	16:15 to 16:20	18	138	1	0	17	**174**
11	16:20 to 16:25	21	124	2	2	16	**165**
12	16:25 to 16:30	18	121	1	2	12	**154**
Total Vehicle/ Type		**240**	**1446**	**20**	**27**	**148**	

Direction: Azimpur to Palashi Day: Thursday weather: sunny Date: 10-16-11

Time Pack	Time	1	2	3	4	5	6	**Total Traffic/ Time pack**
		Car, Jeep, Taxi, Micro	NMT	Bus	Other Trucks, Pickups	3 Wheelers	Motorcycle	
1	15:30 to 15:35	18	71	2	5	2	1	**99**
2	15:35 to 15:40	18	116	2	4	4	1	**145**
3	15:40 to 15:45	10	124	1	6	10	4	**155**
4	15:45 to 15:50	19	78	1	2	2	5	**107**
5	15:50 to 15:55	23	137	0	2	0	2	**164**
6	15:55 to 16:00	22	122	2	7	4	4	**161**
7	16:00 to 16:05	21	118	2	5	3	7	**156**
8	16:05 to 16:10	15	114	3	6	3	0	**141**
9	16:10 to 16:15	17	119	2	11	0	0	**149**
10	16:15 to 16:20	13	123	1	13	2	2	**154**
11	16:20 to 16:25	12	109	0	9	6	0	**136**
12	16:25 to 16:30	11	110	2	7	0	4	**134**
Total Vehicle/ Type		**199**	**1341**	**18**	**77**	**36**	**30**	

Direction: Dhaka University to Palashi Day: Thursday weather: sunny Date: 10-16-11

Time Pack	Time	1	2	3	4	**Total Traffic/ Time pack**
		Car, Jeep, Taxi, Micro	NMT	3 Wheelers	Motorcycle	
1	15:30 to 15:35	23	36	6	6	**71**
2	15:35 to 15:40	27	38	12	7	**84**
3	15:40 to 15:45	24	58	6	18	**106**
4	15:45 to 15:50	24	57	8	19	**108**
5	15:50 to 15:55	23	47	7	7	**84**
6	15:55 to 16:00	12	37	12	8	**69**
7	16:00 to 16:05	17	37	6	12	**72**
8	16:05 to 16:10	18	48	6	15	**87**
9	16:10 to 16:15	21	45	7	18	**91**
10	16:15 to 16:20	23	44	8	18	**93**
11	16:20 to 16:25	23	43	12	20	**98**
12	16:25 to 16:30	21	51	15	17	**104**
Total Vehicle/ Type		**256**	**541**	**105**	**165**	

Direction: Palashi to Lalbag Day: Thursday weather: sunny Date: 10-16-11

Time Pack	Time(Sample 9:00 to 9:05	1	2	3	4	5	Total Traffic/ Time pack
		Car, Jeep, Taxi, Micro	NMT	Other Trucks, Pickups	3 Wheelers	Motorcycle	
1	15:30 to 15:35	4	112	1	13	5	**135**
2	15:35 to 15:40	5	134	3	9	8	**159**
3	15:40 to 15:45	2	114	0	16	2	**134**
4	15:45 to 15:50	8	143	0	11	11	**173**
5	15:50 to 15:55	1	123	3	17	18	**162**
6	15:55 to 16:00	2	125	2	15	15	**159**
7	16:00 to 16:05	4	128	4	14	16	**166**
8	16:05 to 16:10	3	132	2	6	19	**162**
9	16:10 to 16:15	3	134	2	13	14	**166**
10	16:15 to 16:20	7	125	4	8	9	**153**
11	16:20 to 16:25	5	111	1	6	12	**135**
12	16:25 to 16:30	4	109	3	5	7	**128**
Total Vehicle/ Type		**48**	**1490**	**25**	**133**	**136**	

Direction: Palashi to azimpur Day: Thursday weather: sunny Date: 10-16-11

Time Pack	Time	1	2	3	4	5	6	Total Traffic/ Time pack
		Car, Jeep, Taxi, Micro	NMT	Bus	Other Trucks, Pickups	3 Wheelers	Motorcycle	
1	15:30 to 15:35	10	112	3	3	2	1	**131**
2	15:35 to 15:40	13	132	1	2	4	1	**153**
3	15:40 to 15:45	21	112	2	7	10	4	**156**
4	15:45 to 15:50	22	102	1	4	2	5	**136**
5	15:50 to 15:55	12	121	0	3	0	2	**138**
6	15:55 to 16:00	16	133	0	5	4	4	**162**
7	16:00 to 16:05	11	135	0	5	3	7	**161**
8	16:05 to 16:10	9	123	2	6	3	0	**143**
9	16:10 to 16:15	7	124	4	8	0	0	**143**
10	16:15 to 16:20	19	113	3	11	2	2	**150**
11	16:20 to 16:25	23	107	1	5	6	0	**142**
12	16:25 to 16:30	21	105	2	8	0	4	**140**
Total Vehicle/ Type		**184**	**1419**	**19**	**67**	**36**	**30**	

Direction: Nilkhet to Palashi Day: Thursday weather: sunny Date: 10-16-11

Time Pack	Time	1 Car, Jeep, Taxi, Micro	2 NMT	3 Bus	4 Other Trucks, Pickups	5 3 Wheelers	6 Motorcycle	**Total Traffic/ Time pack**
1	15:30 to 15:35	25	78	3	2	5	10	**123**
2	15:35 to 15:40	23	85	2	1	7	7	**125**
3	15:40 to 15:45	26	82	4	0	11	5	**128**
4	15:45 to 15:50	31	87	4	1	5	11	**139**
5	15:50 to 15:55	34	81	6	2	8	9	**140**
6	15:55 to 16:00	32	89	2	1	9	10	**143**
7	16:00 to 16:05	28	95	3	1	5	11	**143**
8	16:05 to 16:10	32	92	4	1	3	8	**140**
9	16:10 to 16:15	29	83	5	1	6	13	**137**
10	16:15 to 16:20	31	76	4	2	10	8	**131**
11	16:20 to 16:25	34	79	2	2	7	7	**131**
12	16:25 to 16:30	28	82	2	1	4	5	**122**
Total Vehicle/ Type		**353**	**1009**	**41**	**15**	**80**	**104**	

Equivalent Passenger Car Unit (PCU).

VEHICLE TYPES	FACTORS
Car, Jeep, Taxi, Micro	1
NMT	0.5
Bus	3
Other Trucks, Pickups	3
Motorcycle	.5

3 Wheelers	.8

Intersection inflow:
From-
Bakshibazar approach: 52*1+19*0.5+77*3+36*0.8+30*0.5=336.3 PCU/hr
Lalbag approach: 80*1+1453*0.5+30*3+136*0.8+121*0.5=1065.8 PCU/hr
Azimpur approach: 199*1+1341*0.5+18*3+77*3+36*0.8+30*o.5=1198.3 PCU/hr
Nilkhet approach: 353*1+1009*0.5+41*3+15*3+80*0.8+104*0.5=1141.5 PCU/hr
Dhaka university approach: 256*1+541*0.5+105*0.8+165*0.5=693 PCU/hr
Total volume: 336.3+1065.8+1198.3+1141.5 +693=4434.9 PCU/ hr

3.2 SPEED STUDY

It is a basic measure of traffic and roadway performance and it influence the traveler in selecting routes or transportation modes

DATA:

Name of approch	Car, Jeep, Taxi, Micro	NMT	Bus	Other Trucks, Pickups	3-wheelers	Motorcycle
BAKSHIBAZAR TO PALASHI	21.77	10.98		15.77	12.38	19.87
PALASHI TO BAKSHIBAZAR	19.87	11.44 11.44 fps	23.69 fps	23.69 15.49 fps	19.38 15.49 fps	19.38
AZIMPUR TO PALASHI	12.79	10.01	21.69	13.91	16.89	18.37
DHAKA UNIVERSITY TO PALASHI	22.78	7.86			19.85	21.78
PALASHI TO LALBAG	18.97	11.08		15.45	14.38	17.37
LALBAG TO PALASHI	17.95	12.07		16.32	14.88	19.46
PALASHI TO AZIMPUR	14.39	12.01	21.00	15.91	18.89	16.37
NILKHET TO PALASHI	21.24	12.23	17.43	16.69	19.12	23.21

AVERAGE SPEED:
BAKSHIBAZAR APPROACH: 16.15 fps or 10.90mile/h
NILKHET APPROACH: 18.32 fps or 12.49mile/h
LALBAG APPROACH: 16.14fps
DHAKA UNIVERSITY APPROACH: 18.07 fps
AZIMPUR APPROACH: 15.61 fps

Site Analysis & Problems associated with the intersection:

No light on signals works.
No marking on road or markings are insufficient or not clearly visible**.**
No facilities for pedestrian crossing ,no zebra crossing ,no Stop control.
Road signs are incorrect or blocked with poster.
Footpaths & roads are partially jammed with kitchen market & dustbin &slum.
Road dividers are comparatively wider than road width.
Drainage system is inadequate.
Road visibility for right turn from Bashibazar road to lalbagh Road Rd is very poor as at the corner Building is situated.
Again as the agle between the Bashibazar road to lalbagh Road is far less than 75^0 ,it is about 50^0 ,so taking turn is difficult & conflict in movement occurs.
Road blocker & heavy vehicle resistant bar blocks a good percentage of road which reduce road

Remedial Measures :

- Width of dividers & footpaths are comparatively large with respect to road width. So by decreasing the width of divider & footpath ,road width can be increased.
- If width of road is increased then a separate lane for rickshaw can be provided & conflict of movement of vehicle of higher speed with vehicle of lower speed can be avoided.
- Kitchen market & slums should be removed from the footpath & on the footpath only pedestrian should have the right of way.
- Light of the signals should be repaired.
- Road markings are not clearly visible ,as the paint color is weather susceptible proper maintenance or renovation should be taken.
- Road blockage bar or heavy vehicle resisting bar should be placed on the footpath so that road width can be increased & capacity of road can also be increased.
- Drainage system should be improved to eliminate water clogging on road.
- Road signs should be correctly introduced.
- Provision for pedestrian crossing should be introduced such as Zebra crossing or Pedestrian call buttons should be introduced.
- As traffic jam occurs due to vehicle turning to bashibazar from the lalbagh Rd . So if a separate side road from bakshibazar to lalbagh road can be made then traffic jam can be reduced to a large extent.

Preferable Intersection:

Signal Design:
As Polashi is a five legged intersection ,so by designing signal no significant improvement can be done rather more complication in vehicle movement can occur . Another thing can be done that is the lalbagh approch can be closed & a separate side road that connect lalbagh & bakshibazar can be introduced & than a two phase signal can be designed for the new four legged intersection with total cycle time of 182sec. The detailed calculation of signal design is provided below :

Bajshibazar: 1.5*1.3*336.3 pcu/hr =655.78pcu/hr
Azimpur: 1.5*1.3*1198pcu/hr=2336.1pcu/hr
Nilkhet: 1.5*1.3*1141pcu/hr=2226.5pcu/hr
Dhaka University:1.5*1.3*693=1351.35 pcu/hr

	North(DU)	South(Azimpur)	East(Bakshibazar)	West(Nilkhet)
Flow	1352.35	2336.1	655.78	2226.5
Sat. Flow	4000	4000	4000	4000
y	0.33	0.58	0.16	0.55

$y_{dominating.}$ 0.58 0.55
Y=0.58+0.55=1.13

	N-S	E-W
Inter green(i)sec	7	6
Lost time(l)sec	3	2

Amber, a = 3sec
L=all red+ green-ends lost times =sum(i-a)+sum(l)=4+3+3+2=12sec
Optimum Cycle Time, C_0= (1.5L+5)/(1-Y)
=(1.5*12+5)/(1-1.13)= 176sec
Total effective Green Time,C_0-L= 176-12=164sec
Effective green For NS arm ,g_{NS}= y_{ns}*(C_0-L)/Y= 84sec
Effective green For EW arm , g_{EW}= y_{ew}*(C_0-L)/Y= 80sec
Green+Amber period for NS arm= g_{ns}+l=87sec
Green+Amber period for EW arm= g_{ES}+l=82sec

G_{NS}=84sec
Intergreen,NS=9s
G_{EW}=80sec
Intergreen,NS=9s

Total Cycle Time=182sec

Here the classification of roundabout is given below:

Roundabouts - Basic Design and Operational Elements						
Design Element	Mini-Roundabout	Urban-Compact	Urban Single Lane	Urban Double Lane	Rural Single Lane	Rural Double Lane
Maximum entry speed	15 mph	15 mph	20 mph	25 mph	25 mph	30 mph
Max. number of entry lanes per approach	1	1	1	2	1	2
Typical Inscribed Circle Diameter	45 ft. to 80 ft.	80 ft. to 100 ft.	100 ft. to 130 ft.	150 ft. to 180 ft.	115 ft. to 130 ft.	180 ft. to 200 ft.
Typical ADT on 4-leg roundabout (veh/day)	10,000	15,000	20,000	20,000 +	20,000	20,000 +

As the size of our intersection is not so big ,so a mini roundabout can be provied .The total diameter of our intersection is about 95 ft,The maximum average speed in all the road is almost 13mile/hr which match with the design .So I think a mini roundabout of diameter 45ft can be an appropriate option then there will be available circulatory road with 25ft width.

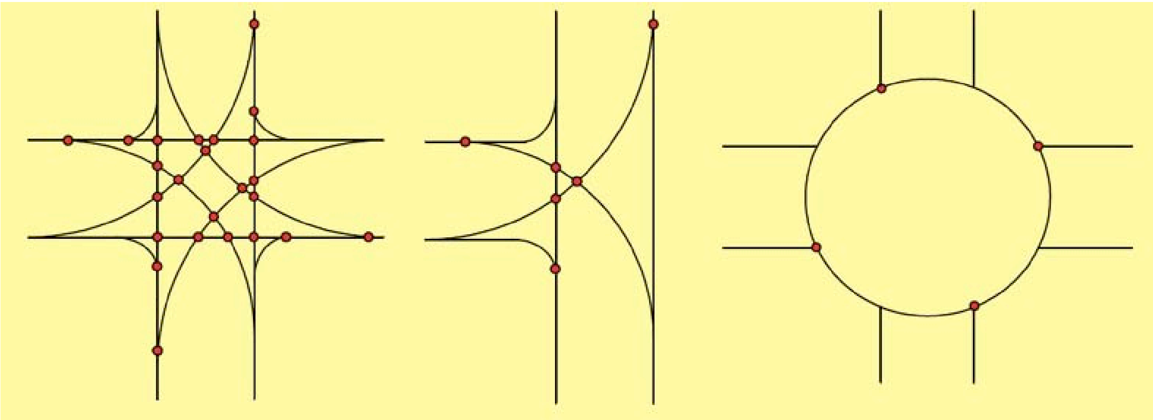

Fig: Number of conflicts is reduced due to Roundabout

CONCLUSIONS:

From the condition which we obtained from visiting the site, it can be said that there are many problems associated with the intersection & proper steps should be taken as soon as possible. If correct measurements are taken then traffic jam can be removed totally & movements of vehicles can be very smoother.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE EVALUATION OF SPEED REDUCING DEVICES ON NATIONAL HIGHWAYS OF BANGLADESH

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ABSTRACT

Recently, different types of speed reducing formal and informal devices are introduced on the different types of roads and highways in Bangladesh. Though, most of the physical devices are not properly designed and installation process is not pre-planned, that's have significant positive and negative impact on safety performance of the road. But, for the lack of proper evaluation of the performance of those devices, the effectiveness and competency of those devices is not yet known. Even, proper pragmatic scientific efficient and optimum beneficial design and installation policy not yet standardized. That's why; local post induced devices are creating sometimes adverse impact on the road environment. In this study an ambitious attempt has been made to evaluate the performance of speed reducing devices on national highways of Bangladesh. It is found that, after imposing speed hump on the road, accidents and resulting casualties are reduced around 50 to 75 percent in different study sites and the reduction of speed was from 80 to 60 percent for different types of motorized vehicles. For the study, some highly accident prone location on Dhaka-Mawa highway in which speeds reducing physical devices are superimposed has been considered for the impact evaluation. Different statistical and analytical tools have been used to evaluate the performance in the study. At the very outset of the study, world standard and Bangladesh practices related to speed reducing devices have been discussed.

Keywords*: National Highway, Speed Reducing Device, Performance Evaluation, Before-After Study, Control Site Study*

1. INTRODUCTION

Road traffic accidents and injuries are now a serious and growing problem in many developing countries including Bangladesh. Of all reported accidents and fatalities nearly 38% percent and 42.3% respectively occurred on national highways in Bangladesh and the victims are predominantly pedestrians (about 54%) in particular rural people who are ignorant about the traffic rules and regulations (Maap5). It is one of the established and fully recognized facts that vehicular speed has a remarkable effect on road traffic accidents and severities and many major accidents occurred due to aggressive speed on road. To control as well as to reduce the vehicular speed according to the road operational and functional environment, different types of devices are used in all over the world in modern transportation system due to control of accidents and their severity. In Bangladesh, also there are various types of speed reducing devices in particularly speed bump are using in different types of roads including national highway.

On the other hand, it is specified that the sole purpose on national highway is to maintain highest mobility not accessibility. For ensuring high speed with safety, comfortable and efficient mobility of the national highway, the road should be free from any obstruction which could be generated from planning problem, geometric default, operation and management deficiencies, road and road side environmental hazards etc. That is, the national highway should be free from physical speed reducing devices like speed hump or bump. But, there are huge number physical speed reducing devices are imposed on the almost all of the national highways in Bangladesh. In the study area, Dhaka-Mawa highway (a section of national highway N8) one of the major gateway of the capital city of Bangladesh has found 44 speed reducing devices in 32 km segment. Almost all of the devices are superimposed by the local community/authority on the pressure of local people to prevent accident and their severity as well as to save lives through reducing speed of the vehicle. But, no concrete results are known yet on effectiveness of these contradictory measures. Recently, a study has been conducted by Accident Research Institute (ARI) to evaluate the performance of these devices on a small scale in the Dhaka-

Mawa highway. This paper formed from the part of that study. At the very outset of the paper, an overview of the dimensions and characteristics of the road traffic accident problems in the national highways of Bangladesh, with a special reference to the situation Dhaka-Mawa national highway route N8 in between km post 6 to 38.

2. BACKGROUND OF THE STUDY

In order to get a clear picture of the whole highway system the first prerequisite is to know the elementary functional elements of a highway. The modern highways should have all those requirements to fulfill it to be served as a highway. But highway system of Bangladesh has hardly followed those requirements. Of all the inaccuracy, the most common is the presence of physical speed reducing devices (speed breaker, speed bumps, and rumble strip).

The National Highways can be classified functionally which is the process by which they are grouped into classes, or systems, according to the character of traffic service that they are intended to provide. There are three highway functional classifications (AASHTO, 2010):

- Arterial roads,
- Collectors roads and
- Local roads.

Considering the factor it can be said that a national highway serves as the arterial road which provides the maximum level of service including maximum speed, most uninterrupted driving area, lots of mobility and so on. If speed reducing devices (speed breaker, speed bumps, rumble strip, etc.) are installed in this type of arterials, there is a definite possibility of interrupting the service of arterials by reducing the designed speed, increasing the journey time and creating uncomfortable situation for the passengers while travelling. If these are installed only for the sake of speed reducing purpose other than speed breakers there are many more measures to be utilized. Are these speed breakers are enough to provide for the safety in highway? If enough then why it is found that in some places the speed reducing devices are prone for the accidents? Why people call them spine crackers instead of speed breakers and often as accident makers? There has been a discussion on this problem in this research work.

Of many deficiencies of highway system of Bangladesh the above is one of the major concern of this research work. These issues are almost recent dilemma of the road transports system in Bangladesh. Many discussion as well as table talk has been occurring on the significance or impact of these super imposed measures/devices, but no significant research based evidences are available on these issues. Almost all are of the discussions, table talk are taking place on the basis of general idea or common sense without considering in-depth aspects or multi-dimensional issues and factors. That’s why, no fruitful pragmatic and sustainable recommendations is not coming out from that initiatives. So, it is the time to conduct in-depth investigation and research on these issues to find out the actual and authentic facts and evidence. In this study, an attempt has been to evaluate the significance of speed breakers on the national highway in particular and to assess the impact of direct local access with the national highway.

This paper formed from the part of the broader study on “performance of speed reducing devices on the national highways of Bangladesh”. Detailed methodology, data collection and interpritetion procedure could be seen in Hossain (2011).

3. ACCIDENT PROBLEM CHARACTERISTICS IN N8 HIGHWAY

From the statistics it is revealed that Bangladesh has one of the highest fatality rates in accidents on national highways. About 60-70 percent of road accidents are fatal in the national highways of Bangladesh. According to the ARI MAAP data, there were at least 591 fatalities and 165 injuries in 756 reported road accidents has been occurred in the period of 1998 to 2009 in the route N8 of Dhaka-Mawa national highway i.e. around 100 accidents are reported in this highway (Maap5, 1998). Among this around 20 accidents are recorded in the segment of Dhaka-Mawa highway section (6-32 km post) though the actual number is much more than that which is manifested from the on-site observation and local people observation. The following are the striking accident problem characteristics in Bangladesh as evaluated from the observation of selected site

- Accidents on National Highways: From the total reported accidents nearly 37 percent occurred on the national highways of Bangladesh. Hazards associated with roads and roadsides were particularly by collision type. The national highways of Bangladesh are very accident prone and the accidents are of various kinds which depend upon various time.

- Accidents Trends at N8: From Figure 1, it is evident that the linear trend of accident is in increasing from the year 1998 in this route. In the year 2001 the rate was lowest but with time it increased to high enough in both ascending and descending rate. Periodic maintenance of the road and reporting and recording inconsistency could be the behind reason of this ascending and descending rate of accident. If the rate continues then it will be a for the road traffic to continue its intended service.

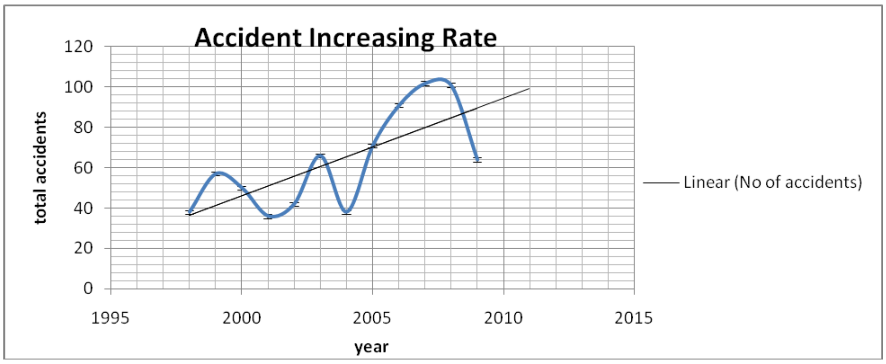

Figure1: Accident Increasing Rate on route N8 (1998-2009)

- Accident Severity by Collision Type: Accident type analysis showed 'hit pedestrian' as the dominant accident type in the national highways of Bangladesh. Other common accident types are: head on collision (18%) and overturning (11%), rear end collision (8%). These four accident types account for nearly 85 percent of the fatal accidents (Figure 2).

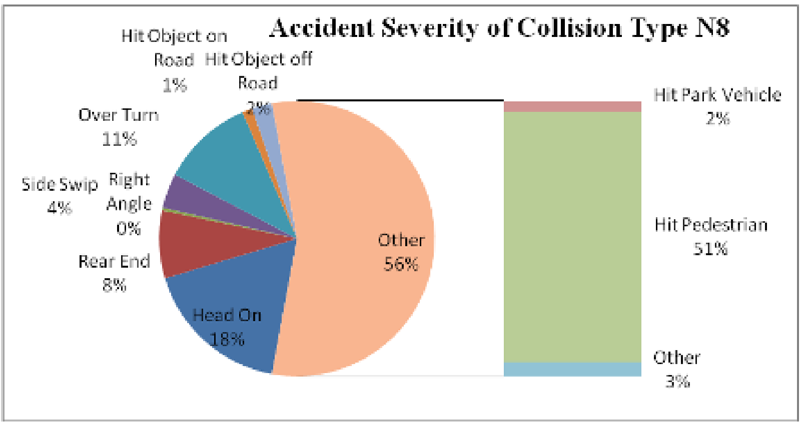

Figure 2: Accident by Collision Type

- Pedestrians: The Most Vulnerable Road User Group: In the national highways of Bangladesh, about 51 percents accidents occur with the pedestrian which make the pedestrian as the most vulnerable road user group. In the highways the vehicles move at a speed and the pedestrian cannot accommodate themselves with the highway environment. Lack of signs and markings make the pedestrian more vulnerable.

- Vehicular Involvement: Of the vehicles involved in all fatal accidents, nearly two-thirds are buses and trucks. This group of vehicles are particularly over involved in pedestrian fatalities. Involvement of car and other light vehicles in fatal accidents is 13 percent, followed by motorcycle and auto rickshaws, 7.5 percent. The shares of buses and trucks in registered vehicles are around 13 percent.

- Accident Casualty by Casualty Class: Figure 3 shows the accident by casualty classes. Pedestrian casualties are by far the predominant which involvement is 49 percent of all casualties, followed by bus occupants 15 percent, three wheelers occupant 13 percent and microbus occupants 9 percent.

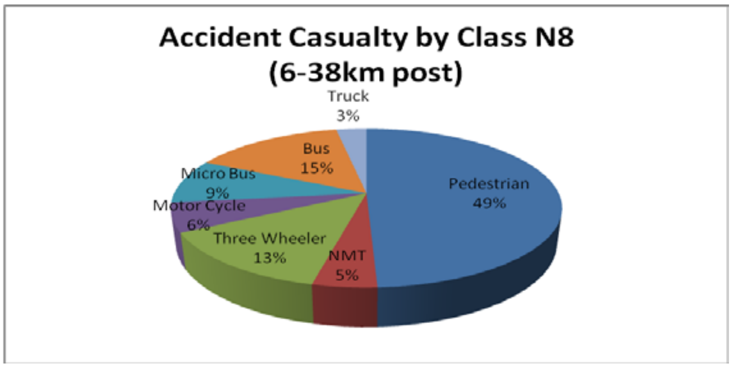

Figure 3: Accident Casualty by Casualty Class

- Socio-economic Burden of Accidents: Together with the social impact in terms of pain, grief and suffering, there is a serious economic burden.

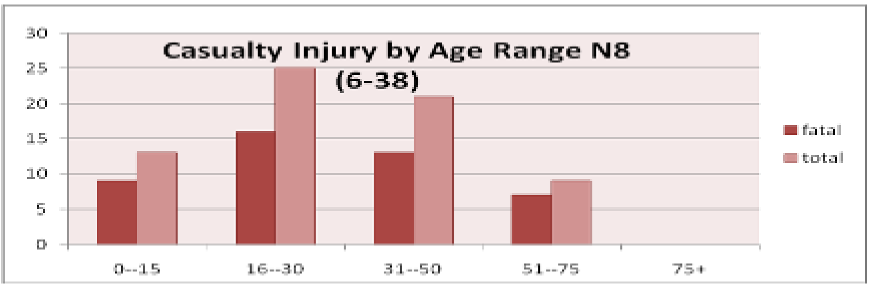

Figure 4: Casualty Injury by Age Group

From the ARI MAAP data analysis it is revealed that Bangladesh lose the most economically active years from road accident victims and most of the victims are of range 16-30 and in that case the fatalities are also more high rate. Thus in case of road traffic accidents in national highways of Bangladesh the loss is creating burden in case of socio economy.

- Use of Excessive Amount of Speed Reducing Devices: In the national highways of Bangladesh there are lots of use of speed reducing devices makes the highway sometime safe but also making it a prone area for accidents. Though there are lots of speeds reducing devices (around 44 speed reducing devices in about 32 km studied area) the drivers' loss respects for them and thus they go over them recklessly. Most of the time this situation creates heavy jerking to the vehicles causing damage to chassis of the vehicles and also discomforts to the passengers. Many a time overturning and side swipe is common for this excessive use.

4. PRACTICE OF SOME SPEED REDUCING DEVICES IN NATIONAL HIGHWAYS OF BANGLADESH

In Bangladesh road traffic accidents take place due to the reckless speed of the vehicles in the national highways. So use of speed reducing devices are common in the highways. There are some commonly used speed reducing devices in the national highways to reduce the vehicle speed and to minimize the accidents. The most used speed reducing devices are:

- Speed humps,
- Speed bumps,
- Rumble strips,
- Speed sign and
- Markings.

4.1 Speed humps in Bangladesh practice

In Bangladesh practice there are several variations from the standard one. The variations are like:

- The gradual raised area is hardly found in Bangladesh. In some highly maintained road there are some gradual raised areas but most of the highways do not have this.
- The height found in the study was about 4 inch.
- In most cases the speed humps in Bangladesh do not create gentle vehicle rocking motion. Though its lacks gradual raised area so there is a sudden rocking motion.
- From the field inspection it was found that the local people constructed the speed humps where it was necessary to take help form an engineer.
- From the field inspection it was seen that some time the speed humps are wired out due to several reasons like abruption, rain, not maintaining the speed humps for a long time.
- In Bangladesh there are no basic difference maintained in speed hump and speed bump.
- The speed humps in Bangladesh do not fulfill the intended service.
- In Bangladesh the vehicles speed does not decrease to 15mph in the approach of speed hump. It is generally more than 25mph in the approach of speed hump.

4.2 Speed bumps in Bangladesh practice

- Though it is for private driveways and parking lots, from field survey it was found that highways contain these.
- From study it was seen that the average height of the speed humps are about 4.5 inch and their average length was about 2.5 feet.
- They cause significant discomfort to the both drivers and passengers.
- The speed bumps in different places were wired out and cause serious discomfort and jerking.
- In Bangladesh the speed bumps are not established by engineering means.

- Most of the speed bumps are provided because of the application of local people. Most of the cases no care was taken to establish and maintain it.
- Because of lack of standard design manual of speed bump, in most of the cases the speed bumps are given by lining one or two layer of bricks and pitching over it.
- In various locations it was seen that there were leveled down of speed bumps with the road thus it failed to do its intended function.
- Some time the alignments of the speed bumps are not in right way. They move a bit away from their actual alignment due to lack of maintenance and operation.

4.3 Rumble strip in Bangladesh practice

- In Bangladesh the rumble strips are not in correct position, they are in a scattered way.
- The purposes of rumble strips are same but the alignment is not correct.
- Most of the cases they are wired out and it is hard to find them from a distance place.
- Though they are for to alert the drivers and to remove their monotony but due to their wrong alignment they cause excessive jerking effect and discomfort to the passengers.
- In Bangladesh the rumble strips are established according to any design specification.

- Though the rumble strips are to reduce the monotony of the drivers, in Bangladesh they are made of small concrete chips which causes heavy shake and jerk to the vehicle and passengers.
- The jerking of the vehicle damage the chassis of the vehicle.

4.4 Road Signs in Bangladesh practice

In Bangladesh the road signs are not maintained in a good way. People do not care about the road sign and the maintenance is very poor.

- The alignment of the road signs are not in specified way. No take care of the road signs were there.
- If the road signs fall in one direction then there is no tendency to maintain this.
- Sometimes different posters were put on the road signs because of lack of awareness which causes serious road hazards.

4.5 Road Markings in Bangladesh practice

- Wiring out of road markings is very common in Bangladesh.
- Though it occurs most of the cases, maintenance of these are very rare.
- Most of the time it is impossible to look at the road markings from a distance place because of it's wired out.

5. METHODOLOGY USED TO EVALUATE THE PERFORMANCE OF SPEED REDUCING DEVICES

In this study basically two methods are used to evaluate the performance of speed reducing devices.

a) Before and after Studies,
b) Comparisons using Control Sites.

5.1 Before and After Studies

The simplest method is to compare the accident record at the site before and after the complementation of the scheme. It essentially involves:

- Determining in advance the relevant objectives (e.g. accidents types intended to be affected) and the corresponding evaluation criteria (e.g. accident frequency, accident rate).
- Monitoring the site or area to obtain numerical values of these criteria before the treatment and again after the treatment.
- Comparing the 'before' and 'after' results, and
- Considering whether there are other possible explanations for the changes, and correcting for them if possible.

5.2 Comparisons Using Control Sites

The process involves

- Determining in advance the relevant objectives (e.g. accident types intended to be affected) and the corresponding evaluation criteria (e.g. accident frequency, accident rate).
- Indentifying a control site or (preferably) a set of control sites where no remedial works have been or are intended to be introduced.
- Monitoring both the treated sites and the control sites to obtain numerical values of these criteria before and treatment and again after the treatment.
- Comparing the 'before' and 'after' results at both the treated and control sites,
- Considering whether there are other plausible explanations for the changes, and correcting for them if possible.
- Selection of the control sites is obviously of key importance. Ideally, they would be randomly selected. However, this is rarely possible, unless a large number of control sites can be identified and random selection made from these.

The control sites should satisfy the following criteria (OECD, 1997)

- Be similar to treated sites in general characteristics (network, geometry, land use, enforcement)
- Be geographically close,
- Have similar traffic flow,
- Not affected by treatment at test site,
- Not treated within before-after period
- Have accident data

6. PERFORMANCE EVALUATION OF SELECTED SITES ON DHAKA-MAWA HIGHWAY

In the Dhaka-Mawa highway from route N8 and 6 to 38 km post, there are about 44 speed reducing devices. It denotes that there in about 1.4 speed reducing devices per KM. This high amount of speed reducing devices in this small distance signifies very poor condition of the highway system and its surrounding condition.

6.1 Present Safety Scenario of Dhaka-Mawa Highway

From the field observation coupled with the discussion with the local people, it is found that most of the speed reducing devices has been provided within 2008 to 09. From the analysis of the accident data, it is found the highest number of accident has been recorded in 2008. Most of the speed bump has been imposed at that period. In 2009, the recorded number of accidents and fatalities are dramatically reduced (Figure 5).

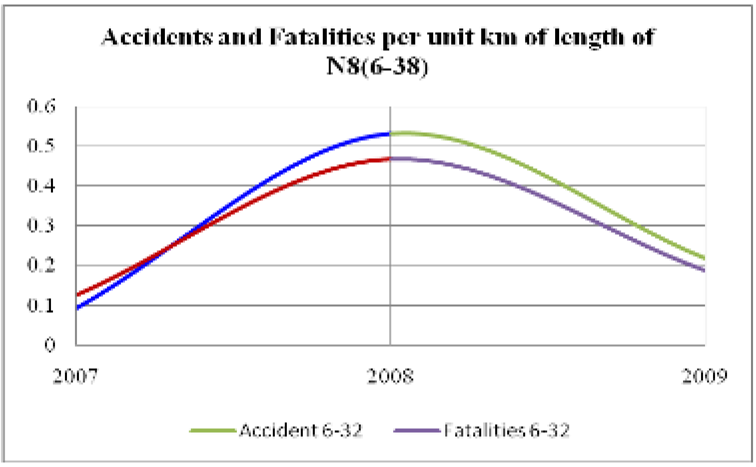

Figure 5: Accident Trends in Last Three Years

6.2 Case Study to Evaluate the Performance of Speed Hump

Performance of speed reducing devices in connection with the main highway is illustrated on the data collected from site investigation and review of on-site people's comments (Field Survey, 2011). Among enormous performance of speed reducing devices the main or striking performance are intended to highlight here on different sites' perspectives.

6.2.1 Case study – 1: Nimtali-Sirajdikhan Intersection
Location: Nimtali-Sirajdikhan Intersection
KM post: MAWA 18
Land use pattern: Market, Bus-stand, 4-leg road intersection

Accident Data:
- From MAAP 5 : 2- Fatal, 1-Greivious
- From Site Visit: 3-4 accidents per month
- Time of accident: 11.00 A.M. to 12.00 P.M.
- Speed of vehicle moving: High enough
- Use of adequate sign/marking: No

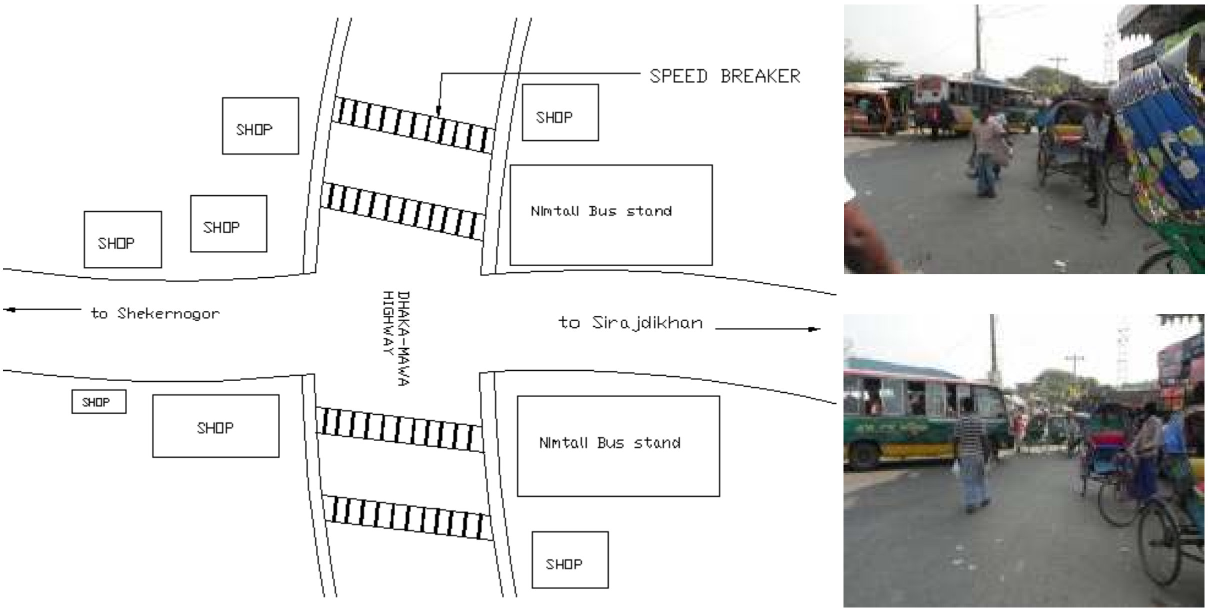

Figure 6: Layout of Nimtali-Sirajdikhan Intersection

Before-after study based on field survey

Based on the field survey of the area it was found that the speed reducing devices were placed at this point due to the demand of the local people because of lots of accidents.

Table 1: Before-After Study based on Field Survey

Before application of Speed Reducing Devices accidents per month	After application of Speed Reducing Devices accidents per month
4	2

Source: Field Survey,2011

From Table 1 it is obvious that before the placement of speed reducing devices the rate of accidents were about 4 accidents per month, but after the installment of speed reducing devices the accident now a day reduced to about 2 accidents per month. Which shows that after the installment of the speed reducing devices the rate of accident decreased but it is not completely off which is very much alarming.

Before-After Study based on MAAP 5 data

From the MAAP data the accidents occurred in the Nimtali-Sirajdikhan Intersection shows that:

Table 2: Before-After Study based on MAAP 5 Data

Number of Accidents Before 2008		Number of Accidents After 2008	
Fatal	Non Fatal	Fatal	Non Fatal
4	1	0	0

Source: MAAP5 data

From the Table 2 it is found hat, after the application of the speed reducing devices the accidents occurred in this area is reduced to zero. This reflects that after the application of the speed reducing device the accidents were reduced to great amount and thus it seems that the application of the speed reducing device in this place is playing a very much effective rule.

Control site study

To do the control site study the author has taken two distinctive points at 14 km post and 43 km post where no remedial works have been taken.

Table 3: Control Site Study of Nimtali-Sirajdikhan Intersection (No. of Accident)

Control Site 1 (km post14)			Control Site 2 (km post43)			Treated site 1 (Nimtali-Sirajdikhan Intersection)		
Before 2008	2008 -2009	% of accident reduction	Before 2008	2008-2009	% of accident reduction	Before 2009	2008 -2009	% of accident reduction
8	6	25	3	2	33.33	5	0	100

Source: ARI, MAAP data

By comparing the result with the Nimtali-Sirajdikhan intersection it is seen that the difference in the percentage of the treated area-1 and the control site is 75 to 66 percent. Which indicate that due to the use of speed reducing devices the percentage of reduction of accidents has a remarkable high value. So, it can be said that the speed reducing devices placed in these area are working in efficient way.

6.2.2 Case study- 2: Hashara Bridge Approach

Location: Hashara Bridge Approach Bazaar
KM post: MAWA 15
Land use pattern: Bazar, Bus-stand, intersection

Accident Data:

- From MAAP 5 : 4 accidents per year
- From Site Visit: 5-6 accidents per year
- Time of accident: School hours
- Speed of vehicle moving: High enough
- Use of adequate sign/marking: No

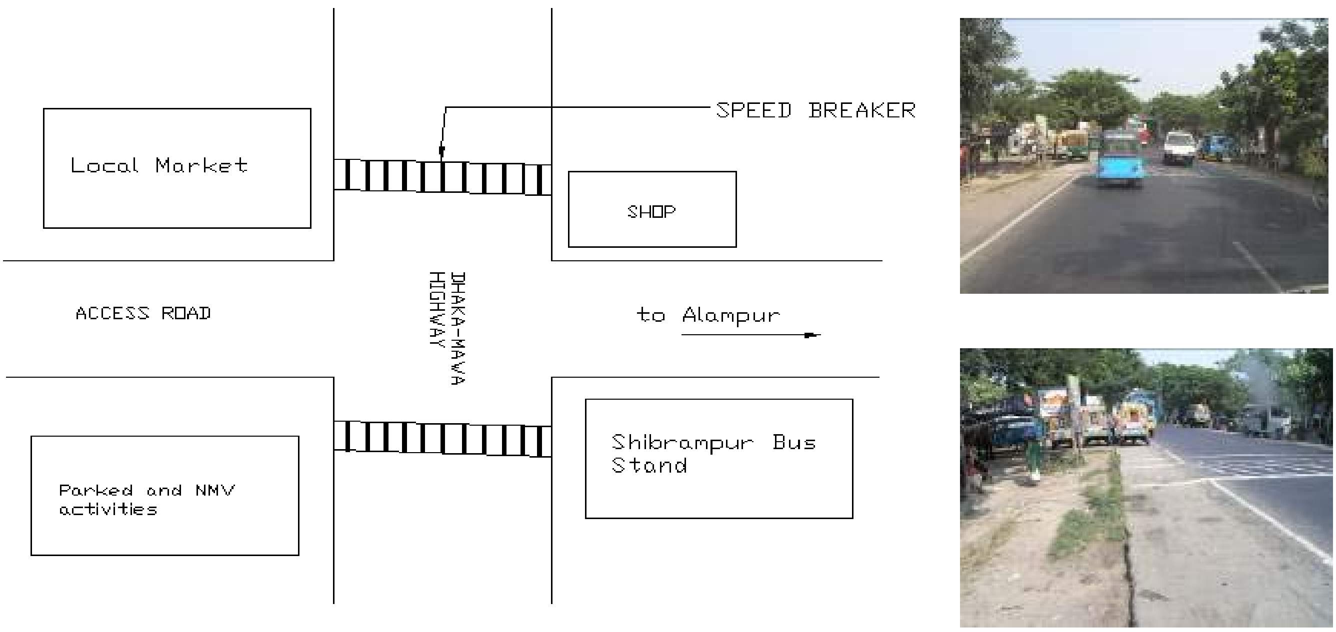

Figure 7: Layout of Hashara Bridge Approach

Before-After Study based on Field Survey

Based on the field survey of the area it was found that the speed reducing devices were placed at this point due to the demand of the local people because of lots of accidents.

Table 4: Before-After Study based on Field Survey:

Before application of Speed Reducing Devices accidents per year	After application of Speed Reducing Devices accidents per year
6	4

Source: Field Survey,2011

From the Table 4, it is obvious that before the placement of speed reducing devices the rate of accidents were about 6 accidents per year, but after the installment of speed reducing devices the accident now a day reduced to about 4 accidents per year. Which shows that after the installment of the speed reducing devices the rate of accident decreased in a very little amount which is very much alarming.

Before-After Study based on MAAP 5 data:

From the MAAP data the accidents occurred in the Hashara shows that:

Table 5: Before-After Study based on MAAP 5 Data

Number of Accidents Before 2008		Number of Accidents After 2008	
Fatal	Non Fatal	Fatal	Non Fatal
4	4	2	0

Source ARI, MAAP data

From the Table 5 it is seen that, after the application of the speed reducing devices the accidents occurred in this area is reduced but the number of fatal accidents are not reduced. This reflects that after the application of the speed reducing device the accidents were reduced to a goof amount. Thus we can say that the speed reducing devices places here are working more or less efficiently.

Control Site Study:

To do the control site study the author has taken two distinctive points at 14 km post and 43 km post where no remedial works have been taken.

Table 6: Control Site Study of Hashara (No. of Accidents)

Control Site 1 (km post14)			Control Site 2 (km post43)			Treated site 2 (Hashara)		
Before 2008	2008 - 2009	% of accident reduction	Before 2008	2008-2009	% of accident reduction	Before 2009	2008 - 2009	% of accident reduction
8	6	25	3	2	33.33	8	2	75

Source: ARI, MAAP data

By comparing the result with the Hashara it is seen that the difference in the percentage of the Hashara and the control site is 50 to 36%. Which indicate that due to the use of speed reducing devices the percentage of reduction of accidents has a remarkable high value. So, it can be said that the speed reducing devices placed in these area are working in efficient way.

Case study -3: Keotkhali
Location: Keotkhali Primary School
KM post: MAWA 13
Land use pattern: Primary School, Small baazar

Accident Data:

- From MAAP 5 : 2
- From Site Visit: 2-3 accident per year
- Time of accident: School hours
- Speed of vehicle moving: High enough
- Use of adequate sign/marking: Yes

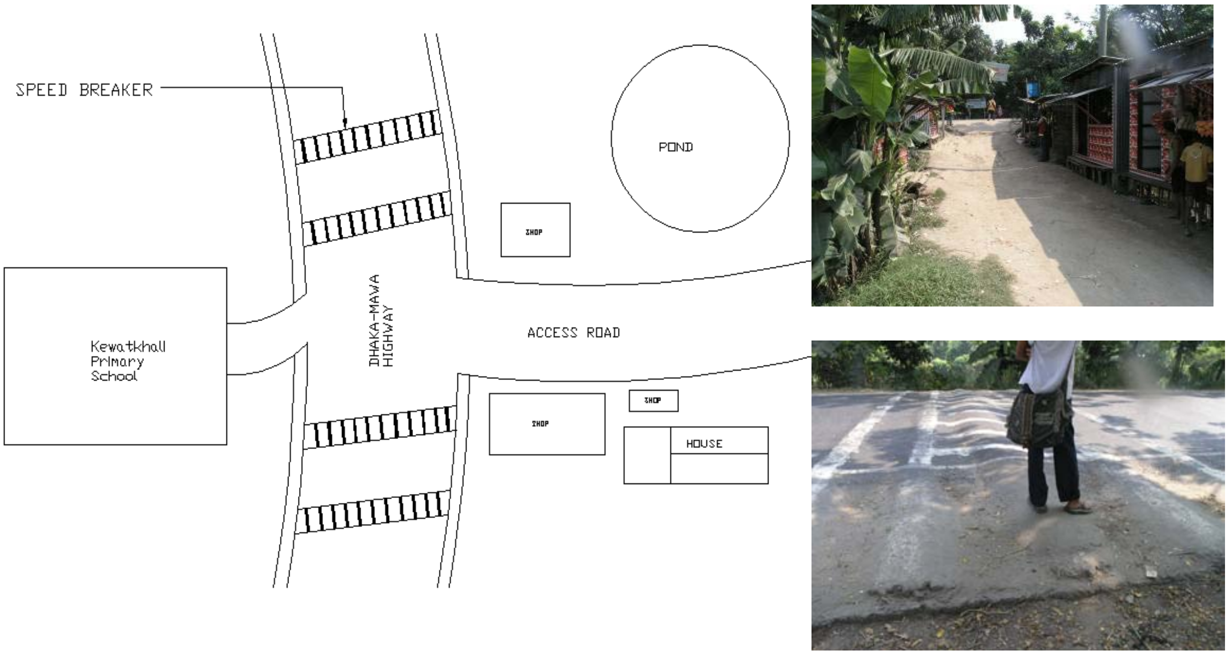

Figure 7: Layout of Kewatkhali Point

Before-After Study based on Field Survey:

Based on the field survey of the area it was found that the speed reducing devices were placed at this point due to the demand of the local people because of lots of accidents.

Table 7 : Before-After Study based on Field Survey:

Before application of Speed Reducing Devices accidents per month	After application of Speed Reducing Devices accidents per month
3	2

Source Field Survey, 2011

From the Table7 it is obvious that before the placement of speed reducing devices the rate of accidents were about 3 accidents per month, but after the installment of speed reducing devices the accident now a day reduced to about 2 accidents per month. This fact implies that after the installation of the speed reducing devices, the rate of accident has decreased but not so significantly.

Before-after study based on MAAP 5 data

From the MAAP data the accidents occurred in the Kewatkhali shows that

Table 8: Before-After Study based on MAAP 5 Data

Number of Accidents Before 2008		Number of Accidents After 2008	
Fatal	Non Fatal	Fatal	Non Fatal
5	2	0	0

Source ARI, MAAP data

From the Table 8 it is seen that, after the application of the speed reducing devices the accidents occurred in this area is reduced to zero. This reflects that after the application of the speed reducing device the accidents were reduced to great amount and thus it seems that the application of the speed reducing device in this place is playing a very much effective rule.

Control Site Study

To do the control site study the author has taken two distinctive points at 14 km post and 43 km post where no remedial works have been taken.

Table 9: Control Site Study of Kewatkhali (No. of Accident)

Control Site 1 (km post14)			Control Site 2 (km post43)			Treated site 3 (Kewatkhali)		
Before 2008	2008 - 2009	% of accident reduction	Before 2008	2008-2009	% of accident reduction	Before 2009	2008 - 2009	% of accident reduction
8	6	25	3	2	33.33	7	0	100

By comparing the result with the Keawatkhali it is seen that the difference in the percentage of the treated site-3 and the control site is aroudnd 75 to 66%. Which indicate that due to the use of speed reducing devices the percentage of reduction of accidents has a remarkable high value. So, it can be said that the speed reducing devices placed in these area is effective for improving safety.

7. SPEED EVALUATION OF SPEED BUMP

Table 10: Reduction of Speed at Speed Hump

Serial	Category	Free flow condition	At Speed breaker	
		AVG SP	AVG SP	% Rduction
1	Large Bus	45	15	67
2	Bus	47	16	66
3	CNG	39	11	72
4	Car	46	9	80

From the Table 10 it is obvious that the due to the use of speed reducing devices in the zone the speed of the vehicles are reduced to about 60% from the free flow condition.

8. EVALUATION OF THE DHAKA-MAWA ROAD AS A NATIONAL HIGHWAY

Dhaka-Mawa road is treated as a national highway of Bangladesh. Daily hundreds of heavy vehicle travel through this road which makes the road so important. In this road from 6 to 38 km post there are about 44 speed reducing devices, about 126 access points. But form the functional classification of the road network system, the definition of national highway was that, a highway will give high mobility but with less access (Wright, Paul.1996). A highway must also fulfill some basic criteria regarding geometry, operation, maintenance and some other services. The author surveyed in four different places in this N8 route from 6 to 38 km post. The functions found in these roads are not enough to recognize it as a national highway.

Table 11: Evaluation of the Dhaka-Mawa road as a National Highway

Items	Number of locations		Remarks
	Has/have	Has/have no	
Signs	2	2	Very little amount of signs are there
Shoulders	4	0	Not in proper dimensions
Height of Embankment	20-24 ft		
Slope of Embankment	1:2.5 – 1:4.5		
Guard Rail	0	4	
Good surface	1	3	
Contrast With Markings	1	3	Not enough
Roadside Hazards	1	3	
Safety Measures	0	4	
Sight Distance	1	3	Not so much clear
Road Junction	4	0	Continuous
Bridges and Culverts	3	1	
Intersections	2	2	With lots of congestion
Traffic Signals	0	4	Not a little bit
Roundabouts	0	4	
Lightings	0	4	No facility for lightings
Median Barriers	0	4	
Design Speed	0	4	Not specified
Speed Limit	0	4	No limit given
Mixing of Traffic	4	0	Heavy mix of traffic
Access Per Km.	4		Lots of access
Speed Reducing Devices	1.4/km		Huge amount of speed reducing devices in short distance

Source: Field Survey, 2011

From the Table 11, it is evident that most of the elements which should have in a functional highway configuration are absent in the Dhaka-Mawa highway segment. It lacks all the major elements of a highway like safety measures, traffic signals, lightings, speed limit and lots of mixing of traffic. The amount of speed reducing devices placed in the road and the number of access it have do not refer it as a national highway.

9. CONCLUSIONS AND RECOMMENDATIONS

From the study, it is evaluated that road safety is a major problem in the Dhaka-Mawa highway. After imposing speed hump on the road, accidents and resulting casualties are reduced around 50 to 75 percent in different study sites and the reduction of speed from 80 to 60 percent for different types of motorized vehicles. It is also evaluated that the so called national highway could not refer as a national highway for causes of huge geometric and operational deficiencies. Though, speed hump is not compatible with the national highway, standard speed reducing physical devices should be continued on the road until the highway could be standardized or converted as a functional national highway with standard geometric and functional elements. At present condition, for improving safety as well as ascertaining aggressive driving, the following low cost measures could be taken:

- Use of standard speed reducing devices like speed humps which should have a gradual raised area in the pavement surface extending across the entire travel width.
- Speed humps should be designed and installed with proper planning and engineering.
- The rumble strips should be designed in such a way thus can reduce the monotony of the drivers in the national highways.

- Use of pictorial signs in place of words based on international protocols should be there for better understanding of the road condition.
- Traffic signs or road signs should be erected at the side of roads to provide information to road users.
- Road surface markings should be used on paved roadways to provide guidance and information to drivers and pedestrians.
- Uniformity of the markings is an important factor in minimizing confusion and uncertainty about their meaning, and efforts exist to standardize such markings across borders.
- Gateway or entry treatment can have a good effect in reduction speed of vehicle and thus reduction accidents.
- Many vehicles pass thorough the shoulder without facing the speed reducing devices and thus it is required to extend speed reducing devices in shoulders.

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TRAFFIC STUDY AND TRAVEL FORECAST CASE STUDY: TRAFFIC SURVEY AT BEKUTIA BRIDGE OVER KHOCHA RIVER AR PEROJPUR-JHALOKATHI ROAD

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ABSTRACT

Ttraffic surveys are required to identify present traffic characteristics of an area as well as to predict future travel pattern in the influence zone of a proposed bridge or road site. This paper presents an approach to and methodology of traffic surveys and data analysis procedure that had been followed for the feasibility study of Bekutia Bridge at Jhalokathi-Perojpur road. A reconnaissance survey and available guidelines showed that this study mainly demands 4 types of traffic surveys such as classified volume count, O-D survey, turning movement and travel time- delay survey. Total traffic comprised of normal,diverted and generated traffic, is the outcome of these survey .Total traffic is then forecasted upto the economic life of the bridge in order to provide adequate capacity for the bridge as well as to assess economic benefits. **.** *Analysis of survey data indicated that at the end of economic life (2048) of the bridge bridge, the AADT value is 43853 veh/day and a two lane bridge is adequate to accommodate the present and future traffic at that location(ADB 2009). The final result is considered to be an important parameter for taking decision whether a bridge is feasible to construct or not.*

Keywords: *Traffic Survey, feasibility study ,travel forecast, AADT, Pavement design*

1. INTRODUCTION

1.1. Background

A traffic survey is needed for existing roads that are proposed for improvement such as pavement expansion or bridge construction. Such surveys may also be useful in prioritizing maintenance activities. Traffic volumes determine both the number of travelers who will benefit from a highway improvement project and, in the case of capacity enhancement projects, the future congestion relief provided by the project.

The Government of Bangladesh represented by Bangladesh Bridge Authority (BBA) intends to construct Bekutia Bridge over Kocha River on Perojpur-Jhalokathi road in order to replace existing ferry crossing along Perojpur–Naikati-Rajapur Road (Z8702). Construction of this bridge is expected to divert and generate increased volume of traffic along the route, remove the ferry crossing, reduce travel time and upgrade socio economic condition of that region.

A traffic survey, forecast and projection would depict the traffic and travel characteristics of the mentioned project site.The study result along with other site information would draw the conclusion whether at this stage, the bridge together with both side approach roads is feasible to construct or not.This paper addresses initiation, nature and extent of traffic surveys conducted at the proposed bridge site as well as the approach to and methodology of data analysis followed for this purpose.

1.2 Purpose of traffic survey

Traffic data collection provides the basis for identifying problems, confirming earlier hypotheses, quantifying the impact of changes, and determining the nature or magnitude of needed improvements. Various types of traffic surveys are required to understand traffic and travel pattern of an area. Classified volume count, Origin destination study of passenger and goods vehicles, turning movement study, travel time delay study, axle load survey are the most common survey types. Parking and pedestrian surveys, household and socio economic surveys, road inventory and condition surveys, Public and IPT operator and user surveys, rail gate closer survey etc are also conducted depending on project type and data requirement. These surveys differ in location, duration, objective and analysis procedure.

The purpose of classified traffic volume count survey is to identify the Average Annual Daily Traffic (AADT) in the existing roads or bridges. Traffic is likely to be the highest in the post-harvest season. But the survey cannot always wait till the post-harvest season, and has to be taken up immediately after a particular project is identified for being included in that year's program. If the traffic survey is done in a lean season, local enquiry should be done to ascertain the post-harvest traffic, particularly of agricultural tractor-trailers, LCVs and trucks, and necessary adjustments for traffic is to be made for seasonal variation.

In order to determine the transportation needs and appropriate solutions for an area it is important to have an understanding of the underlying characteristics of travel. OD survey is required to know the origin and destination of the normal traffic. Data adequacy and reliability, which are absolutely essential to any traffic engineering study, require careful, standardized collection and analysis to ensure valid interpretation and comparability.

Turning movement survey is required to estimate the diverted traffic which is expected to divert from other routes in the influence zone to the proposed bridge or roads after construction. Waterway boat traffic survey is also needed as a percentage of present boat traffic will be converted to motorized traffic on the bridge.

Travel time and delay study determines the amount of time required to travel from one point to another on a given route. Information may also be collected on the locations, durations, and causes of delays. Travel time and delay study has many uses such as efficiency check, problem location identification, evaluation of performance before and after improvement, model calibration, user cost analysis etc. From this study, number of induced traffic due to the opening of the bridge can be evaluated.

In addition, significant data from secondary sources pertaining to demographic, socio economic characteristics, public transport system etc. are also needed to be collected as part of the data collection activity. Past years volume count data helps to evaluate the traffic growth condition, seasonal variation etc

2. METHODOLOGY

2.1 Types of Surveys Conducted at the Bridge Site

Proposed Bekutia Bridge over Khocha River at Jhalokathi-Perojpur Road can ensure smooth and uninterrupted communication between Barisal Division and Khulna Division and other inter-district communication. The bridge might possess a significant role on economic activities over the area. Feasibility study of Bekutia Bridge involves arrangements of traffic survey to establish current traffic situation of the area and to identify the potential of traffic growth.

Ar first, a reconnaissance survey was conducted at the bridge site to determine the traffic analysis zone. Existing road alignments were also recognized to select the points where enumerators would be assigned. Classified volume count surveys were required to conduct at the proposed bridge sites. A ferry is being operated at the location and it is assumed that after the construction of the proposed bridge, present ferry traffic will be converted into bridge traffic.Duration of classified traffic volume survey should be at least 7 consecutive days which would include weekdays and weekend data (TRL 2004, Transportation Planning Analysis Unit 2009) Hourly variation, weekly variation, modal composition, peak and off peak hours of traffic flow were the direct outcome of this survey.

Collected data represents Average Daily Traffic (ADT) for the month of May, when the survey took place. But for design purpose, Average Annual Daily Traffic (AADT) had to be calculated which requires a seasonal factor. For this reason, past yearly volume data was collected from local Roads and Highways Department (RHD). Besides, People were asked about the peak and lean season of that area. It had been found that Traffic load increases in winter season because of tourists and picnic tours. June to August is considered as the peak period for carrying fish and vegetable.(O-D Survey) Finally a factor 1.1 is used to convert ADT to AADT of that area. (Consultancy Services 2003)

O-D survey data of both Passenger and freight vehicles were collected on random sample basis by adopting roadside interview method.The survey was carried out for 16 hours on a weekday covering morning and evening peak hours Assuming the population >100,000 and with 95% confidence level and 5% precision sample size of O-D survey is 400(Glenn D 1992)

A turning movement survey was conducted at major junctions that include Rajarpur junction and Old Bus Stand intersection. This study would help to obtain the directional turning movements of fast moving and slow moving vehicles. The study was continued for 16 hours covering both peak and off peak hours on a typical working day. (TRL 2004, Transportation Planning Analysis Unit 2009)

For travel time delay study, a test car was run considering two common origin and destination, one location is Perojpur and the other is Rajapur (vise-versa).The survey helped to identify vehicle travel time which is required to estimate generated traffic at the location.

2.2 Data Collection Procedure and Resource Management

A reconnaissance survey helped to identify site location of each enumerator based on their objective and data requirement. 12 traffic Engineers were selected based on survey duration and volume of required data. Enumerators were sent at the project site under experienced supervision. Concerned authorities were informed about the survey plan and were requested to help the enumerators if required.

For classified volume count survey, enumerators were assigned at the existing ferry site to count vehicles using ferry service. Four enumerators had collected two-way classified traffic volume data at a time. Each enumerator worked for alternative 6 hours a day and total eight enumerators were assigned for every day count.

For origin-destination survey, three enumerators were given the responsibility for collecting total 400 freight and vehicle origin destination data. Enumerators were guided to survey at ferry parking area and also at nearby intersections where vehicles were stopped with the help of traffic police.

Turning movement study was conducted at Rajarpur intersection and Old Bus Stand intersection which are at the east and west side of the river, respectively. To count turning vehicles at two major junctions, total four enumerators were required. Each enumerator worked for eight hours a day and total 16 hours data at both intersections were collected.

Two surveyors were given the responsibility to carry out travel time delay survey.They determined the time required to travel from one point to other.The survey was conducted at morning peak, evening peak and off peak hour to determine the average travel time.

After collecting adequate data from the field, it was analyzed to establish the baseline data for the proposed bridge and to estimate the potential traffic. A number of assumptions had been made in the following data analysis due to the absence of some required data and also to find a rational outcome of the study.

2.2.1 Locations of Enumerators

Locations of enumerators are shown in the map (local RHD) of Bekutia Bridge site in Fig 1.

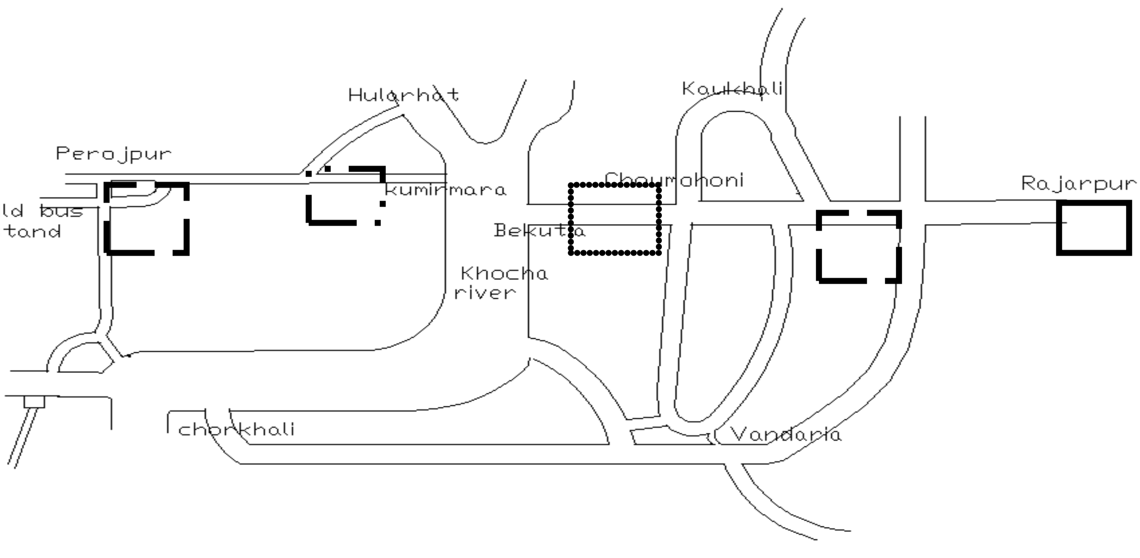


Fig 1: Influence Area of Bekutia Bridge Site with location of enumerators for each survey

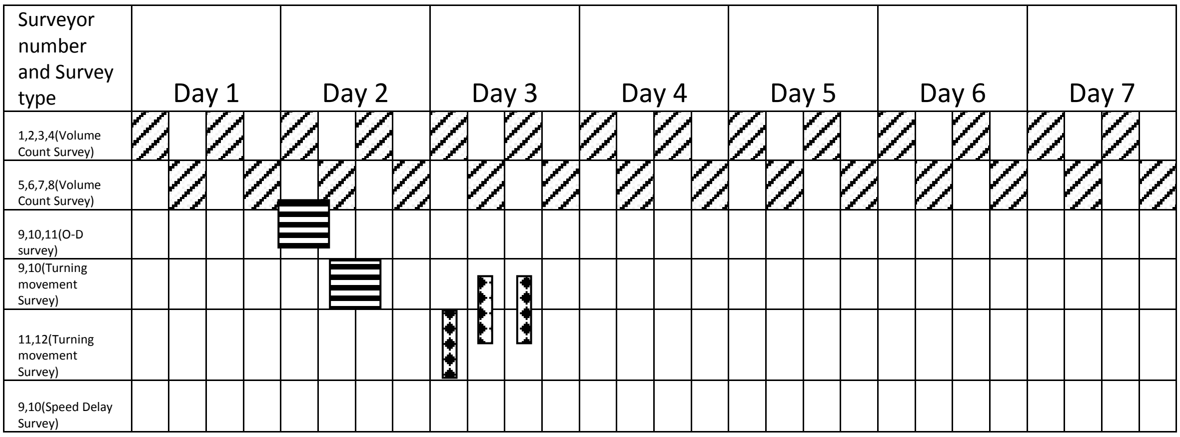
Legend:

Turning Movement Survey
Volume count Survey
Speed delay study
O-D survey

2.2.2 Resource management

Duration and work schedule of each enumerator, illutrated in a bar chart, are shown in Table 1.

Table: 1 Bar Chart Showing Work Schedule of Assigned Enumerators



Legend

6hrs

8hrs

2hrs

3. ILLUSTRATIONS

3.1 Traffic Volume Characteristics

Based on the analysis the traffic volume characteristics can be summarized as follows:

- Commercial traffic in this section of road is lower in comparison to passenger traffic.
- Two-wheeler is significantly high compared to other type of traffic
- Ferry system is mostly used by motorized traffic.
- There is some localized movement of slow moving vehicles on both sides of Kocha River ferry ghats.
- The ADT of weekday traffic is plotted in Graph and is shown in Figure 2 It shows that highest daily traffic is in Mondays and lowest daily traffic is in Tuesdays. Daily traffic in the other days is in between.
- The percentage composition of combined traffic is plotted in Pie Chart and is shown in Figure 3. The figure shows very high percentage of light traffic (car, jeep, pickup, microbus, motor cycle) compared to other traffic. The percentage of non-motorized traffic is very low compared to motorized traffic.

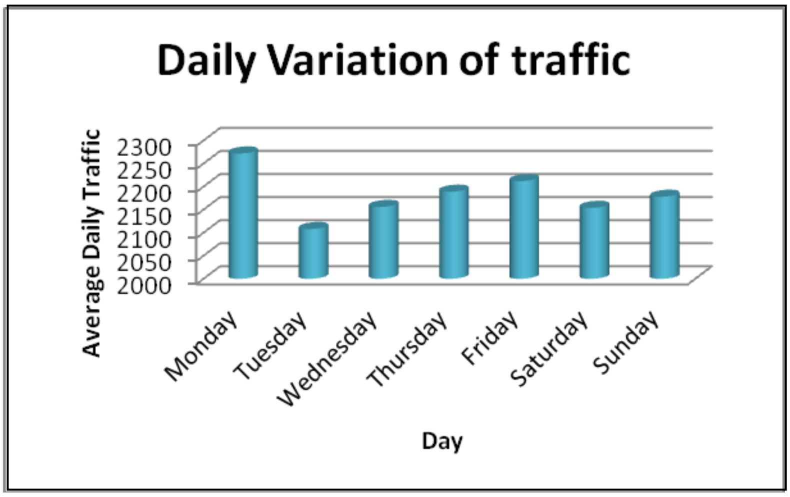


Figure 2: Daily Variation of Traffic

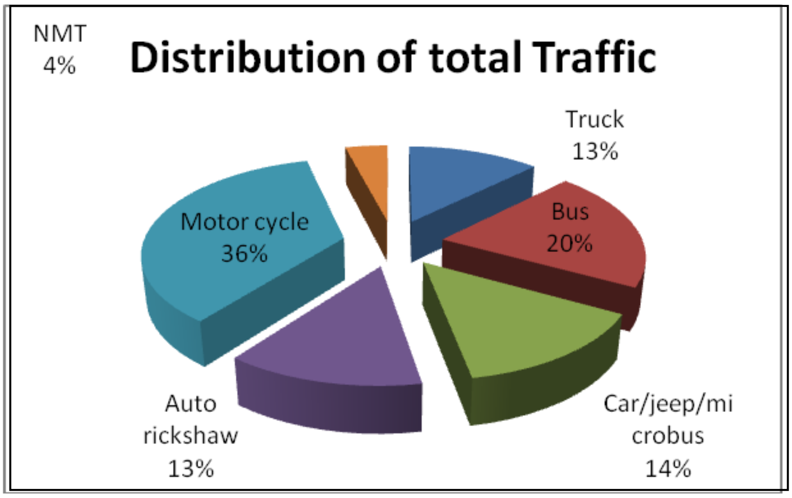


Figure 3: Distribution of Total Traffic

3.2 Evaluation of Potential Traffic at the Location

Traffic forecast up to economic life is one of the major objectives of traffic survey. The forecasts help to inform the many individuals and interest groups who wish to know by how much road traffic can be expected to grow under present policies and give an indication of the effect of measures that they might propose to influence this growth. Overestimation or underestimation of forecasted traffic may lead to undertake non-viable or non-feasible decision.

Future traffic development can be categorized into three groups
1. Normal traffic
2. Generated traffic
3. Diverted traffic

3.2.1 Normal traffic

Passenger and freight traffic, presently crossing the river using ferry services is termed as normal traffic. After opening of the bridge, this traffic will be able to cross the river by bridge and will be transformed into road traffic.

A detailed ferry traffic survey had been conducted at Bekutia Ferry Ghat. It is assumed that 100% of total ferry traffic would be converted to normal traffic of the bridge. (O-D Survey)

3.2.2 Diverted traffic

Traffic which are expected to divert from other routes within the zone of influence to the bridge after opening, are considered as diverted traffic. This traffic will get diverted to the proposed bridge as a result of direct connectivity and due to savings in distance and time. Turning movement survey result helped to predict the percentage of traffic that is assumed to be diverted at the bridge after construction.
Diverted Traffic were found from two sources
i. Diverted traffic from nearby routes
ii. Diverted traffic from waterways

i) Diverted traffic from nearby routes
Turning movement study had to be conducted at nearby junctions of the bridge site. But from the reconnaissance survey, it was found that 90% of total traffic at the nearest Choumohoni and Hularhat intersections do not take any turn and travel directly to the ferry ghat. So,traffic at these locations were not considered in the evaluation of diverted traffic. In order to identify diverted traffic, turning movement count was performed at Rajarpur and old bus stand intersections which were also within the influence zone of the proposed bridge. Turning traffic AADT was calculated and assumed that 100% of buses and trucks and 80% of light vehicles would divert to the new route after the bridge construction (O-D survey).

ii. Diverted traffic from waterways

A considerable amount of boat traffic could be seen at Bekutia Ferry site. A count of engine boats carrying passenger and freight indicated that an average daily number of seventy (35x2) boats are plying 6 times a day carrying 12 passengers in each trip and an average daily number of sixty (30x2) boats are plying 5 times a day carrying a total quantity of 600 tonnes of goods every day across the Kocha river. The movement of the passengers and goods in this mode of river crossing is limited to short distance from the river banks. Thus, an appropriate transformation of the boat trips could be four hundred and twenty (35x2x6) numbers of tempo trips with the same passengers. Similarly, considering small pick-up or tempo carrier of 2.0 tonnes capacity, an equivalent number of 300 light vehicle trips will be required in the traffic survey time reference of May 2011.

Chorkhali Ferry Ghat, within the influence zone, also carries significant number of vehicular traffic.It was assumed that 100% of the total Chorkhali ferry traffic would divert to the proposed bridge as a result of time and cost savings(O-D survey).Chorkhali ferry traffic data was collected from local Roads and highways department.

3.2.3 Generated Traffic

Construction and operation of the bridge at Bekutia over Khocha River would induce significant number of traffic at that area. To determine generated traffic, gravity model was used which states the inverse relationship between traffic flow at a location and trevel time required. (Consultancy Services 2003)

The model formulation can be given as

$$\frac{Qn}{Qo} = \left(\frac{to}{tn}\right)^n$$

The power n is usually taken as 2. .(Consultancy Services 2003)

Where

$Q_0 = Old\ traffic\ flow$
$t_0 = Old\ journey\ time$
$t_n = New\ journey\ time$
$Q_n = New\ traffic\ flow$

Therefore, percentage increase in traffic due to time savings

$$Q_n = Q_0\left(\frac{t_0}{t_n}\right)^2$$

$$\%\ \text{increase} = \frac{Q_0\left(\frac{t_0}{t_n}\right)^2 - Q_0}{Q_0} \times 100\%$$

$$\%\ \text{increase} = \frac{Q_0\left[\left(\frac{t_0}{t_n}\right)^2 - 1\right]}{Q_0} \times 100\%$$

$$\%\ \text{increase} = \left[\left(\frac{t_0}{t_n}\right)^2 - 1\right] \times 100\% \quad \text{...(1)}$$

For this project considering two common origin and destination, one location is Perojpur and the other is Rajapur (vise-versa). Travel time from Perojpur to Kumirmara is 10 minutes, ferry crossing from Kumirmara ferry ghat to Bekutia ferry ghat is 50 minutes and from Bekutia ferry ghat to Rajapur is 30 minutes. So, total time required in the existing poor condition of road is 90 minutes. It is estimated that implementation of the bridge project will reduce the travel time to 75 minutes. Applying these time values, t_o = 90 minutes and t_n = 75 minutes, in the following formula:

$$\%\ \text{increase} = \left[\left(\frac{t_0}{t_n}\right)^2 - 1\right] \times 100\% = 44\%$$

The use of a formula like the equation (1) for determination of generated traffic for the project road with the bridge facility would indicate that enormous traffic (44%) would be induced with the construction of the bridge. In reality this might not happen.

It may be noted that people of our country always tend to avoid ferry service as it adds additional delay to their travel. Loading and unloading maneuver of ferry demands extra times. Besides, if the traffic flow is more than ferry capacity, vehicles have to wait in long queue.Besides, after construction and opening of the Padma Bridge, a boost up in socio-economic development is expected to take place in the south-western region of Bangladesh. This in turn will increase the normal and diverted traffic through the Bekutia Bridge project.

In order to be conservative, this study had considered it reasonable to use a minimum proportional factor for estimating generated traffic as a result of construction of the proposed bridge. For a project like Bekutia Bridge, it was considered rational to use a minimum standard factor of 0.3, which indicated that generated traffic at the opening year of the bridge would be 30% of total normal traffic and diverted traffic of the opening year .(Consultancy Services 2003)

3.3 Traffic at the opening year

In order to evaluate average daily traffic of the area, average value of 7 days traffic volume was taken.Then, a seasonal factor was used to convert ADT into AADT.Turning traffic data was also converted into average daily traffic making necessary adjustments in order to evaluate diverted traffic. Normal and diverted traffic is needed to be forecasted upto the opening year of the bridge considering population growth rate. For this project, it has been analyzed and found that the earliest completion year for this bridge shall be year 2018. The growth rate between the year 2011(base year) and 2018(assumed year of completion of project), had been derived from an accepted population growth rate of Bangladesh (1.4%) for the perspective years (World Bank 2011)

Total traffic at the opening year = Forecasted (Normal traffic + Diverted traffic + Generated traffic)

Table: 2 Estimated Motorized AADT on Bridge in the Opening Year (2018-2019) Following End of its Construction

Sl. No.	Vehicle type	Normal traffic at Opening year	Diverted Traffic *	Diverted traffic **	Diverted traffic ***	Generated traffic	Total traffic
1	Truck	346	87	74	0	153	660
2	Bus	565	120	116	0	241	1042
3	Car/jeep/microbus	389	56	65	330	252	1092
4	Auto rickshaw	362	72	63	463	288	1248
5	Motor cycle	983	103	111	0	359	1556

*Diverted Traffic from Rajarpur-Bhandaria road, **Diverted traffic from Chorkhali ferry traffic, ***Diverted traffic from Boat passenger traffic

3.4 Traffic Forecast

Traffic projection is an essential estimation for computing the trend of future traffic. It is required for design of the bridge, highway pavement and economic analysis of the project. For determining economic viability of a bridge project, it is standard procedure to consider an economic life span of 30 years. So, it is necessary to make a traffic projection up to the year 2048 for this project.There are several approaches for projecting future traffic of an area. These Approaches differ by considerations and assumptions.

i. Theoretical Approach

This approach involves analysis of key socio-economic parameters such as gross domestic product (GDP), transport elasticity demand and different vehicle types, population, economic activities including income accruing to major economic sectors such as agricultural, industry, trade, transport, services etc in a country like Bangladesh.

ii. Practical Approach

Practical approaches of traffic forecast involves traffic growth trend or rates on project road or similar roads and/or average traffic growth rates on national, regional or feeder roads. The ideal approach should be adoption of past traffic growth trends or rates on concerned road or roads. But unfortunately reliable past traffic growth trend or rates are difficult to obtain in this country for particular road or roads.For this reason, an established traffic growth rate from a similar study or from a published documents can be considered.However recent past studies by different consultants on road projects have indicated that on national roads vehicular traffic growth rates have been between 6-8% per annum depending on the strategic/commercial importance of the road infrastructure and comparative role of the type of vehicles moving on the road facility. (Asian Developemnt Bank 2007)

3.4.1 Forecasting Methodology

Traffic forecasting of Bekutia Bridge involves consideration of realistic practical growth rates which was found from Padma Multipurpose Bridge study report.In this study, the actual rate of traffic growth after completion of Jamuna Bridge has been used, which showed a sudden jump in growth for the first 5 years after opening of the bridge and then a slowly reducing growth rates. These growth rates are presented in table3 (Asian Developemnt Bank 2007).The future traffic growth rate that had been used in Padma Multipurpose Bridge study, is considered to forecast traffic at the proposed bridge location.The estimated traffic growth rate is given in table 3

Table: 3 Traffic Growth Rate

Period	Truck	Bus	Light vehicles
2006-13	6%	8%	8%
2014-18	8%	10%	10%
2019-23	7%	9%	9%
2024-28	7%	8%	8%
2029-33	6%	8%	8%
2034-38	6%	7%	7%
2039-46	5%	6%	6%

Source: (Asian Developemnt Bank 2007)

3.4.2 Traffic Projection

Classified total traffic at the opening year(table 2) is forecasted upto the economic life of the bridge(30 Years) using traffic growth of table 3. The projected traffic data has been presented in Table 4.

Table: 4 Forecast of Motorized AADT on Bekutia Bridge at Perojpur-Jhalokathi road over an 30 years

Vehicle year	Truck	Bus	Car/ jeep/micro/pi ck up	Auto rickshaw	motor cycle	Total AADT
2018	660	1042	1092	1248	1556	5598
2019	706.2	1135.78	1190.28	1360.32	1696.04	6088
2020	755.634	1238	1297.405	1482.749	1848.684	6622
2021	808.5284	1349.42	1414.172	1616.196	2015.065	7203
2022	865.1254	1470.868	1541.447	1761.654	2196.421	7835
2023	925.6841	1588.537	1664.763	1902.586	2372.135	8453
2024	990.482	1715.62	1797.944	2054.793	2561.905	9120
2025	1059.816	1852.87	1941.779	2219.176	2766.858	9840
2026	1134.003	2001.1	2097.122	2396.711	2988.207	10617
2027	1213.383	2161.188	2264.892	2588.447	3227.263	11455
2028	1298.32	2334.083	2446.083	2795.523	3485.444	12359
2029	1376.219	2520.809	2641.769	3019.165	3764.28	13322

2030	1458.792	2722.474	2853.111	3260.698	4065.422	14360
2031	1546.32	2940.272	3081.36	3521.554	4390.656	15480
Vehicle year	Truck	Bus	Car/ jeep/micro	Auto rickshaw	motor cycle	Total AADT
2032	1639.099	3175.494	3327.869	3803.279	4741.908	16687
2033	1737.445	3397.778	3560.82	4069.508	5073.842	17839.39
2034	1841.692	3635.623	3810.077	4354.374	5429.011	19070
2035	1952.193	3890.116	4076.782	4659.18	5809.041	20387
2036	2069.325	4162.425	4362.157	4985.322	6215.674	21794
2037	2193.484	4453.794	4667.508	5334.295	6650.772	23299
2038	2325.093	4721.022	4947.559	5654.353	7049.818	24697
2039	2441.348	5004.283	5244.412	5993.614	7472.807	26156
2040	2563.415	5304.54	5559.077	6353.231	7921.175	27701
2041	2691.586	5622.813	5892.621	6734.424	8396.446	29337
2042	2826.165	5960.181	6246.179	7138.49	8900.233	31071
2043	2967.474	6317.792	6620.949	7566.799	9434.246	32907
2044	3115.847	6696.86	7018.206	8020.807	10000.3	34852
2045	3271.64	7098.671	7439.299	8502.056	10600.32	36911
2046	3435.222	7524.592	7885.657	9012.179	11236.34	39093
2047	3606.983	7976.067	8358.796	9552.91	11910.52	41405
2048	3787.332	8454.631	8860.324	10126.08	12625.15	43853

4. LIMITATION OF THE SURVEY

Some difficulties were faced by the enumerators during data collection procedure. During the survey, adequate past year data of the mentioned area could not be found from local authorities. As a result, some assumptions had to be made for forecasting traffic.

As past yearly data were not found, local people were asked about peak, lean, post harvest seasons and necessary factors were used to adjust AADT from ADT.

Besides, 24 hours Volume count is required to calculate average daily traffic of the area. But at the proposed bridge site, Ferry doesn't operate at night and vehicular flow could only be found from 8 am to 9 pm. The same flow condition could not be expected after the opening of the bridge. A factor was also used to convert 13 hours counted traffic to 24 hours flow.Based on O-D survey data and consultant's own estimation,night traffic(from 9 pm to 8am) is considered 30% of the actual day traffic.

As 90% of the total traffic at Choumohoni and Hularhat intersection are through traffic, so turning movement study is conducted at Rajarpur and Old Bus Stand intersections which are also within the influence zone of the proposed bridge.

During origin destination survey, only a few moving vehicles could be stopped and questioned at junctions. Maximum number of total passenger and freight vehicles O-D survey data were found from ferry parking area.

Other than these, weather had played a significant role on data collection procedure. During heavy rain and storm, enumerators could not count vehicles comfortably. Besides, number of vehicles reduced abnormally during rain as the ferry would not be operated at that time. But it doesn't represent the scenario after the bridge opening.

5. CONCLUSION

Manual methods of traffic survey have been adopted to carry out the study.Various automatic devices for measuring traffic flow are now available in our country. These devices give more precise results than the same were found manually. For a similar study where more accuracy is required, automatic devices should be chosen over manual procedure.Similarly, computer modeling for forecasting traffic considering past traffic growth trend, per capita income, elasticity of transport demand to income and population, pattern of economic activity etc. could result an accurate traffic projection. In this study, a limited influence zone was selected and was assumed that traffic of the influence zone will be directly influenced by the bridge construction. But the assumption has its limitation as the bridge has broad impact over the society and the area.After the construction of Padma Bridge a considerable amount of traffic will be induced at the south part of the country.Construction of Bekutia Bridge project will further improve the communication between different districts of Chittagong and Khulna Divisions.

Moreover, minimum percentage of generated and induced traffic is considered for this study. A significant number of traffic can be expected at this location as the new route will provide a direct connectivity and savings in travel distance and time.

For a similar traffic study at different road and bridge sites, an axle load survey should be included in the survey scope to estimate anticipated traffic loading and performance of the pavement structure. Due to technical constraints, axle load data is obtained from O-D survey results during this study. Finally, a land use survey should be conducted simultaneously with the traffic survey, as the land use pattern has the direct relation with traffic generation. The demand generated by tourism, new industries and other development schemes coming into existence during the economic life of the project should be taken into consideration.

The forecasted traffic determines the number of motorized and non motorized lanes and minimum pavement thickness of the bridge . From traffic projection of total motorized traffic it was found that AADT in the year 2048 is 43853 veh/day which exceeds 35000 veh/day, the saturated capacity of 2-lane carriageway (ADB 2009).From the traffic survey, forecast and projection it may be concluded at this stage that the bridge together with both side approach roads is feasible for 2-lane single carriageway roadway.The result of this survey will be further used for estimating benefit stream arising from implementation of the projects.

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TRAFFIC INFORMATION DERIVING USING GPS PROBE VEHICLE DATA INTEGRATED WITH GIS

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ABSTRACT

To evaluate road network performance determination of Congestion Index based on Speed Study is very effective. This paper describes an accurate and reliable method of extracting transportation information using Global positioning system(GPS) receiver integrated with Geographic Information System(GIS). U-BLOX ANTARIS AEK-4H Model Global positioning system (GPS) receiver connected with a car was used as the probe vehicle along the major arterial from Shahbag to Hazrat Shahjalal International Airport, length 14.8 km) in Dhaka city. This probe vehicle automatically collects time, local coordinates and speed at regular sampling periods (1 sec) in NMEA(National Marine Electronics Association) format. The data reduction procedure filters and aggregates Global positioning system (GPS)data to compute travel time and speed along the segments (11 segments based on Intersection). Based on the analyzed data necessary traffic information is derived and presented by Geographic Information System(GIS) (ArcGIS 9.2). The average speed on the arterial was found 14.5 kmph. Mostly free flow condition was in Kuril -Airport segment & congested condition was in Shahbag – Hotel Ruposhi Bangla. The congestion behavior was found unorthodox. Congestion was found almost same throughout the day though it is supposed to be higher at morning and evening peak. It is because of unplanned growing of Dhaka city with no separate commercial and residential zone. Basing on this effective transportation planning and necessary measures can be taken.

Keywords: *Probe Vehicle, GPS, GIS, NMEA, Congestion Index*

1. INTRODUCTION

Evaluating the performance of road traffic system is a major concern to transportation planners for both planning and proper management of transportation facilities. Traffic information deriving has been very effective for evaluating the performance of urban arterials. The urban road network, in most of the cities of developing countries is constantly failing to provide expected Level of Service (LOS) due to improper planning and adoption of inefficient and unsuitable measures. The inability of understanding the factors causing congestion and lack of proper planning to improve the situation is the main reason of this condition. There is also a general conception among the people that overburdened by huge population and extreme poverty causing too much traffic and non-motorized vehicles and also lacking of enforcing laws are responsible for the current congestion. For huge possibility of uncertain situation people have not a clear conception against congestion, travel time and speed. For this reason this research work is a need to assess congestion levels based on some established measure of effectiveness from an engineering point of view. Therefore, the goal of this study is to evaluate a set of GPS probe vehicle data along with GIS (Geographic information system) data to use in developing a system to collect traffic data & derive useful transportation information.

2. LITERATURE REVIEW

Travel demand analysis has played an important role in different parts of the world because of economic & urban development project. Some studies have been done in many countries according to their demand. The studies made in different developing countries that deserve mention and have relevance to the present study are presented in the following section.

Takyi (1990) made a household based travel behaviour analysis that used the cross-classification method in a developing country context. This study was carried out in Kumasi, the second-largest city in Ghana, being a rapidly expanding urban centre (population 700,000) with a growing demand for travel

Levinson (1998) conducted a study regarding speed and delay on signalized arterials. In his study, a model was developed to predict the influence of traffic flow on the running speed of signalized arterials in Maryland, USA, while controlling for link length, the number of lanes, and route type

Daoqin Tong (2004) made a thesis based on the methods for extracting transportation information using global positioning system (GPS) receivers and geographic information system (GIs) technology. Compared to traditional traffic data collection techniques, the new technology provides more accurate information, wide spatial coverage and cost-efficient data to measure vehicle travel time and speed.

An AgGPS 132 DGPS receiver was used to collect probe vehicle data along several highways of interest in Columbus, Ohio for 2002 and 2003. The time and speed records were collected every second. A Franklin County DOQQ (2000) with a ground resolution of 0.5 ft (0.15 m) was used as a backdrop to develop the highway centerline data file. The collected GPS points were mapped to the centerline using a "snapping" technique. Several sets of loop detector data along the highway were also extracted for 2002.

3. METHODOLOGY

An urban arterial in Dhaka City from Shahbag Intersection to Hazrat Shahjalal International Airport (14.8 km) is selected as the study route. The total route was divided into 11 segments mostly based on intersections for the accuracy of the analysis and prediction of road performance as well as assessing the contributing factors behind congestion on different locations. A Toyota private car was equipped with a GPS receiver (Model: U-BLOX ANTARIS AEK-4H) connected with a portable computer. This was used as the probe vehicle to collect necessary information and data to extract required traffic information. The collected data was then analysed and required traffic information was derived. The road network performance was done mainly based on the average speed of the vehicle. The output of the study was then presented by GIS (ArcGIS 9.2 version) by using coloured polygon features. Total methodology has been shown below with a flow diagram.

Route selection

⇩

Data collection with GPS probe vehicle in NMEA format

⇩

Filtration of NMEA data and analysis with MS Excel

⇩

Output shown by GIS

3.1 Data Collection

The data was collected in the week days of 4 consecutive weeks. Data collection was mostly done in the Morning and Evening Peak to evaluate the road network performance in the most extreme circumstances. The probe vehicle collected data in NMEA format consisting $GPVTG, $GPRMC,$GPGSV, $GPGSA, $GPGLL. Amongst them only $GPRMC format data was used to extract travel speed, travel time, local coordinates. The data was then filtered and aggregated to derive necessary traffic information.

3.2 Traffic Information Derived

Some traffic Parameters has been computed here. These are Average Speed, Free Flow Time, Actual Delay, Travel Rate, Delay Rate, Relative Delay Rate, and Delay Ratio.

Some traffic parameters have been also compared here. These are Free Flow Speed vs. Actual Speed, Free Flow time vs. Actual Delay, Travel Rate vs. Delay Rate, Relative Delay Rate vs. Delay Ratio, and Travel Rate vs. Actual Speed.

3.3 Standard for Evaluating Congestion State

For assessing the congestion level, some standards speed limits and travel rate are considered based on earlier studies and urban travel standards which are more representative and applicable for an arterial of a heavily crowded city like Dhaka.

Table 1: Congestion Standards

Travel speed(Kmph)	Travel Rate(min/km)	Congestion State
>35	<1.72	Free Flow Condition
25~35	1.72~2.4	Slightly Congested Condition
15~25	2.4~4	Moderately Congested Condition
7.5~15	4~8	Congested Condition
<7.5	>8	Severely Congested Condition

4. ILLUSTRATIONS

4.1 Derived Traffic Parameters

The travel speed, travel time and local coordinates collected from probe vehicle for 4 weeks are analysed and the following traffic information is produced.

Table 2: Derived Traffic Parameters

Segment Name	**Length(km)**	**Speed(km/hr)**	**Travel Time(min)**	**Travel Rate(min/km)**	**Free Flow Speed(km/hr)**
Shahbag to Hotel Ruposhi Bangla	0.25	6.48	2.35	9.26	35
Hotel Ruposhi Bangla to Banglamotor Intersection	0.51	7.52	4.09	7.98	35
Banglamotor intersection to Karwan Bazar	0.44	7.94	3.36	7.56	35
Karwan Bazar to Farmgate	0.93	11.59	4.83	5.18	35
Farmgate to Bijoy Sarani	0.77	8.55	5.37	7.01	35
Bijoy Sarani to Jahangir Gate	1.25	11.41	6.55	5.26	35
Jahangir Gate to Setu Bhaban	1.43	22.93	3.73	2.62	35
Setu Bhaban to AIUB Intersection	3.37	25.14	8.05	2.39	35
AIUB Intersection to Army Stadium	1.28	12.43	6.17	4.83	35
Army Stadium to Kuril	2.41	15.91	9.08	3.77	35
Kuril to Airport	1.09	30.04	2.17	2.00	35

Table 3: Derived Traffic Parameters

Segment Name	Free Flow Travel Time(min)	Actual Delay Time(min)	Delay Rate(min/km)	Relative Delay Rate	Delay Ratio
Shahbag to Hotel Ruposhi Bangla	0.44	1.92	7.55	4.40	0.81
Hotel Ruposhi Bangla to Banglamotor Intersection	0.88	3.21	6.27	3.66	0.79
Banglamotor intersection to Karwan Bazar	0.76	2.59	5.84	3.41	0.77
Karwan Bazar to Farmgate	1.60	3.23	3.46	2.02	0.67
Farmgate to Bijoy Sarani	1.31	4.05	5.30	3.09	0.76
Bijoy Sarani to Jahangir Gate	2.14	4.42	3.54	2.07	0.67
Jahangir Gate to Setu Bhaban	2.44	1.29	0.90	0.53	0.34
Setu Bhaban to AIUB Intersection	5.78	2.27	0.67	0.39	0.28
AIUB Intersection to Army Stadium	2.19	3.98	3.11	1.82	0.64
Army Stadium to Kuril	4.13	4.95	2.06	1.20	0.55
Kuril to Airport	1.86	0.31	0.28	0.17	0.14

4.2 Comparisons Between Parameters

To determine the relative state of Level of Service at different segments, a comparison is made between the average speed at different segments with the free flow speed in Fig 1.Here, we see, The Kuril to Airport segment has the maximum average speed(30 Kmph) while it is the lowest at Shahbag to Hotel Ruposhi Bangla Segment(6.48 Kmph)

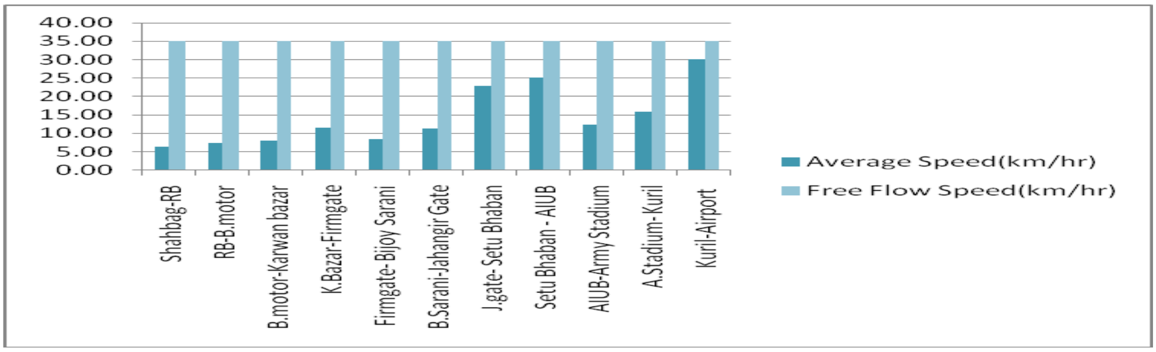

Fig 1: Comparison of Average Speed of different segments with Free Flow Speed

Comparison of Travel Rate and Delay Rate is showed in Fig 2 in order to evaluate how the Travel Rate increases with the increase of Delay Rate. Here, at Kuril to Airport segment, Delay Rate is minimum (0.28 min/km) as well the Travel Rate (2 min/km). While opposite scenario occurs at Shahbag to Hotel Ruposhi Bangla as Delay Rate is maximum (7.55 min/km) here.

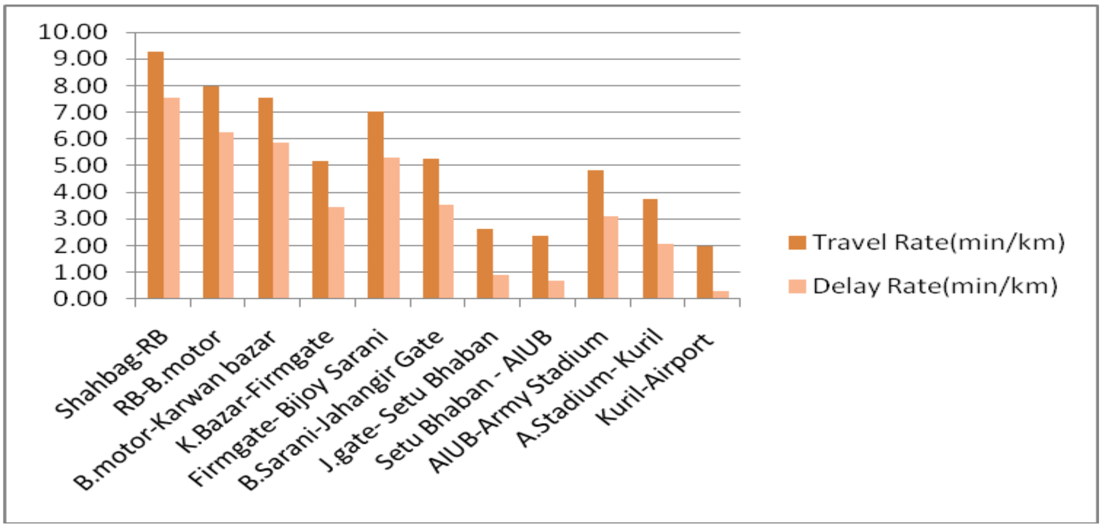

Fig 2: Comparision of Travel Rate and Delay Rate

With the increse of Average Speed the Travel Rate decreses. Because it requires less time to traverse a segment with increased travel speed.Fig 3 shows the comparison between Travel Rate and Average Speed at different segments . As the Average Speed is the maximum (30 Kmph) in Kuril to Airport Segment, travel rate is minimum (2 min/km)here. On the other hand, the decreased Average Speed (6.48 Kmph) at Shahbag to Hotel Ruposhi Bangla segment results in the maximum Travel Rate (9.26 mim/km) here.

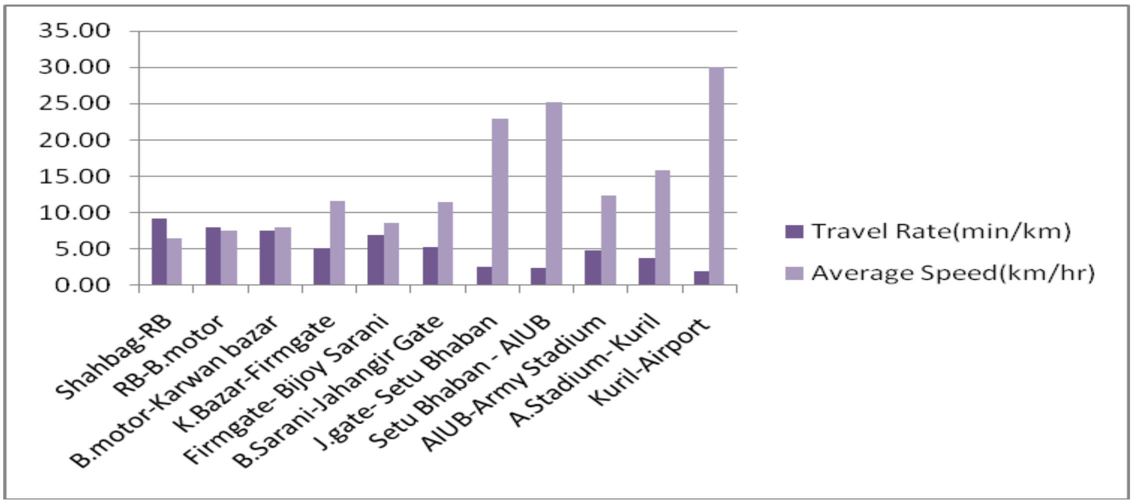

Fig 3: Comparision between Travel Rate and Average Speed

4.3 Determination of Change of Speed with Time

From the Average Speed and travel time, a Speed vs. Time graph is drawn (Fig 4) to determine the required Average time to traverse the study arterial and average change of speed with respect to time.

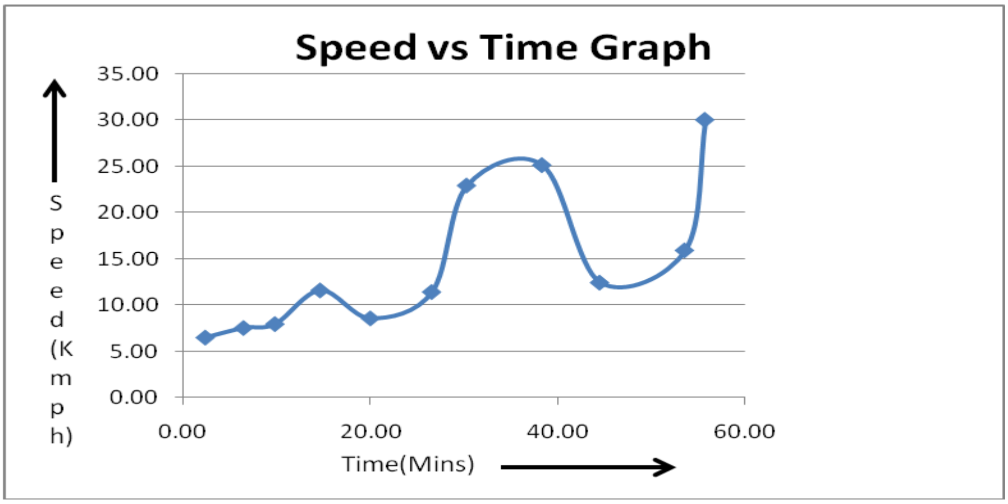

Fig 4: Speed vs Time Graph over the Study Route

From this graph it is can be said that average speed gradually increases from Shahbag to Hazrat Sahjalal International Airport and vice versa. Moreover, the state of congestion at different portion of the study route can also be visualised quite easily and quickly. That is why this Speed vs. Time graph is very important in determining the performance of road network. Hence, for the accuracy of the graph, the collected data need to be accurate. The described approach can meet this requirement very well.

4.4 Presentation of overall congestion state by Geographic Information System (GIS)

The overall congestion pattern is presented in a GIS Map with the help of coloured polygon features.

Legend

Severely Congested Condition (Speed < 7.5 kmph)

Congested Condition (Speed 7.5 to 15 kmph)

Moderately Congested Condition (Speed 15 to 25 kmph)

Slightly Congested Condition (Speed 25 to 35 kmph)

Congestion Free Condition (Speed > 35 kmph)

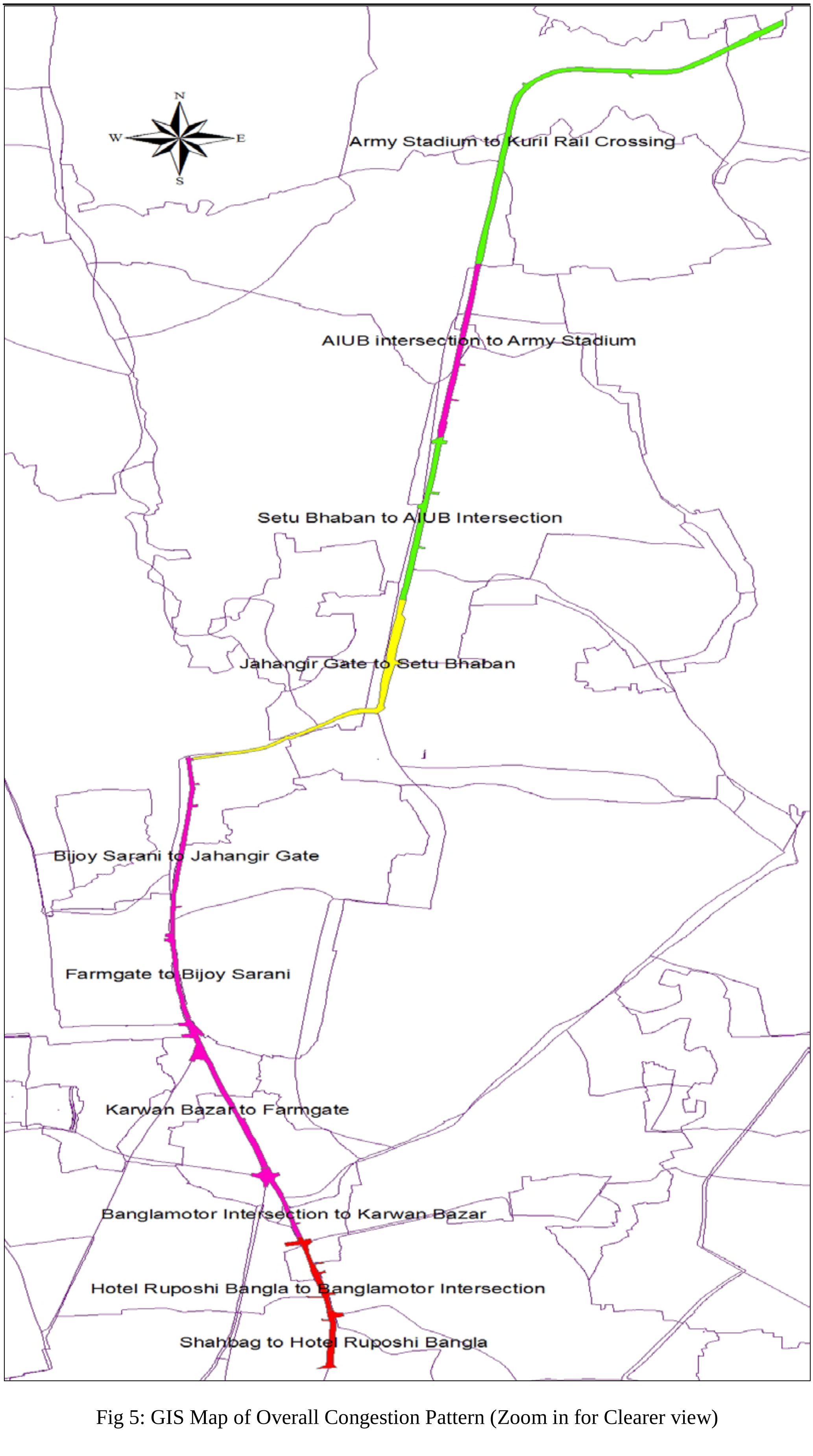

Fig 5: GIS Map of Overall Congestion Pattern (Zoom in for Clearer view)

5. CONCLUSIONS

Transportation data are related with spatial informations. For collecting accurate data GPS was used & for storing & manipulating data GIS was used widely. This was done in this paper that has been described already. One second accuracy was maintained by Antaris 4H GPS with proper altitude, global direction and vehicle speed. With that reasonable accurate data average speed, free flow time, actual delay, travel rate, delay rate, relative delay rate, delay ratio has been computed . After completing analysis total result has been shown byGIS (ArcGIS 9.2 version) by using coloured polygon features. The proposed system offers the opportunity to cost-effectively expand traffic & environment data collection coverage, being a promising tool to medium-size municipalities, usually characterized by severe budget constraints.But proper accuracy loop ditector & other technology for collecting data comparing with GPS may be used. Therefore, proposed methodology can be used in a broader scale for collecting traffic information for cost-effectiveness, convenience and accuracy of result.

ACKNOWLEDGEMENTS

We express our sincere acknowledgement to Dr. Afzal Ahmed, Assistant Professor, Dept. Of Civil Engineering, Military Institute of Science and Technology, Dhaka who provided us the GPS receiver without which conducting this study would not be possible. Moreover, we would like to express our heartiest gratefulness to the MIST authority for the financial and moral support through the entire study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

HEAVY VEHICLE AGGRESSIVITY IN BANGLADESH: CASE STUDY ON LARGE TRUCK

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ABSTRACT

Road traffic accidents are one of the major causes of death, injuries and disabilities in all over the world both in developed and developing countries. The official statistics revealed that the accident fatality rate is very high indicating more than 60 deaths per 10,000 motor vehicles in Bangladesh. Heavy vehicles such as trucks and buses are the major contributors to road accidents and in fatal accidents their shares are thirty three and twenty eight percent respectively. For the case of road death, the share of buses and trucks are nearly 65% and for pedestrian fatalities about 71%. Vehicular structural modification with improper body configuration which increases the aggressivity of the vehicle is one of the major factors of their involvement in accident and severity. In this study, an attempt has been made to find out the factor for reasonably explain this high involvement rate, the author came up with the term 'Heavy Vehicle Aggressivity'. The study found that the active safety index are 6.4 when the heavy vehicle has it's both sides functional mirror and 2.8 when the left side mirror is not functiona and the active aggressivity index is 3.6 and 7.2 respectively. The averge passive i.e. structural and run-over aggressivity index are 5.5 and 4.9 respectively. Vehicular investigation, behavioral attitude observation and focus group discussion has been made to achieve the objective. At the very outset of the study, the involvement of heavy vehicle in accident and their characteristics has been discussed.

Keywords: *Heavy Vehicle, Aggressivity Index, Active Index, Passive Index, Structural Index, Run-over Index*

1. INTRODUCTION

'Vehicle Aggressivity' indicates a certain characteristic of a vehicle where there is a tendency of a vehicle to inflict damage or occupant injury when striking others (Fildes et al. 1993). A comprehensive study has been conducted to find out the factor for reasonably explain this high involvement rate and came up with the term vehicle aggressivity. Vehicular investigation, behavioral attitude observation and focus group discussion has been made to achieve the objective. At the very outset of the study, the involvement of heavy vehicle in accident and their characteristics has been discussed. A field study on heavy good vehicles has been made to find out the present condition of physical characteristics of the heavy goods vehicle in Bangladesh which included the facts as structural modification along with other deficiencies which may like to impart a risk on road traffic safety and also co-relate the physical characteristics of heavy vehicles with the heavy vehicle aggressivity. Finally the study estimated the heavy vehicle aggressivity index of the surveyed vehicles considering the facts and specifications evaluated from the survey manipulation. The active safety index for a general heavy good vehicle considering two critical situations, found out to be 6.4 when the heavy vehicle has it's both sides functional mirror and 2.8 when the left side mirror is not functional and the active aggressivity index are 3.6 and 7.2 respectively. Passive index has been subdivide into two parts, structural and run-over. Using crash involvement and severity of injury data the study found the average structural and run-over aggressivity index of heavy vehicles are 5.5 and 4.9 respectively. These aggressivity indexes provides an overall idea of heavy vehicle aggressivity in Bangladesh, which ultimately defines the key vehicular factors responsible for high involvement rate of heavy vehicles in road traffic accident under one terminology. This paper formed from the part of that broader study on the evaluation of heavy vehicle aggressivity index of the road traffic accident severity in Bangladesh.

2. BACKGROUND OF THE STUDY

In general, the occurrence of an accident due to the combined effect of a number of factors or deficiencies which are associated with road users behavior, vehicular movement, road and roadway environmental conditions.

Accidents are therefore complex events and the elimination of any deficiency associated with the aforementioned may prevent an accident from occurring.

While human error is found to be the most frequent contributing factor to road accidents, in comparison vehicle factors are reported as playing a role much less frequently. As a result, among the contributing factors; Human, Road geometry & Environment factors has been evaluated as factor and provide a great amount of emphasis. In comparison the vehicle factor has been given less priority. Subjectively, however, it appears that in recent times a significant and rapidly increasing number of vehicles suffer from serious blatant defects – such as bad tires and severely cracked windscreens, dangerously customized body. Vehicle factors, therefore, would play much larger role as a contributory factor to road accidents than is currently being reported and it cannot be neglected.

From the analysis of accident data collected from Accident Research Institution of Bangladesh (1998-2009) revealed that heavy vehicles such as trucks and buses are the major contributors to road accidents and in fatal accidents their shares are 30% and 35%, respectively. This group of vehicles is particularly over involved in pedestrian accidents accounting for about 68 percent (bus 38% and trucks 30%). For the case of road death, the share of buses and trucks are nearly sixty five percent and for pedestrian fatalities about seventy one percent.

There are many reasons behind this high rate of accident involvement in heavy vehicles. One of the most important factors is their aggressiveness which is a vehicle factor. Moreover in Bangladesh, the heavy vehicles are customized in such a way that enhances their aggressivity, which may increase the rate of crash involvement and in some cases it surely increases the accident severity.

3. INVOLVEMENT OF HEAVY VEHICLE IN ROAD TRAFFIC ACCIDENTS IN BANGLADESH

The heavy vehicles such as bus and truck are the most involved vehicle in road traffic accidents in Bangladesh. From the detailed analysis of the police reported accident data 1998 to 2009 which has been complied, retrified and modified by the Accident Reseaech Institute (ARI), BUET using Microcomputer Accident Analysis Package (MAAP) 5 software, the following characteristics of heavy vehicle involvement of in road traffic accidents have been found:

- Heavy vehicles are involved in 71% of the accidents and 52% of the casualties.
- The percentage of fatal accidents related with heavy vehicles is 65%.
- The percentage of fatal casualties associated with heavy vehicle is 58%.
- The pick hour of accident occurrence is from 10:00 hours to 12:00 hours and from 12:00 hours to 17:00 hours. At the peak hour the Heavy vehicle accident & casualty rate is also the maximum, and the involvement rates at peak hours are 71.5% in fatalities and 51% in casualties.
- In urban areas, the heavy vehicle accident rate is 69% and rural areas the percentage is 72%.
- The accident involvement rates both are high at national & regional road classes which are 78% and 72%, respectively, comparing to rural and city road classes which are 49% and 67%, respectively.
- The casualty involvement rates are also high at national & regional road classes which are 60% and 53% respectively, comparing to rural and city road classes which are 39% and 40% respectively.
- In all the major routes the heavy vehicle involvement rate is 64-70% in casualty & 74-87% in accident.
- The casualty rate of male and female associated with heavy vehicle is respectively 52.4% and 52.2%.
- In all the age group the heavy vehicle involvement rate in casualty is above 50% and the age groups of 0-16 and 35-45 has the maximum involvement rate 53.5%.
- The rate of pedestrian casualty by heavy vehicle is 68%.

4. PHYSICAL PROPERTIES OF HEAVY GOODS VEHICLES (TRUCK) IN BANGLADESH

To find out the prevailing physical condition of the vehicles riding on the roads of Bangladesh, a survey was conducted on the truck terminal situated in Tejgaon industrial area in Dhaka. About 103 heavy good vehicles were surveyed which more or less depict the current conditions of the heavy good vehicles in Bangladesh. The summary of the findings are provided below:

- Around 47% of the front windshields were dirty or muddy. There might be a possibility that, these windshields might get cleaned before the vehicle hitting the roads. Otherwise, 41% of the front windshields were in good condition & 9% of them were little broken.
- More than 18% of the heavy good vehicles do not have any rear windshield. Among the other which found rear windshield, around 47% are not functional and 17% are dirty or muddy.
- Around 29% vehicles both wipers are working correctly and 26% vehicles the right wiper is working only. Other 45% vehicles wipers are nonfunctional.

- Almost 63% of the heavy good vehicles do not have any front mirror.
- Among the others, around 9% of the right mirrors are broken and 10% are muddy.
- More than 17% of the vehicles do not have any left view aiding mirror. Among the rest, 9% of them are broken and 14% are muddy.
- About 95% of the vehicle have their right mirror altered position & 94% of the vehicle those having left mirror have altered their position.
- Nearly 99% of the left & right headlights are functional.
- Around 96% of the left indicator lights are functional and 97% of the right indicator lights are functional.
- Nearly 44% of the tires are new or in good condition and 56% of the tires are worn out or in bad condition.
- More than 48% vehicle's bumper are extended beyond the width of the vehicle and 52% vehicle's bumper are extended up to the width of vehicle
- The average bumper height is 51.4 inch.
- Around 89% of the vehicles have a sign of hit on the right side of their bumper, 85% have on the left side and 33% have on the middle.
- More than 91% bumpers are highly modified which makes the vehicle more aggressive.
- Around 87% heavy vehicles have sharp angles on the sides of the carrier. 64.1% heavy vehicles have sharp edges on their carrier front.
- Heavy vehicles those having side angles, 91% of them have a sign of hit or slide on their left side and 92% have a sign of hit or slide on their right side.
- Nearly 75% heavy vehicles have their carrier width extended beyond the vehicle width, 61% heavy vehicles have their length extended on the rear side and 21% have their height extended.
- Around 55% heavy vehicles have footboard on both sides, 35% have footboard on one side.

5. AGGRESSIVITY INDEX AND ITS DIFFERENT CRITERIA

The word "aggressivity" or "aggressiveness" is already used in "safety"-literature to indicate a certain characteristic of a vehicle. Newstead et al., 2000 describes the words in the following manner: While "Crashworthiness ratings measure the relative safety of vehicles in preventing severe injury to their own drivers in crashes", the "aggressivity ratings measure the serious injury risk vehicles pose to drivers of other vehicles with which they collide." Further they explain "Aggressivity rating scores estimate the risk of a driver of a vehicle impacting with the focus vehicle being killed or admitted to hospital when involved in a two-away crash."

Very similar explanations are found in other research works. Terhune (1984) defines "aggressiveness," as the tendency of a vehicle to inflict damage or occupant injury when striking others. Gabler and Hollowell define the word like this: "The compatibility of a vehicle is a combination of its crashworthiness and its aggressivity when involved in crashes with other members of the vehicle fleet. While crashworthiness focuses on the capacity of a vehicle to protect its occupants in a collision, aggressivity is measured in terms of the casualties to occupants of the other vehicle involved in the collision." Taking into consideration the definition of linguists, the "Aggressivity Index" is indicating the disregard of others' rights. Perhaps a more appropriate definition is: The "Aggressivity Index" is an indication for the potential threat in relation to others.

5.1 Existing Initial Aggressivity Rating Systems

Existing aggressivity rating systems have been developed by the following international organizations:

1. Transport Research Laboratory (TRL), U.K. (Broughton ,1994; 1996)
2. Road and Transport Laboratory, University of Oulu, Finland (Tapio et al., 1995; Huttula et al., 1997)
3. Monash University Accident Research Centre (MUARC), Australia (Cameron et al., 1998; Newstead et al ., 2000)

All of the three systems have been applied to databases of real crashes from the host countries, but ratings results have been published only the last two countries. The ideas of all these methods are more or less equivalent: The "Aggressivity Index" assumes that in two-car crashes the estimated risk of occupant injury in the other vehicle is a measure of the subject vehicle's aggressivity.

5.1.1 TRL injury risk criterion

Broughton (1994, 1996) proposed an aggressivity rating criterion to be based on Police-reported road accidents in Great Britain, which cover only those in which one or more people are injured. Broughton's aggressivity index and its expected value are:

$$RTK = (Niik+Nnik)/ (Niik+Nink+Nnik)$$

Where,
RTK = Broughton's aggressivity index of the focus vehicle make/model k
Niik= Observed Number of injuries where driver of the focus vehicle model k & other vehicle both injured
Nink= Observed Number of injuries where driver of the focus vehicle model k injured but other vehicle not injured
Nnik= Observed Number of injuries where driver of the focus vehicle model k not injured but other vehicle injured

Boughton's index measures the risk of driver injury in the other vehicle.

5.1.2 Oulu injury risk (absolute) criterion

The University of Oulu have developed aggressivity rating criteria measuring either the absolute or relative risk of the injury to the other driver, but the principal criterion used in the published ratings in Finland is the former. The Oulu criterion reflects the inclusion of non-injury crashes and is defined as:

$$ROK = (Niik+Nnik)/Nk$$

Where,
ROK = Oulu aggressivity index of the focus vehicle make/model k
Niik= Observed Number of injuries where driver of the focus vehicle model k& other vehicle both injured
Nnik= Observed Number of injuries where driver of the focus vehicle model knot injured but other vehicle injured
Nk= Observed Number of total injuries in two-car crashes involving focus vehicle model k

5.1.3 MUARC injury risk and aggressivity rating criteria

The MUARC injury risk criterion, RMk, is the same as the Oulu injury risk (absolute), though in their application to real crash data they are each adjusted for crash exposure differences between makes/models of cars in different ways. The expected value of each injury risk criterion is:

$$E(RMk) = E(ROK) = P2k$$

Where,
E(RMk) = expected value of injury risk criterion
P2k= the average injury probability to the drivers of vehicles colliding with the subject vehicle

The MUARC aggressivity rating is calculated in two steps: (1) other driver injury risk criterion, multiplied by (2) injury severity of the other driver. Injury severity is measured by the proportion of injured other drivers who were killed or severely injured. This rating criterion is considered to measure the risk of severe injury to the other driver in a crash.

5.2 Modified Aggressivity Rating Systems

The study also considered three new criteria for rating the aggressivity of car makes/models:

1. A modification of the Folksam safety rating method (developed by Les and Fildes, 2000)
2. MUARC2 method (developed by Les and Fildes, 2000)
3. MUARC3 method (developed by Newstead, 2000)

5.2.1 Modified Folksam method

The Folksam method is a two-step vehicle safety rating, involving an initial estimate of the relative risk of the driver being injured in focus make/model cars in two-car crashes. This relative risk is then multiplied by a measure of injury severity of front seat occupants to produce the Folksam rating (Hägg et al., 1992).

The modified Folksam method is based on the reciprocal of the Folksam relative injury risk, providing an estimate of the relative risk of injury to the other driver, RFK. The relative injury risk estimate is:

RFK = (Niik+Nnik) / (Niik+Nink)
Where,
RFK = Folksam relative injury risk of the focus vehicle make/model k
Niik= Observed Number of injuries where driver of the focus vehicle model k& other vehicle both injured
Nink= Observed Number of injuries where driver of the focus vehicle model kinjured but other vehicle not injured
Nnik= Observed Number of injuries where driver of the focus vehicle model knot injured but other vehicle injured
The relative injury is multiplied by the injury severity of injured other drivers to provide the modified Folksam aggressivity rating, which is considered to measure the risk of severe injury to other drivers in two-car crashes.

5.2.2 MUARC2 method

MUARC2 is a two-step aggressivity rating method which combines the MUARC and modified Folksam approaches to estimate the relative risk of injury to other drivers. Following MUARC, the other driver injury risks in two-car crashes with the focus make/model, k, and with all other makes/models, t, respectively, are estimated by:

$$RM2k = (Niik+Nnik)/ N$$
$$RM2t = (Niit+Nnit)/ N$$

The relative risk of other driver injury, RM2,is:

$$RM2 = RM2k/ RM2t \quad = (Niik+Nnik)/ (Niit+Nnit)$$

Where,
Niik= Observed Number of injuries where driver of the focus vehicle make/model K & other vehicle both injured.
Nnik= Observed Number of injuries where driver of the focus vehicle make/model K not injured but other vehicle injured.
Niit= Observed Number of injuries where driver of the other vehicle make/model t & all other vehicle both injured.
Nnit= Observed Number of injuries where driver of the other vehicle make/model t not injured but all other vehicle injured.
N= Observed Number of total injuries in two-car crashes involving vehicle model k.

5.2.3 Newstead method (MUARC3)

To overcome the problem of crashworthiness and aggressivity being confounded in these aggressivity rating systems based on two-car injury crashes, Newstead proposed an estimator of the other driver injury risk in crashes with vehicle make/model k, given by:

$$RNK = Niik/ (Niik+Nink) \quad \text{with } E(RNk) = P2k$$

Where,
P1k = the average injury probability to the driver of the focus vehicle model, k,
P2k= the average injury probability to the drivers of vehicles colliding with the subject vehicle
Niik= Observed Number of injuries where driver of the subject vehicle & other vehicle both injured
Nink= Observed Number of injuries where driver of the subject vehicle injured but other vehicle not injured
RNk is an unbiased estimator of P2k and as such is not confounded with the crashworthiness parameter for vehicle model k, P1k.

From the above discussion it could be summarized that:

- The term “aggressivity index” is used in literature to assess the risk for fatalities and/or injuries in the “other” vehicle imposed by the “subject” vehicle
- The term “aggressivity index” is not used so far, to assess the risk for vulnerable road users. The word is only used for vehicle-vehicle accidents.
- The term “aggressivity index” is used in some publications to assess the safety of different cars and constructions based on real life accident data (by contrast to crash tests, which are aiming to assess the “aggressivity” in the laboratory)

Indeed, the term “aggressivity index” in it’s traditional meaning is not applicable or inappropriate for VRU-HGV accidents, for a couple of reasons like:

- VRU-HGV accidents are compared to car-car accidents seldom

- There is broad variety of HGV makes and models available on the market
- There is number of accident HGV-VRU scenarios

Hence, "aggressivity index" in it's traditional meaning is not applicable to assess the aggressivity of a certain HGV make and model in relation to a VRU since the number of casualties is too small to draw sound conclusions. However, it is possible to estimate the aggressivity index for heavy goods vehicles as a collective by statistical approach.

6. HEAVY VEHICLE AGGRESSIVITY INDEX & ITS COMPONENTS

By derivation and definition, the Heavy vehicle aggressivity index estimates the risk of Vulnerable Road Users (VRU) being seriously or fatally injured in accidents involving heavy goods vehicles. The index herewith can be measured experimentally or numerically by obtaining the stiffness and strength characteristics of the bodies in contact. As a result, simple comparisons can be made possible by assessing different HGVs' body structures

By components, the aggressivity index comprises of:

• Active Index (I_{Active}):
  - Direct view
  - Indirect view

• Passive Index:
  - Structural aggressivity ($S_{tructural}$)
  - Run over aggressivity ($I_{RunOver}$)
  - Geometrical aggressivity ($I_{Geometrical}$)

• Active Index: The ability for the accident to be avoided through good visibility and/or active safety systems.
• Structural Index & Geometrical Index: Direct contact between the casualty and the vehicle structure.
• Run-over Index: Risk of the casualty being run over by the HGV.

Since these components are a function of the aggressivity index, a mathematical equation can therefore be formulated for the derivation of a single value that facilitates the comparison between different HGVs. The aggressivity index is

$$AI_{HV} = f\,(\,I_{Active} + I_{Structural} + I_{RunOver} + I_{Geometrical}\,)$$

7. HEAVY GOODS VEHICLE (LARGE TRUCK) AGGRESSIVITY INDEX

Heavy vehicle aggressivity index consists of major two indexes, which are

- Active index
- Passive index

7.1 Heavy Vehicle (Truck) Active Aggressivity Index

For the determination of active index, methods described in Feist et al.(2009) have been followed and for two critical situations of a general Heavy Goods Vehicle the active indexes has been calculated for comparison. Two critical situations are:

1) Heavy vehicle with both of it's side mirrors functional
2) Heavy vehicle with it's left side mirror not functional

The method by Fiest et al. (2009) proposed for comparing the driver's field of view by considering four basic components:

- Areas only seen directly; (Direct Field of View)
- Areas which can be seen directly and at least through one mirror; (Direct Field of View)
- Areas only seen through the various mandatory mirrors; (Indirect Field of View)
- Blind spots /areas not visible at any time

The assessment is carried out using CAD software packages.The current proposal is restricted to the assessment of cab-over-engine HGVs and includes only the primary structures affecting the visible area (edge of glazing and mirrors). Obstructions due to the dash board or the steering wheel were not considered.

The index evaluates the field of view separated into:

- A primary area of interest (in the close surroundings of the vehicle) and
- A secondary area of interest (>5m away from the right front edge of the HGV)

The areas mentioned above are combined as shown in Figure 1 & Figure 2 and the following areas are calculated for both the PAI and SAI:

- Blind spots;
- Mirror view only;
- Direct view only; and
- Combined mirror and direct view

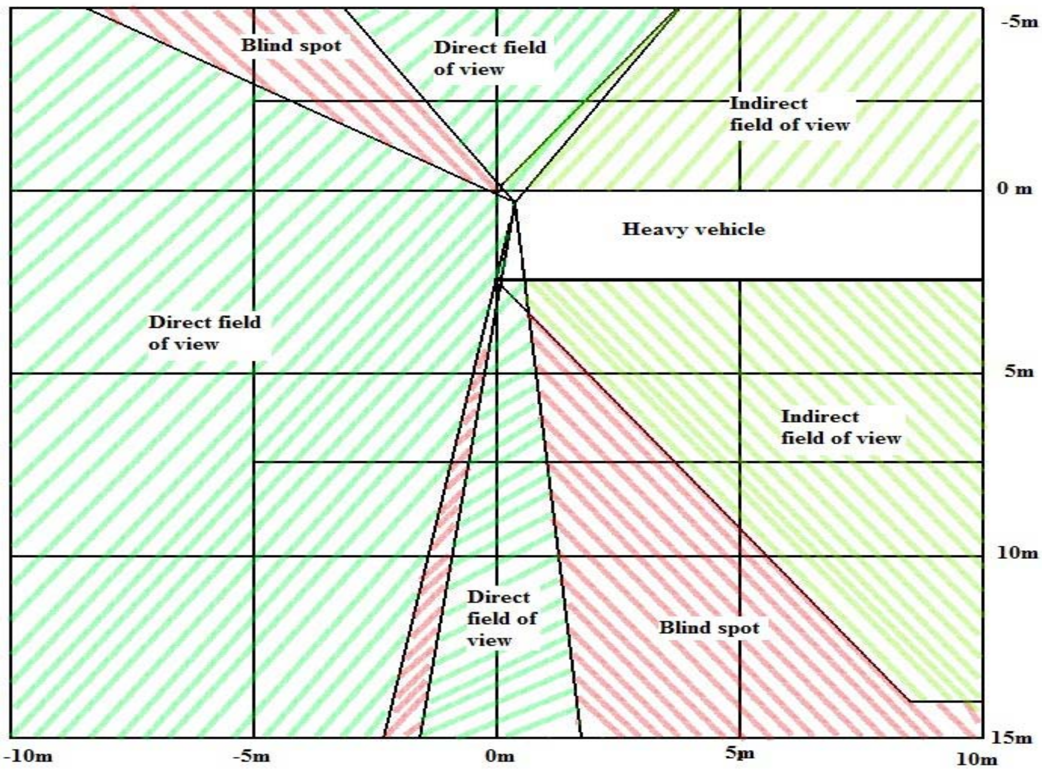

Figure 1: Diagram combining direct and indirect fields of view and blind spots when both the side mirrors are working

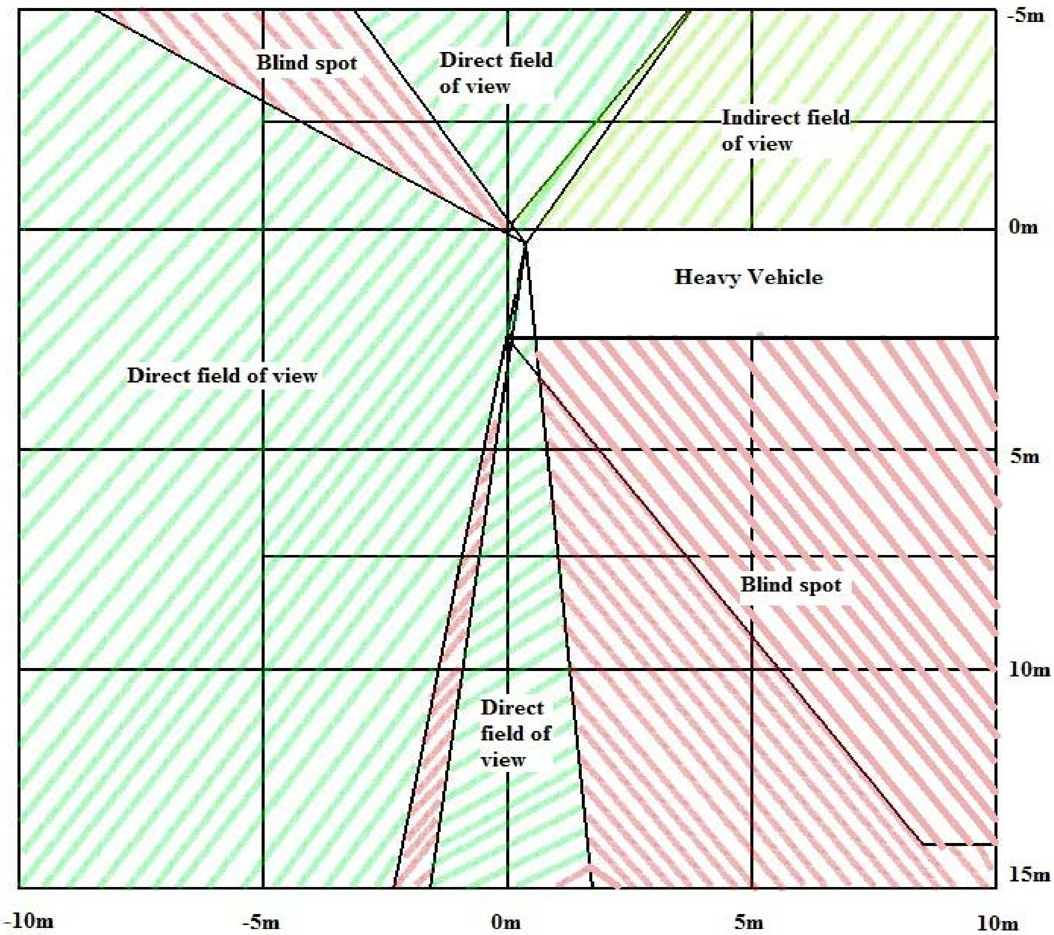

Figure 2: Diagram combining direct and indirect fields of view and blind spots when left side mirror is not working

To allow for the relative importance of the different view areas with respect to the safety of VRU, weighting factors were applied:

- Blind spots in the PAI were considered more relevant than those in the SAI because of the close proximity to the vehicle. These were therefore upgraded by 20% by applying a weighting factor;
- It is often more difficult to correctly identify an object in a mirror than through the windshield (direct view). Therefore areas that were only visible in mirrors were downgraded by 20%;
- Mirror views in the SAI were downgraded a further 20% because the mirror views at that distance are even more difficult to interpret.
- Where the direct and indirect views overlap there is an increased probability that the driver will be able to correctly identify the VRU, for example if part of the head is visible through the window and part of the leg is in the view given by a mirror. However, because this area is also part of the direct view, a neutral weighting is applied.

Finally, area has been measured using the Figure 1 and Figure 2 and weighted areas has been calculated multiplying the factors and the percentages of different field of views are determined which are shown in Table 1.

Table 1: Calculated areas for two situations

Situations	Primary Area of Interest (PAI)					Secondary Area of Interest (SAI)			
	Field of view	**Direct view**	**Mirror view**	**Blind spot**	**Total**	**Direct view**	**Mirror view**	**Blind spot**	**Total**
	Factor	1.00	0.80	1.20		1.00	0.64	1.00	
Both side mirrors are working	**Area (m^2)**	45.88	65.79	12.13	123.80	165.0	57.95	28.85	251.80
	Weighted area (m^2)	45.88	52.63	14.56	113.07	165.0	37.09	28.85	230.94
	Percent	40.6%	46.5%	12.9%	100.0%	71.4%	16.1%	12.5%	100.0%
Left side mirror not working	**Area (m^2)**	45.88	19.30	58.62	123.80	165.0	18.25	68.55	251.80
	Weighted area (m^2)	45.88	15.44	70.34	131.66	165.0	11.68	68.55	245.23
	Percent	34.8%	11.7%	53.4%	100.0%	67.3%	4.8%	28.0%	100.0%

Modifier points are awarded to vehicles where the following criteria apply:

1. If there is an overlap area of direct view and indirect view by one of the mirrors in the PAI around the vehicle. modifier of 3 will be awarded;
2. If there is an overlap of at least 100mm between direct view through the nearside window and the mirror view. The modifier number increases to a maximum of 6.
3. If there is a point where the overlap of the direct view through the left window and the left wide angle-mirror view. The modifier number increases to a value of up to 10.

Finally score from 0 to 10 to be applied to each of the four parts (direct view PAI, blind spot PAI, direct view SAI and blind spot PAI) from the benchmark criteria of PAI and SAI for active index assessment as shown in Table 2. These four values are then combined with the modifier score and averaged to give the Active HVAI score out of 10 as shown in Table 3.

Table 2: Benchmark criteria of PAI & SAI for Active Index Assessment procedure

PAI	Direct View	Blind Spot	SAI	Direct View	Blind Spot
10	58%	4.0%	10	80.0%	9.0%
9	55.5%	5.5%	9	77.5%	10.5%
8	53.0%	7.0%	8	75.0%	12.0%
7	50.5%	8.5%	7	72.5%	13.5%
6	48.0%	10.0%	6	70.0%	15.0%
5	45.5%	11.5%	5	67.5%	16.5%
4	43.0%	13.0%	4	65.0%	18.0%
3	40.5%	14.5%	3	62.5%	19.5%
2	38.0%	16.0%	2	60.0%	21.0%
1	35,5%	17,5%	1	57,5%	22,5%

Source: Fiest et al.(2009)

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Table 3: Determination of active index

Situations	Both side mirrors are working	Left side mirror is not working
Modifier	10	6
Direct View PAI	3	1
Blind Spot PAI	4	1
Direct View SAI	7	5
Blind Spot SAI	8	1
Sum	32	14
Active Safety Index	6.4	2.8
Active Aggressivity Index	3.6	7.2

7.2 Heavy Vehicle Passive Aggressivity Index

Passive aggressivity index also consist of two indexes, which are:

- Structural index
- Run-over index

For the determination of heavy vehicle passive aggressivity index the methods described by Feist et al.(2009) cannot be followed due to limitations of getting access to such lab arrangement and software required for that. Moreover, the heavy good vehicles in Bangladesh have some typical criteria differing from other countries which demand a different approach than the previously mentioned.

To find out a convincing aggressivity equation had to go back to the general aggressivity rating systems. Based on, the Oulu injury risk (absolute) criterion, a modified equation is developed which includes the relative injury risk of all the contributing factors which will influence the structural and run-over aggressivity of the heavy vehicle.

The modified equation for determining Structural index is:

$$SI = [W_B*(BH+BW) + W_A*AL + W_E*EL + W_M*M*(\sum W_{ext})]*C_{SI}$$

Where,
SI = Structural index
W_B = Weightage factor for bumper = 0.629
W_A = Weightage factor for sharp angles = 0.072
W_E =Weightage factor for sharp edges = 0.266
W_M = Weightage factor for mass = 0.911
W_{ext} = Weightage factor carrier size extension
= 1.20 for carrier length extension
= 1.20 for carrier width extension
= 1.67 for carrier height extension
BH = Bumper height in inch
BW = Bumper width in inch
AL = Total length of sharp angles on carrier body
EL = Total length of sharp edges on carrier front
M = Vehicle service load in ton
C_{SI} = Conversion factor for structural index = 0.042

This conversion factor is applied to convert the structural aggressivity index within a value of 10. To find out this conversion factor, a hypothetical heavy vehicle is considered which contains all the above mentioned variables to the maximum possible value which results in maximum aggressivity value and then conversion factor required to convert that value into 10 is calculated.

Following are the equations for determination of Weightage factors,where each weightage factor represents the relative risk injury due to each contributing components of heavy vehicle according to the Oulu injury risk (absolute) criterion.

$$W_B = \frac{\sum HV\ casualty(Head-on\ collison+Right\ angle+Obejcet\ hit\ 1+Object\ hit\ 2+Pedestrian+Parked\ vehicle)}{Total\ HV\ casualty}$$

$$W_A = \frac{\sum HV\ casualty\ (\ Side\ hit + Parked\ vehicle\ )}{Total\ HV\ casualty}$$

$$W_E = \frac{\sum HV\ casualty (Head-on\ collision + Rear-end\ collision + Parked\ vehicle)}{Total\ HV\ casualty}$$

$$W_M = \frac{\sum HV\ casualty (Head-on + Rear-end + Right\ angle + Obj\ hit\ 1 + Obj\ hit\ 2 + Pedestrian + Parked\ vehicle)}{Total\ HV\ casualty}$$

The modified equation for run-over index is the following,

$$RI = [(W_{BT}*BH + W_{BT}*M_T*N_{WT})*C_{RI}]$$

Where,
RI = Run-over index
W_{BT} = Weightage factor for bumper & tire = 0.637
M_T = Total vehicle service load on tires in ton
BH = Bumper height in inch
N_{WT} = Numbers of worn-out tire
C_{RI} = Conversion factor for run-over index = 0.107
This conversion factor is applied to convert the run-over aggressivity index within a value of 10.To find out this conversion factor, a hypothetical heavy vehicle is considered which contains all the above mentioned variables to the maximum possible value which results in maximum aggressivity value and then conversion factor required to convert that value into 10 was calculated.

Following is the equations for determination of Weightage factor,where the weightage factor represents the relative risk injury due to contributing components of heavy vehicle according to the Oulu injury risk (absolute) criterion.

$$W_{BT} = \frac{\sum HV\ casualty (Head-on\ collison + Rear-end\ collision + Right\ angle + Pedestrian + Parked\ vehicle)}{Total\ HV\ casualty}$$

Structural index and Run-over index for all the 103 surveyed heavy vehicles have been determined by applying the above mentioned empirical equations. Table 4 is representing the average, maximum & minimum structural and run-over aggressivity index.

Table 4: Average, maximum & minimum value of structural & run-over aggressivity index

Category	Structural Aggressivity Index	Run-over Aggressivity Index
Maximum	6.8	7.6
Minimum	3.2	2.7
Average	5.5	4.9

8. CONCLUSIONS AND RECOMMENDATIONS

Heavy vehicle in particularly goods vehicle are the major risk factor in road traffic accidents and injuries in Bangladesh. High aggressivity due to the modified physical configuration in particular is one of the major concerning reasons for their high involvement and utmost severity in accidents and injuries. The study evaluated the aggressivity index of the heavy goods vehicle based on a modified equation using the Oulu injury risk (absolute) criterion, which includes the relative injury risk of all the contributing factors and which influence the structural and run-over aggressivity of the heavy vehicle. Indeed, the heavy good vehicles in Bangladesh have some typical criteria differing from other countries which demand a different criteria and method for the evaluation of the aggressivity of the vehicles. Therefore, to find out a convincing aggressivity parameter for different types of vehicles in different conditions, there need to develop a locally calibrated model adjusting with the prevailing configuration and local peculiarities which demands extended and in-depth research in future.

After all, this study suggested the further following recommendations for the safety improvement on the heavy vehicle on the road:

- Most of the heavy vehicles in Bangladesh do not follow the standard rules or regulation determined by Bangladesh Road Transport Authority (BRTA).
- Conforming to vehicle safety standards is necessary to ensure that unsafe vehicles are not imported and to develop a safety culture among vehicle operators, owners, and users.
- Heavy vehicles must be backed by adequate roadside checks in order that the overall standard of vehicles is gradually raised. Random roadside inspection checking should be introduced involving the police and vehicle inspectors day and night to encourage compliance with safety standards.
- Considering poor maintenance practice and aging vehicular fleet, introduction of six-monthly fitness checking could be most appropriate than the current yearly fitness practice.
- Heavy vehicle aggressivity is an uncommon and less known term in Bangladesh. This term should be properly depicted to researchers, policy makers, law enforcers, vehicle owners, manufacturers and mostly the drivers.
- The term "Heavy vehicle aggressivity" should be introduced in the standard rules and regulation of BRTA.
- Each heavy vehicle must be measured it's aggressivity before it gets permitted to run on the road.
- There is a need for the introduction of computer based checking system to make the service more reliable, quicker and most importantly to eliminate subjective judgment of vehicle inspectors and thereby to reduce the scope of illicit financial practice of the fitness issuing system.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

IMPACT OF DIRECT LOCAL ACCESS ROAD ON THE CLASSICAL NATIONAL HIGHWAYS IN BANGLADESH: A CASE STUDY OF DHAKA-MAWA HIGHWAY

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ABSTRACT

The highway system of Bangladesh faces enormous challenges of providing equitable level of service and act as accident prone highway system for the absence of appropriate policy, absence of guidance and control on road configuration which results lack of functional and classical hierarchy of road network, huge road and road side hazards which is induced from the unrestrained road side development, lack of control of direct connectivity of local road with the highway, etc. Dhaka-Mawa highway facing enormous operational, management safety and mobility problems for its 126 direct local access roads and 90 single and multi drive-way within its 36 km of roadway. Every year around 20 accidents are recorded by the police in this segment though the actual number is much more than that which is manifested from the on-site observation and local people observation. Road environmental and operational conditions as well as traffic safety situation are worsening mainly due to adverse impact of huge numbers of direct uncontrolled local access road with the highway. In this paper, an attempt has been made to evaluate the impact of direct local access road on this highway segment using different techniques and methods. At the outset of the paper, the highway configurations system, access management technique and Bangladesh practices are also discussed.

Keywords: *Local access, access management, impact, vision obstruction, mobility, safety*

1. INTRODUCTION

Bangladesh is one of the developing countries of the Asian subcontinent foreseeing many problems in the various sectors among them transportation sector is seemingly the worst. Transport progresses have been driven in Bangladesh mostly by impulsive considerations having no clear-cut focus on future necessities and the means of meeting these requirements on a competitive as well as sustainable basis. The unplanned combination of rapid urbanization and motorization has been a key cause of abundant transport problems in Bangladesh. The rapid urbanization process, high vehicular population growth and that of the mobility, derisory transportation facilities and policies, varied traffic mix with over concentration of non-motorized vehicles, absence of dependable transport system and inadequate traffic management practices and parking facilities have created a noteworthy worsening of traffic and environmental problems (Hoque et al., 2009). The so called highway system of Bangladesh faces enormous challenges of providing equitable level of service and act as accident prone highway system for the absence of appropriate polices, guidance and control on road configuration of road network system results lack of functional and classical hierarchy of road network, huge road and road side hazards which is induced from the unrestrained road side development, lack of control of direct connectivity of local road with the highway etc.

Dhak-Mawa highway, a part of the national highway N-8 (6 to 38 km post) is one of the rising and crucial gateways of capital city Dhaka already facing enormous operational, management safety and mobility problems for its 126 direct local access road and 90 single and multi drive-way within its 36 km of roadway (Field Survey, 2011) . Interaction of local and through traffic, huge pedestrian activity, obstructed vision, road and road side hazard, on road loading and unloading, frequent turning, reduction of main carriage, recurrent interruption of traffic flow, mobility lose, huge accidents and casualties, loss of life and property, superimposed non-standard speed hump etc are the major consequence of the uncontrolled direct local access with the broad band national highway. Every year around 20 accidents are recorded by the police in this segment though the actual number is much more than that which is manifested from the on-site observation and local people observation (Goni, 2011). In this study, the main focus has been provided on the impact of the direct connectivity of local access with the national highway and urgent and long term requirements have been suggested for the improvement the safety as well as functionality of the highway system in Bangladesh. This study will be a concern for the consultants to implement an effective and efficient highway system for Bangladesh in near future.

2. HIGHWAY CONNECTIVITY CONFIGURATION

Highways are commonly classified in one of two ways: *by purpose* or *by ownership*. Functional classification is the process by which highways are grouped into classes, or systems, according to the *purpose*, or the character of service they are intended to provide. This process recognizes that most travel involves movement through a network of highways and defines the nature of this traffic channelization process. This is done by defining the role a particular highway should play in serving the flow of trips through the highway network. The major concept behind the functional classification system is the relationship between highways and the functions they serve. Once the major function of a highway is established, its classification is more easily determined.
In general, the two basic functions of a highway are:

1. Access to property
2. Travel mobility

Almost all highways perform both of these functions, but in varying combinations. The determination of these relationships (basically a determination of the highway's principal use) will also determine to which of the three major functional classifications the highway belongs. These three major classifications are arterial, collector and local. In general, the access/mobility relationship of these three major classifications is as follows:

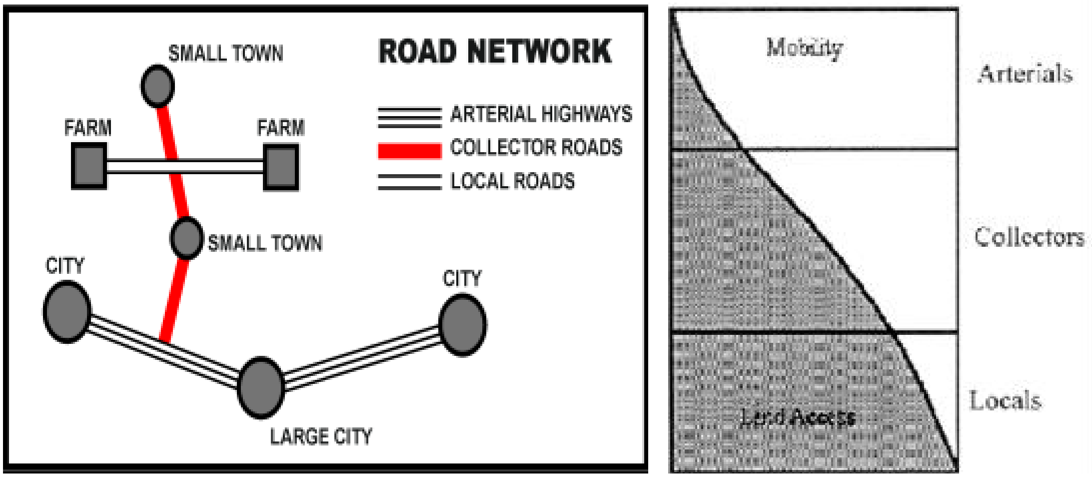


Figure 1: Road Network Types with Functional Classification (AASHTO, 2010)

A major arterial is connected to a collector road and a collector road is connected with both arterials and local roads. A collector road serves for both accessibility and mobility. Arterials serve for the most mobility and local roads as the most accessibility. Collectors act as the transition of accessibility towards mobility. Direct access of local roads or connection of local road the national highway without transition could be termed as a short circuit in which circuit breakdown is apparent. This ridiculous phenomenon is recurring almost all of the road network system in Bangladesh. Indeed, there need special treatment like corner widening, merging diverging lane, appropriate sign, marking etc. in the connecting point of two roads which is pointed out below:

2.1 Access Point Design

Design of access point is same as the design of junction between roads. Continuous research and experimentations is needed before junction design details can be ascertained for handling the traffic in a safe and efficient way.

Key points to consider:

- Cross roads tend to have a high rate of accidents, where staggered T-junctions are much safer.
- Y-junctions are also dangerous and should not be used.
- T-junctions performs best when the minor roads meet the major road with 90º.
- Corner radii at T-junctions can be between 6m to 10m depending on the turning traffic and volume of traffic (AUSTROADS, 2004).

Some of the types of junctions:

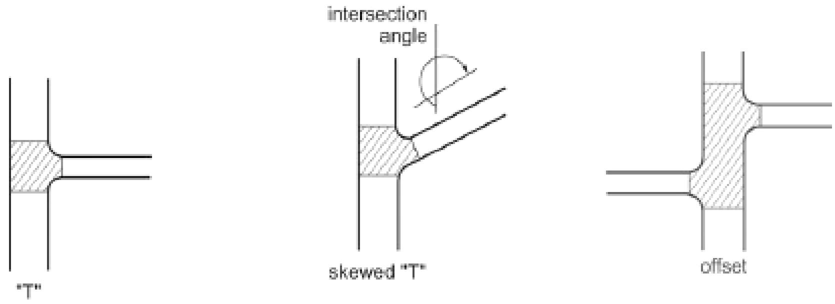


Figure 2: Types of junction (not in scale) (AUSTROADS, 2004)

2.1.1 Design principles:

- Designing for all road user including the NMVs and pedestrians.
- Minimizing the traffic conflicts.
- Ensuring good visibility.
- Keeping the paved area to the minimum needed for maneuvering.
- Designing for appropriate speeds at junctions.
- Ensuring clear and simple layout.
- Provision of adequate signs and markings (AUSTROADS, 2004)

3. ACCESS MANAGEMENT

Access management is the process of providing safe, efficient ways of getting on and off our streets and highways. The concept, “concentrates on restricting the number of direct accesses to major surface streets, providing reasonable indirect access, effectively designing driveways, and enforcing safe and efficient spacing and location of driveways and signals,” according to Ron Giguere, former chairman of the Transportation Research Board Access Management Committee. “There are a variety of techniques available for achieving access control. These include geometric design considerations such as medians and channelized islands that prohibit certain turning movements, consolidation actions such as shared driveways and service roads, and others such as removal and relocation of existing access and the introduction of auxiliary lanes for left and right turns. If these types of improvements are implemented correctly, users can expect smoother vehicle flow, reduced delay and fewer crashes.

These benefits equate to larger aggregate cost savings in travel time, fuel consumption, property damage and injuries. In addition, there is potential for expanding market area for local businesses, reducing vehicular emissions and fostering quicker emergency response.”

Access management can support property values by preventing situations where on street congestion blocks drivers from entering and exiting driveways. Likewise, good access management maintains the effective service area of businesses by controlling congestion, making it possible for more drivers to reach the business within an acceptable time.

Access Management concentrates on:

- Restricting the number of direct access to major highways
- Providing reasonable indirect access
- Effectively designing driveways
- Enforcing spacing and location of driveways

Access management techniques should address the following areas:

- Facility Hierarchy
- Intersection and Interchange Spacing
- Driveway spacing
- Traffic signal spacing
- Median treatments and median openings
- Turning lanes and auxiliary lanes
- Street connections(TRB, 2003).

3.1 Access Management Techniques

Access Spacing: Increasing the distance between traffic signals improves the flow of traffic on major arterials, reduces congestion, and improves air quality for heavily traveled corridors.

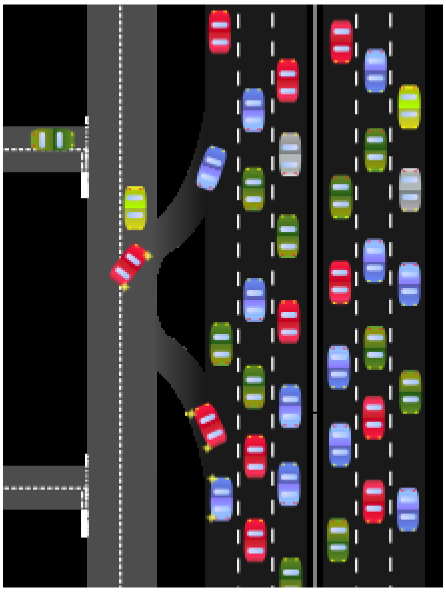
Driveway Spacing: Fewer driveways spaced further apart allows for more orderly merging of traffic and presents fewer challenges to drivers.

Safe Turning Lanes: Dedicated left- and right-turn, indirect left-turns and U-turns, and roundabouts keep through-traffic flowing.

Median Treatments: two-way left-turn lanes (TWLTL) and non traversable, raised medians are examples of some of the most effective means to regulate access and reduce crashes

Frontage roads: A frontage road (also access road, service road, and many other names) is a non-limited access road running parallel to a higher-speed road, usually a freeway, and feeding it at appropriate points of access (interchanges). In many cases, the frontage road is a former alignment of a road already in existence when the limited-access road was built.

In other situations they may be built prior to construction of the limited-access road. In urban areas, frontage roads are frequently one-way roads when they exist on both sides of a highway. In more rural ones, such roads are typically two-way.



Fiure 3: Service Road

Frontage roads provide access to homes and businesses which would be cut off by a limited access road and connect these locations with roads which have direct access to the main roadway. Frontage roads give indirect access to abutting property along a freeway, either preventing the commercial disruption of an urban area that the freeway traverses or allowing commercial development of abutting property. At times, they add to the cost of building an expressway due to costs of land acquisition and the costs of paving and maintenance.

However, the benefits of development nearby real estate can more than offset the cost of building the frontage roads. Furthermore, a frontage road may be a part of an older highway, so the expense of building a frontage road may be slight. And finally, the cost to purchase access rights from adjacent property may exceed the costs to build frontage roads. Conversely, the existence of a frontage road can increase traffic on the main road and be a catalyst for development; hence there is sometimes an explicit decision made to not build a frontage road. A backage road is a similar concept, but lies on the back side of the land parcels that abut the controlled access's right of way. Like the frontage road, it serves mainly to provide access to those parcels as an alternative to a frontage road.

Access Consolidation : Adjacent properties abutting major roadways should be encouraged to share a common approach road connection. This will reduce the number of conflict points and separate the conflict areas. The longer spacing between approach road connections will also facilitate the provision of right-turn deceleration bays. The smoother traffic flow on the abutting street will help reduce vehicular crashes and increase egress capacity (TRB, 2003).

Joint access and inter parcel circulation (cross access easements) can be readily implemented in the subdivision approval process. Once subdivision has already occurred, adjacent property owners may be encouraged to share a common access where it can be shown that customer convenience and safety can be improved. Reconstruction, which adds a non traversable median or median opening modification, offers opportunities for encouraging joint access agreements.

Cross access easements that permit on-site circulation between adjacent properties decreases the number of vehicle trips that would normally use the abutting roadway.

Providing for inter parcel trips can reduce traffic volumes on the main roadway and, as standards should be required to provide for joint and cross access easements, wherever feasible. Abutting properties under different ownership are encouraged to comply but generally not required until they redevelop or expand. In the meantime, the applicant should be allowed a temporary driveway.

Flexibility is needed on an administrative level to work with the unique circumstances of each development site. Communities could relax driveway spacing standards for properties that agree to consolidate access, and provide for variances where compliance proves impractical. Some ordinances provide incentives, such as density bonuses, for combining access points, or relax parking and dimensional requirements where necessary to achieve shared access.

The inter parcel circulation benefits the public and patrons by providing safer circulation. It benefits the private development by making it more convenient to attract patrons. This convenience and safety helps to attract more business to the area and hence, to each individual business

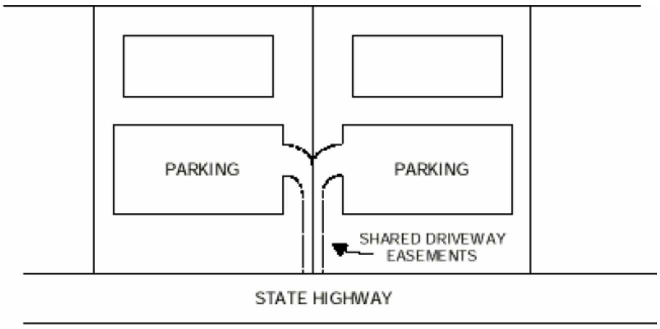


Figure 4: Shared Driveways (TRB, 2003)

4. ACCESS ROADS IN BANGLADESH

There are 126 access roads within 32 km of roadway segment, i.e. per km around 4 accesses in the selected national Dhak-Mawa highway. Indeed, there is no definite guideline for access/local roads in Bangladesh. All the roads that had been built previously were based on the experience and performance of the roads. The typical road network of a region may be either star, grid etc depending on the land use pattern of the area. But in Bangladesh this is not followed at all. Rather it has been found that all the local roads are connected haphazardly to the national highways in Bangladesh resulting in the root cause behind non-functional, unsafe and inefficient transport system.

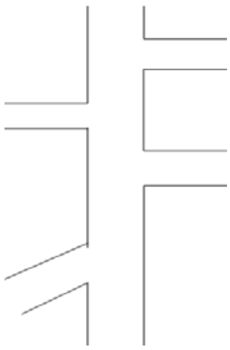
As a result the random position of roads in the highways has proved to be a dangerous cause for the safety of people in both the highways and in the local roads. They made the roads for their convenience of transportation and mobility. Even there are no sign/markings indicating the position of local roads in highways every time. Other major drawbacks as follows:

4.1 No Function Road Hierarchy

The usual road network type of any highway is mainly compatible to the hierarchy of movement. Such as arterials connect to collectors, collectors connect to local roads. But Bangladesh all the road network is haphazard. Every local road directly connects to the main highway. Thus creating a short-circuiting in highway system.

4.2 No regulation to access control

In Bangladesh there is no hard and fast rule to access control. Anyone living by the highway side makes his own road connecting to highway. This frequency of local roads creates the problem in congestion and collision in highways. There is no rule of the number of local roads per unit length of highways.


Fiure5: The Haphazard Road network

4.3 Distance between Access Locations

Criteria for location and frequency of access spacing (i.e. field entrances, intersecting roadways, driveways, etc.) are based on the application of Access Management. Access management also includes techniques to be used to evaluate access issues presented in access applications or during project design. The techniques are listed with an indication whether the technique should be used for evaluating new access or in retrofit situations. There is no such rule in Bangladesh for permissible number, location and dimension of access roads in the highways. So the selection of access roads' location and design is completely haphazard and based on the demand of local users.

4.4 No Standard Design Guideline

There is no such guideline for the construction of local roads and their connection to highways. Even all the local roads connect to the highway without maintaining proper clearance and curve. All the roads are made from the level of experience and practice.

4.5 Frequent Traffic Mix

Traffic mixing is one of the common phenomena that one can find in the highways of Bangladesh. The NMV's from local roads easily mix with the speedy vehicles in highways and travel side by side in the same lane. They do not use the shoulder or any segregated lane. This mixing of traffic results in the occurrence of accidents.

Picture 1: A typical earthen access of Dhaka-Mawa road

5. IMPACT OF LOCAL ACCESS ROADS TO THE HIGHWAY

Impact analysis assesses the changes that can be attributed to a particular intervention, such as a project, program or policy, both the intended ones, as well as ideally the unintended ones. In contrast to outcome monitoring, which examines whether targets have been achieved, impact evaluation is structured to answer the question: how would outcomes such as participants' well-being have changed if the intervention had not been undertaken? This involves counterfactual analysis, that is, "a comparison between what actually happened and what would have happened in the absence of the intervention." Impact analysis seeks to answer cause-and-effect questions. In other words, they look for the changes in outcome that are directly attributable to a program.

Impact analysis helps us to answer key questions for evidence-based policy making: what works, what doesn't, where, why and for how much? It has received increasing attention in policy making in recent years in both Western and developing country contexts. It is an important component of the armory of evaluation tools and approaches and integral to global efforts to improve the effectiveness of aid delivery and public spending more generally in improving living standards. Originally more oriented towards evaluation of social sector programs in developing countries, notably conditional cash transfers, impact evaluation is now being increasingly applied in other areas such as the agriculture, energy and transport.

5.1 Methods and Techniques

On-site field observation, in-depth study on operational and traffic behavior, systematic investigation of road and road side land use pattern, survey of road and roadway elements in the pre-determined format including geometric features like sign, marking, shoulder, road side facilities, surface condition, NMV and pedestrian facilities, access point and curve treatment, sight distance, clear zone etc. and questionnaire survey of local users comprising experience of accident, date of any treatment, traffic behavior etc. accident data analysis has been made to find out the impact of the local access in the study. For impact analysis, road location has been selected using identification of Hazardous Road Location (HRL) technique. At the very outset, police reported ARI accident database has been analyzed and draw the accident location diagram (Figure 6). Most accident prone location/HRL has been ranked from this diagram. To find out the impact of access road, those locations have been selected through preliminary field observation which has at least one access road among the indentified HRL. It is surprising that, almost all of the identified locations found access road. Most of the locations, there are two access roads in both sides of road. In some case that are three or four in number. Finally, four most hazardous locations has been studied using above described techniques and impact of access road has been pointed out separately for different sections.

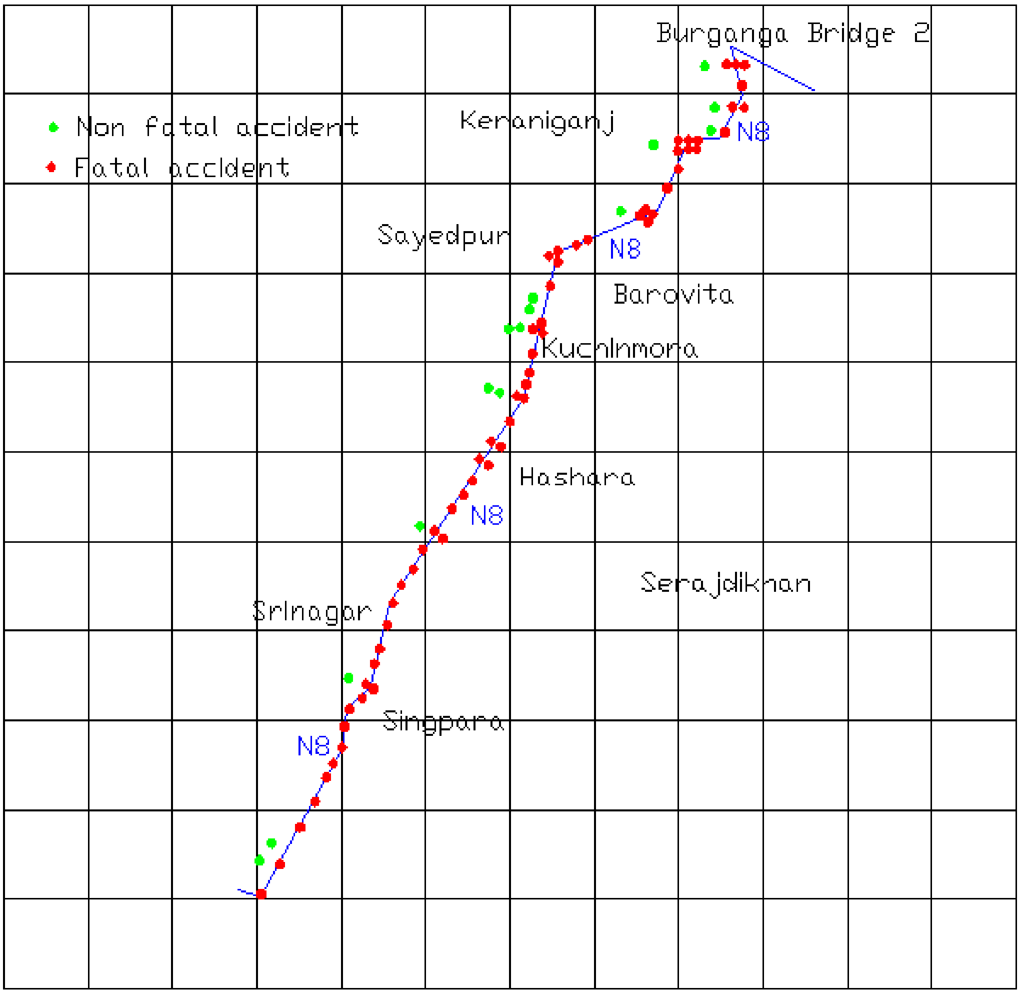


Figure 6: Accidents in various locations of Dhaka-Mawa section of route N8 (ARI database, 2007-09)

5.2 Impact of Direct Access Road

5.2.1 Haphazard Linear Settlement along the Road Side

Unrestrained linear settlement along the side of the road is one of the major operational hazards of the highway in all over the world. From the observation, it is found that most of the liner settlement has taken place in the mouth of the access road on the side of the main road and/or along the side of the access around the connecting point. These linear settlements induce/attract huge traffic, create gathering along the road side sometimes which protruded on the main carriageway, interrupt the mainstream flow.

5.2.2 Conflict of local and through traffic

Conflict between local and through traffic is one of the major and life-threatening problems of untreated frequent local access road of the highway. From the field observation it is noticed that the local traffic frequently mixes with the main traffic from the access roads without any control and conflict between them is a frequent and random nature (Field Survey, 2010). Even the local formal or informal traffic did not use any segregated lane rather they try to compete with the main through traffic. Meanwhile they become the victim of traffic accident in the segment. The speed of through traffic always remains higher to ignore other slow moving vehicles in the highway. Further study is needed to find out the severity level of conflict in a particular point of access.

5.2.3 Damages of road geometry

From the field observation, it is found that damaging of soft shoulder and hard shoulder of the main carriageway is a common picture in the access point due to the concentration of main stream traffic load and local activity at a particular point without any additional treatment or facility. Structural rigidity and aggregate bonding also loses results reducing the life time of the main structure.

5.2.4 Loading and unloading of materials

There are few designated bus bays in the observed highway segment. Unfortunately, all most of the bus bays are not used by the through as well as local traffic. Due to the concentration of local traffic at the access point, passenger loading and unloading activities are occurring at the location.

In addition, in most of the cases, the connecting area is converted as a local market. Most of the shops used the roadside area for the loading and unloading of their materials resulting in the squeezing of carriageway and interrupting the traffic flow.

5.2.5 Sight obstruction

As there has not any regulations and enforcement, road and road side development has occurring without following any set back rules. Most of the temporary shops and stalls are built just besides the road at the mouth of connecting point which obstructs the visibility when traffic from access roads merges into the main traffic stream. That's why, all most all of the access point turned as a blind spot due to the huge vision obstruction at the local road as well as main road. Besides, the parked vehicles and pedestrian activity in the intersection put the sight distance requirement into a danger. Poor sight distances and visibility, unmarked and inappropriate design of intersecting point, serious delineation deficiencies along the route are poses serious safety threat in many sections of the road.

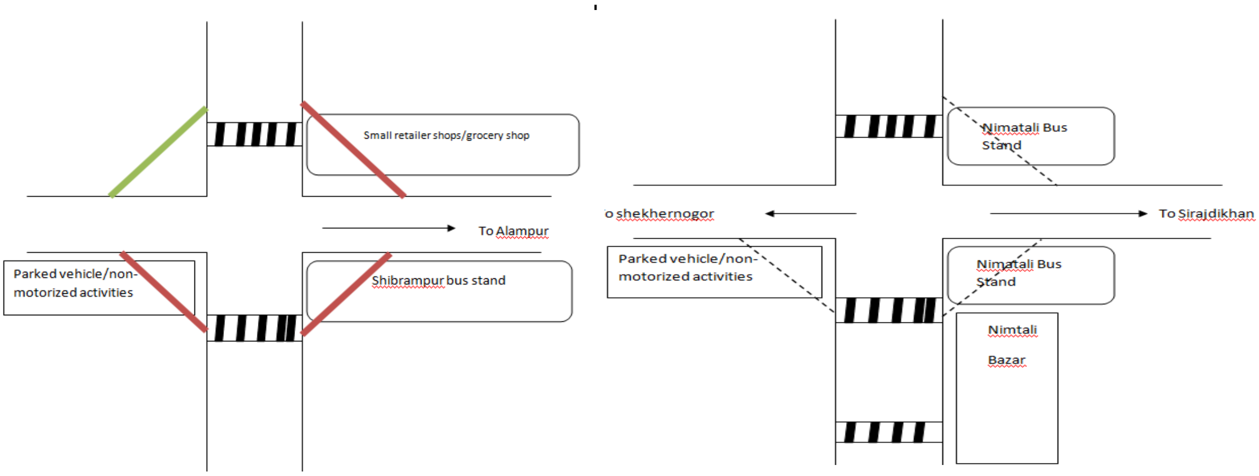


Figure 7: Vision obstruction of two investigated sites Hashra and Nimtoli area (red & dot color line) due to unrestrained settlement on side of access. (not in scale)

5.2.6 Roadside hazard

Road and road side hazardous are one of the major cause of conflict and operational disorder in the transport system in Bangladesh. From the investigation it is proven that all of the hazards are the direct or indirect effect of the access or driveway road. Road side local activities including establishment of retail shop, loading unloading of goods and passengers, parking activity are the major elements of road side hazards. All of those elements induced for the presence of access road. Overall situation is worsen by illegally use of the roadsides or encroachment of the road sides for trading, shopping, marketing, piling of wooden logs and other goods, parking and loading /unloading activities which force the pedestrians use the high-speed road carriage way.

Picture 2: Glimpses of road and road environmental of access road

5.2.7 Reduction of main carriageway

The junction area always remains busy with parked and non-parked vehicles. Also the pedestrian activity is very high. The concentration of non-traffic activities in this area apparently reduces the effective carriageway of highway.

5.2.8 Huge pedestrian activity

There is huge pedestrian activity all the time in the junction of local road and highway and their flow is varied 500 to 1000 ped./hour in different observed segments (Field Survey, 2011). The flow becomes unprecedentedly higher at school period (where have road side school) and weekly bazzer day. The frequent movement of pedestrian hinders the movement of through traffic movement. Random crossing and walking of pedestrian has increased the potentially of accident alarmingly.

5.2.9 Huge Speed Reducing Devices

As described earlier, the sole purpose of highway is mobility for long distance travel. However, there should not have any physical devices which interrupt the free flow speed and loses mobility dramatically like speed bump. Unfortunately, it is found that there are all together 44 speed breaker on the 32 road way segment i.e. more than one speed breaker per kilometer of road. It is surprising that all the speed reducing devices are superimposed for the pressure local people to control accident and casualties. None of the devices are properly designed and now act as a zero speed bump. It is also surprising that all of that informal speed hump are installed just side of the access road i.e. to ensure safe movement of the local traffic which is induced by the access road. Indeed, access road invite this speed hump on the highway and interrupt the though traffic flow devastatingly.

5.2.10 Traffic Delay

One of the major outcomes of access to highways is delays in movement. The movement of through traffic is hampered by the traffic from local roads. As there are no adequate sign/markings to aware the driver of highway about local road ahead, the driver cannot judge the position of local road. When in a rural road junction, the local roads carry the non-motorized vehicle and the slow moving vehicles. These vehicles do not cope up with the high speedy vehicle in the highways. Table 1 represents the speed difference at the free segment and access connecting segment which has been measured from the field survey directly.

Table1: Speed at free flow conditional segment and access intersection								
Speed of Vehicles at Access Road Intersections					Speed at free flow road			
Speed(km/h)	CNG	Bus	Car/jeep	Motorcycle	CNG	Bus	Car/jeep	Motorcycle
Avg	9	14	12.5	13	32.5	49	5	46
Max	12	24	24	24	36	72	72	72
Min	6	9	9	9	24	36	36	36

(Source: Field Survey, 2011)

The speedy vehicle has to reduce their speed up to 60% in the intersections, which elongates the travel time about 65%.

5.2.11 Displacement of residents

According to the local knowledge the newly built kacha road in the junction was due to facilitate the connection of a factory to the major road. So many inhabitants have moved to that area for their business activity.

5.2.12 Traffic Congestion

Hazardous associated with the road and road sides activities due to the induce traffic by the local access and super imposed speed reducing devices some times interrupt the traffic flow so heavily that traffic congestion taking place in some of the busiest segments of the highway corridor.

5.2.13 Damaging of Sign and Making

From the site observation, it is also found that most of the infrequent signpost and guided delineated devices are stolen or damaged by the local users. Its can also relate to the adverse impact of direct access road.

5.2.14 Immense Accidents and Injuries

On-scene in-depth study of accident locations revealed that factors relating to road environment are significant in road accidents in Bangladesh and the road features are indeed associated with particular accident types and hazards (Hoque et al., 2006). Local access are aggravating the road design features and directly and indirectly influencing the occurrence of phenomenon of accident. Every year around 20 accidents are recorded by the police in this segment though the actual number is much more than that which is manifested from the on-site observation and local people observation. Road environmental and operational conditions as well as traffic safety situation are worsening mainly due to adverse impact of huge numbers of direct uncontrolled local access road with the highway.

Hit-pedestrian is the most frequent type of accident which is nearly 48 percent of total accidents and 46 percent of all fatal accidents. This accident type is followed by head on (16%), overturning accidents (14%) and rear end (9%), These four accident type groups accounted for about 87 percent of all accidents and 86 percent of all fatal accidents. Of the vehicles involved in all fatal accidents, nearly two-thirds are buses and trucks. These groups of vehicles are particularly over involved in pedestrian fatalities. Involvement of car and other light vehicles in fatal accidents is 13 percent, followed by motorcycle and auto rickshaws, 7.5 percent. The shares of buses and trucks in registered vehicles are around 13 percent. Figure 8 & 9 represents the type accidents by collision type and accidents in different km post respectively (MAAP, 2006-2009). To arrest this loses resulting from the road traffic accidents and injuries, authority became bound to impose speed bump frequently on the national highway for the demand of local people. All are adverse impact of unguided frequent direct connectivity of local access road.

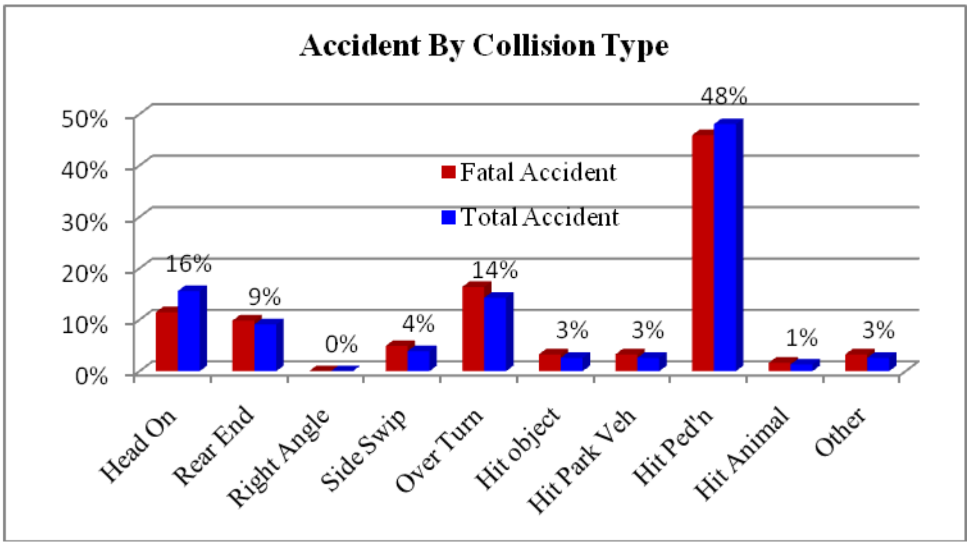


Figure 8: Accident by Collision Type (Source: MAAP5, 2006-2009)

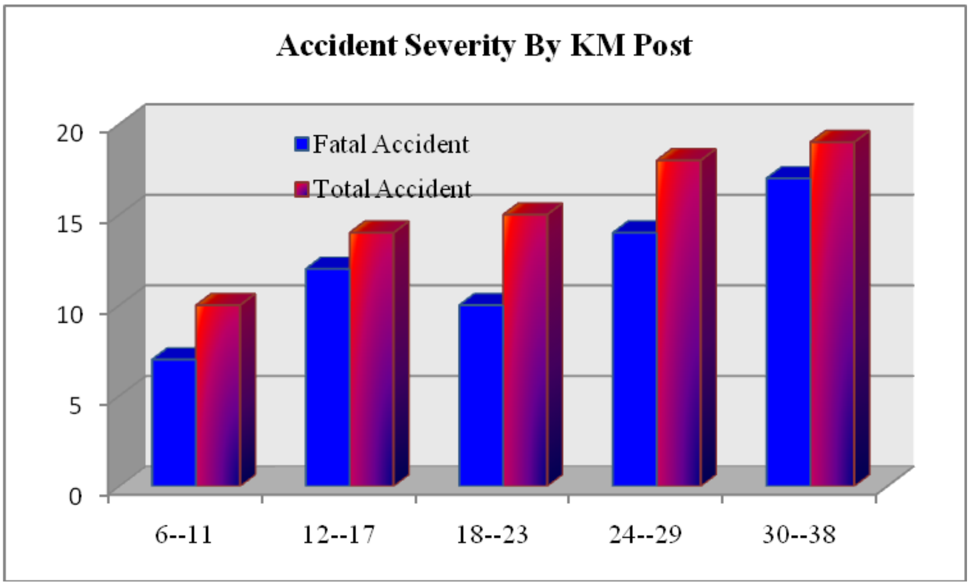


Figure 9: Accident by KM post (Source: MAAP5, 2006-2009)

6. CONCLUSIONS AND RECOMMENDATIONS

Controlling of access is one of the major touchstones of a functional national highway. This study again established this proven fact for the Dhaka-Mawa highway. Road environmental and operational conditions as well as traffic operational, management and safety situation are worsening mainly due to adverse impact of huge numbers of direct uncontrolled local access road with the highway. However, there should have specific and precise guideline, regulation and control on local access road.

The study finally recommended some effective and proven measures for the safety and operational improvement of the highways which are categorized by two types i.e. short term and long term are listed below:

6.1 Short Term Recommendations

Short term steps are to take immediate action to improve the present condition. The immediate steps are as follows:

a) Construction of protective walking and crossing facilities like footpath, zebra crossing as well as lighting facilities.

c) Prohibition of bus stopping near intersection and bridge approaches.

d) Removing of visual obstruction like bush, tree etc in particularly at curve section

e) Installation of median barrier in some busy areas.
f) Introduction of highway surveillance team involving community leaders to control conflicting use of roadway space viz: Temporary bazaar, Drying of crops on shoulder, Parking on the shoulders ,Contra-flow, Keeping domestic animals near the highway for grazing etc.
g) Replacing old signs/markings by retro-reflective signs and markings. Most importantly signs should be writing in Bengali.
h) Discrete type of Chevron marking at the bends should be replaced with continuous retro-reflective type Chevron marking.
i) Discrete type guard-post safety device should be replaced by W-beam type continuous guard rail deflecting barriers.
j) In order to minimize tilting and sudden impact problems, vertical drops between pavement and shoulder as well as between pavement and bridge deck should be reduced.
k) Separate bus bay with pedestrian barrier and if possible construction of passenger shed.
l) Movements of NMV and non-standard type vehicles should be controlled and if possible restricted.

6.2 Long term Recommendations

Long term steps are intended to meet the future demand of the society. These type of steps are taken for the welfare of the project and the efficiency of the project work. Some of the long term steps that could help improve the highway condition are stated as follows:

- Provision of frontage or service roads along the side of the national road at least along the built-up area
- Regulation and implementation of access control

6.3 Recommendation for Future Research

Though in this research work it is attempted to make the study comprehensive but due to time and economic constraints, few important aspects could not be addressed. If the following issues could have been considered in this thesis work, it would have been more comprehensive and complete.

- A detail impact analysis may be conducted at all the access points identified in this study.
- Monitoring the access points for at least three years and conducting before after analysis to evaluate performance of implemented access management measures more conclusively.
- An impact study of road adjacent existing uncontrolled land use development on traffic safety may be conducted.

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A GIS BASED APPROACH TO COMBINE THE WATER WAY AND ROAD NETWORK FOR THE SOLID WASTE MANAGEMENT OF DHAKA CITY

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ABSTRACT

This paper presents an overview of the current solid waste transfer system in Dhaka City and provides an effective solution to overcome the present and future challenges in transporting waste. Optimal route including road network and water ways integrated with Geographic Information System (GIS) is critically assessed to determine the minimum distance efficient collection paths for transporting the solid wastes to the existing landfill sites. Introduction of waste transfer station near the river side and use of surrounding river route makes the proposed transport system more effective for waste hauling of Dhaka City Corporation (DCC). Waste dumping at the transfer stations decreases the haul time of the collection trucks 0.1h, 0.7h, 0.8h, 1 h per trip respectively for different DCC zones. From hydrographic survey data and field survey it is evident that the navigation facility of the river Buriganga is sufficient for waste transport using available water barge. It is also evident from this research that the use of surrounding river route in transporting waste by DCC will make the waste management system economic.

1. INTRODUCTION

In present day's problems regarding with the waste management have become one of the top concerning issues, basically in developing country like Bangladesh because of its rapid and imbalanced urbanization and industrialization. Dhaka (latitude 23.77N, longitude 90.38E) is the capital and the most populous city in Bangladesh. At the present time, it generates about 3500 – 4000 (Rahman and Rahman, 2011) tons of solid waste per day, the per capita generation being 0.5 kg/day. DCC Ordinance1 is the basic law regarding street/drain cleaning, waste collection and transportation. According to Section 78 of the Ordinance, DCC is responsible for secondary waste collection to remove waste from its dustbins/containers, and transport the waste to final disposal sites. Residents are responsible for bringing their waste to DCC's waste collection points where dustbins/containers are located. Because of limited space, the siting and construction of new solid waste management (SWM) facility is a big challenge in Dhaka. The major sources of municipal solid wastes in Dhaka are domestic, streets, market places, commercial establishments, clinics and hospitals. The generation of solid waste would be around 8,478 tons/day by the year 2020 (BCAS, 1998; Hai and Ali 2005) which will be a major challenge for DCC to manage this huge amount with its existing system. The whole city corporation area comprises 90 wards which are divided into ten zones and DCC maintains zone wise solid waste collection and transport with its limited number of trucks. In 2002, DCC introduced an approval system of NGOs/CBOs/private organization for providing door-to-door waste collection services in all wards. DCC has given approvals to 47 NGOs/CBOs, however, not all of them have started their activities yet. Conservancy department of DCC is responsible for transporting the solid waste from the City Corporation dustbin/container to the final disposal site. There are 343 units of vehicles of DCC allocated for collection and transportation services (JICA, 2005).The vehicles are placed at three garages: Saidabad, Zone 7 and Zone 8 offices. In addition to DCC vehicles, the four service providers operate 19 to 27 units of 5-ton open trucks in eight wards. Environmental Grant Aid program (EGAP) provided 100 collection vehicles to Waste Management Department of Dhaka City Corporation. Out of these vehicles, DCC is using 29 compactors and 16 arm roll vehicles quite effectively. On 12 January 2011, one 3 ton capacity container carrier ran by CNG started its operation (Clean Dhaka Newsletter, February, 2011).

Table 1: Ward wise new collection vehicle

Type	Ward Numbers
Arm Rolls	7, 19, 23, 33, 34, 35, 36, 39, 52, 70, 71, 78
Compactors	2, 5, 12, 19, 22, 33, 36, 38, 41, 43, 44, 45, 49, 50, 52, 53, 57, 63, 76

Total number of waste storage facilities is 1,098 units, which include 341 units of dustbins, 346 units of containers, 402 Temporary collection Points, 7 Trailor Container and 2 units of 12 m^3 containers as summarized in Table2. The storage capacity of DCC's dustbins in Zone 9 and Zone 10 is small since the private service providers are engaged in collection and transportation services in the two Zones.

Table 2: Zone-wise Existing Dustbins and Waste Containers of DCC

Zone	Dustbin	Container	Temporary collection Points (TCP)	Trailor Container	Illegal dumping	Total
Zone1	51	29	91	0	0	171
Zone2	91	62	26	0	0	179
Zone3	73	20	45	0	0	138
Zone4	29	77	57	1	0	164
Zone5	41	63	51	0	0	155
Zone6	40	33	62	2	0	137
Zone7	10	16	22	1	0	49
Zone8	3	46	46	3	0	98
Zone9	3	0	2	0	2	7
Total	341	346	402	7	2	1098

DCC has constructed Matuail extension and Amin Bazar Baliapur as sanitary landfill site. It dumps the waste in above mentioned landfill sites using the road network only. According to the clean Dhaka master plan yearly target collection rate and the waste amount are proposed as shown in Table.

Table 3: Targets of Waste Disposal in Master Plan of DCC

	Target for 2015 (t/d)
Collection/ transport	3,054
Final disposal	3,032
Recycling	672
Unidentified disposal	920

The management issue of generated solid waste is not only multifaceted with its increasing quantities but also with its inadequate management system (Tinmaz & Demir, 2005). In this study, Geographic Information System (GIS), a good decision support tool for waste management planning were used to define the possible option for efficient solid waste management. According to Ogra (2003), the more the layers in terms of information, the more will be better decision analysis. Urban solid waste management practices require collection of decisive information which is for taking corrective measures as well as for proper planning to ensure sustainability (Ramachandra & Saira, 2003). Because of huge traffic pressure and long haul distance of disposal site from some zones road network based waste transfer system is not sufficient for efficient management of waste. To collect and transport the solid waste effectively according to the target new collection and transport route have to design. In Shanghai 6,000 tones/day waste was carried by canal barge per day at 2004 (HKWMA, 2004). So there is a possibility to use river for waste transport in Dhaka City since it almost encircled by the river Burigonga and Turag.

2. METHODOLOGY

For The methodology includes the collection of information about the waste management situations in Dhaka city and preparing a database about the waste situations of the case study area. This study includes a discussion and a critical analysis of the new and old route. On the basis of the present situation analysis, the data availability of the case study area and the study analysis, the framing of guidelines for the work to be proposed in dealing with waste management planning for the DCC was carried out. By this method some guidelines for the route preparation was organized which would be the baseline for the proposed routes and transfer stations. Finally the GIS route was implemented on Dhaka city's case study area data for the analysis and the results will suggest some modification in the existing system which is expected to reduce the waste management workload to a certain extent. The over all methodology is described in the flow diagram in Fig1:

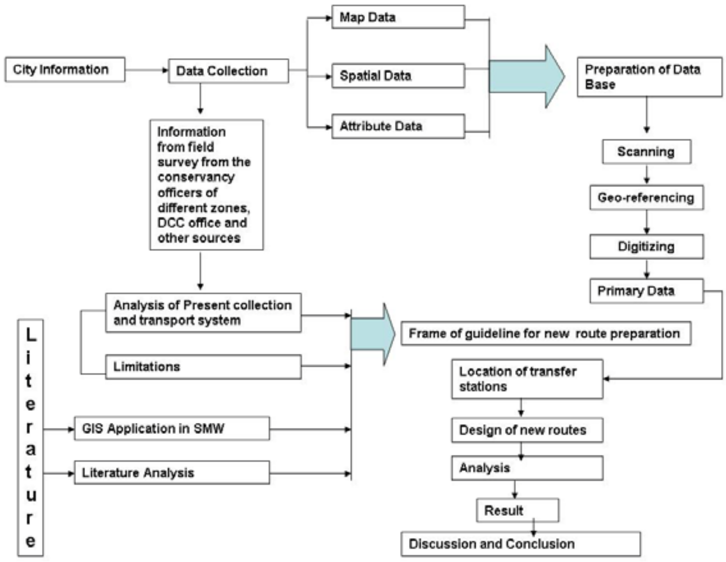


Fig1: Flow diagram of Methodology

This study is based on secondary collected data DCC, BIWTA and Water development board office and performed filed survey with the conservancy officers of different zones to collect information on the present situation. Collected maps from DCC and BIWTA office were digitized using Arc GIS 9.1 to build GIS database. Other collected data was also incorporated in the data base file. Map of different zones collected from DCC was in JPEG format image which was imported in Arc GIS and georeferenced.

3. FEASIBILITY FOR THE USE OF EXITING WATER WAY ROUTE IN RIVER BURIGANGA:

Dhaka city is almost encircled by the river Buriganga and Turag. At present Bangladesh Inland Water Transport Authority (BIWTA) maintains a water way route around Dhaka. In recent time BIWTA is running a water bus for public transport from Gabtoli to Shadarghat. BIWTA has 4 landing station site officially on Burigangaa and Turag River. They are Wise Ghat landing station, Swarighat landing station, Kholamura landing station and the last Gabtali landing station. Those sites are usually use for public transportation from Gabtali to Shadarghat. Gabtali site is also loading unloading place of construction materials such as river sand, bricks, sylhet sand and other materials. There are around 200 local agency offices who import sand, bricks, sylhet sand and other materials. They use the Meghna – Buriganga water ways route to transport their materials. There is some cement company such as chatak who also uses this route to transport their product. Usually big and small water burges are used to transport in this route. A locally made barge is usually 50 feet long and 23 feet in width. The goods containing compartment is 7.5 feet in depth and can carry 4275 cubic feet of sand per trip. Total distance from Shadargagat to Gabtoli is 16 km and a barge take 30 min to travel the distance. Minimum water depth from sadarghat to kamrangir char and amin bazaar to ramchandrapur is 8 before dragging (HSD, 2011). So it's possible to transport solid waste using water barges with small modification.

Table 4: Capacity comparison of barge and truck

Transport vehicle	Water Barge	Container Carrier	Open truck
Capacity	121 tons (one trip)	9 tons (3 trips)	4.5 tons (1.5 trips)

4. PROPOSED NEW TRANSPORTATION SYSTEM FOR DHAKA CITY:

DCC uses road network for the transport of waste to disposal site. Conservancy department of Dhaka City Corporation has no scientific and engineering principle based on solid waste management planning approach. Conservancy department has no solid waste management specialist (Engineering background). Present distribution of labor, transport in different areas has no database demand and conservancy department does not maintain any data base for generation of waste, transport and labor requirement. There is no cleanliness standard. Town planning department is yet to allocate any space for storage of waste bin, placing of container. Presently transport fleet of City Corporation is collecting 43% waste by open truck (JICA, 2005). Open truck collection system needs average 3-5 hr for loading and unloading the waste to and from the trucks. So disposal by open truck yield very low efficiency. Although DCC has introduced Arm Rolls and Compactor trucks into its transport fleet, these are only covering 31 out of 90 wards. To meet the present and future demand DCC need to increase its collection vehicles.

The proposed system includes the use of both water ways and road network at same time. Some transfer station near the river side need to construct considering the coverage of different zones. The collection trucks will collect waste by following the existing route inside the zone and after collection of waste it will go the nearest transfer station depending upon the zones. A water barge will collect the waste from the transfer stations and dispose to the landing station near the landfill site. Finally a covered van will collect the waste from the landing station and dispose to landfill site.

4.1 Route Preparation Using GIS:

In this research Network Analyst tool was used to prepare the route for new waste transfer system. Using ArcCatalog network datasets were prepared from the shape file of Dhaka city road networks. The best route was determined for the order of locations as specified in this case using Network Analyst tool.

5. RESULTS:

At present DCC truck drivers doesn't follow any particular route when they transport the waste from different zones to the land fill/ disposal sites. For a particular zone there some particular collection routes through which the collection vehicles travels to collect waste from the dustbins, temporary collection points, containers but after collection waste there is no predefined route which the collection truck travels to disposal site. Field survey was performed with the truck drivers to know the exact situation. According to the survey they use different route in different times depending upon the jam and road situation. So it is very difficult to determine the haul distance in existing waste transport system. Because of this condition in this case the shortest haul distance from different zone boundary to the Matuail land fill site was determine using the developed network analysis database in ArcMap. Field survey was also done for the water ways route and the required transfer station to develop the new waste transfer system using the river Buriganga. After numerous surveys some location was identified to build the transfer stations. Considering the land availability transfer stations near river side of zone 6, 1 and near aminbazzar landing station was identified using GPS to design new route. Since zone1 and zone2 is close to the matuail land fill site, only the waste from the zone3, 5, 6, 7 and 8 has been considered to bring at the transfer station1(Fig) . The haul distance from the considered zone boundary to the transfer station1 was determined using the developed network data set. The shortest route in existing transport system and in new system is shown in fig. From the analysis it is clear the haul distance for zone3, zone6, zone7 and zone8 decreases significant amount in proposed system. As haul time mostly depends upon the haul distance, the haul time of collection vehicle will also reduce significant amount. The exact amount of haul time reduction was not possible due to lack of haul constants since DCC truck drivers doesn't follow particular route. An approximate haul constant was determined using the field survey data with the truck divers.

Table 5: Haul distance and haul time of different zones

DCC Zone	Average Round Trip Distance (km) according to DCC transport department	Average haul time according to the driver survey
Zone1	14	0.66
Zone7	42	3
Zone8	44	3.5
Zone 10	60	2.5

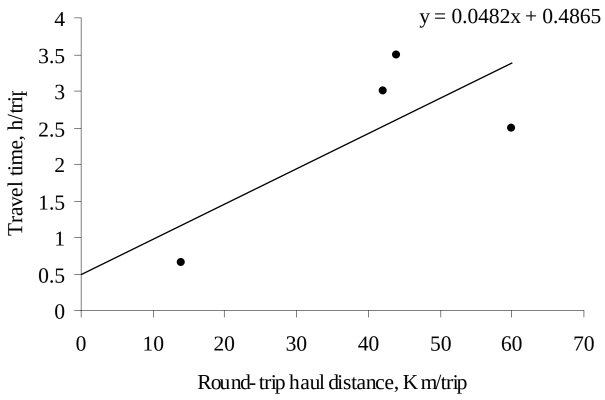


Fig2: Travel time vs Round-trip distance

Table 6: Haul time determination

Zone	Haul distance in Existing system (km)	Haul distance in Transfer station system(km)	% Reduction of haul distance	Haul Constant, b	Haul Constant, a	Haul time Reduction (hour) in new system
Zone 3	12.2	10.5	13.8	0.0482	0.4865	0.10
Zone 5	10.0	8.0	20.0	0.0482	0.4865	0.58
Zone 6	18.8	4.3	77.0	0.0482	0.4865	0.70
Zone 7	25	9.3	62.7	0.0482	0.4865	0.80
Zone 8	31.8	11.9	62.5	0.0482	0.4865	1.00

From route analysis it is clear that the haul time for the collection vehicles in road network is reducing which will increase the waste collection efficiency for existing number of vehicles. The vehicles may finish more trip than the present number in new proposed system. The route map is shown in Fig3.

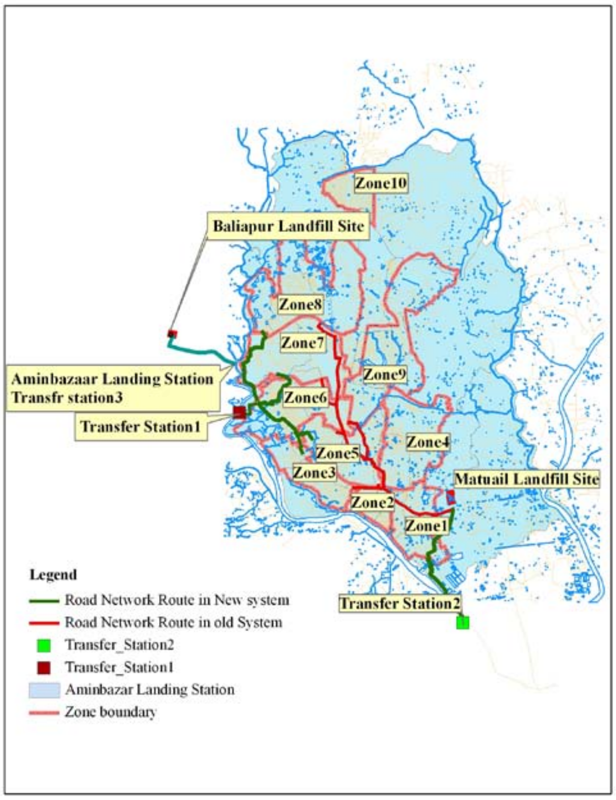


Fig3: Map of the prosposed and old route

5.1 Water Ways Route Preparation:

The river map collected from the Hydrographic division of BIWTA office was ammonia printed form in A3 paper. It has the sounding depth as well as the actual river width during the hydrographic survey. There was five A3 sheet covering the route from Shadorghat to Amin bazzar. The maps were scanned in JPEG format. The raster image was importuned in ArcMap and georeferenced. Finally water ways route was prepared from the georeferenced raster data set by batch vectorization method.

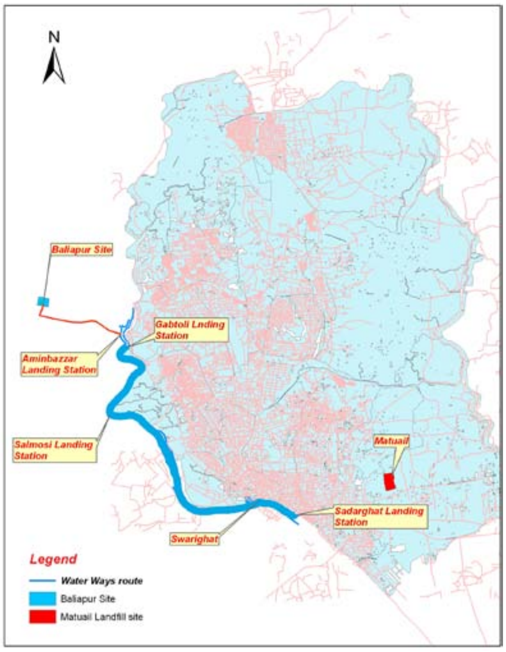


Fig4: Water ways route for solid waste transfer

From the route query the following route summary presented in Table7

Table 7: Water ways route Summary

Route Name	**Route Length (Km)**
Gabtoli to Salmosi	5
Salmosi to Swarighat	9
Swarighat to Sadaghat	2
Total length	16

Table 8: Road Network route length for Baliapur Site

Route	**Length(Km)**
Amin Bazar Landing station to Baliapur site	4.4

Since DCC uses both Matuail and Balipur landfill site there is two options in the new system. The existing collection vehicles of zone 3, 5, 6, 7, 8 will collect waste in the existing collection route and will dispose the waste to the transfer station1. The water barge will collect the waste from transfer station1 and unload to the transfer station near amin bazzar landing station. Finally a covered van will collect the waste and dispose in the Baliapur Landfill site. The second option is the water barge will collect the waste from transfer station1 and then unloads at transfer station2. A covered van will collect the waste form the transfer station2 and transport to the disposal site in Matuail. Since Matuail landfill site can take care of 59% of the total collected waste (1700 ton/day) of Dhaka City Corporation (DCC) per day, this amount will be transferred by the water barge from transfer station1 to 2. The remaining waste will be transferred by the water barge form transfer station1 to 3.

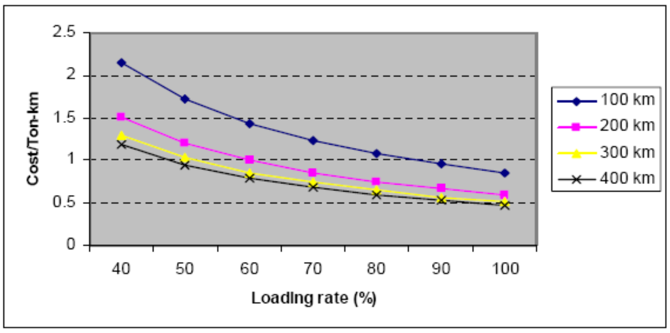
5.2 Economic Analysis of Water ways Transport system:

In Bangladesh Inland Water Transport (IWT) is a major mode for the transport of goods and people. IWT is important for the poor as well as for the competitiveness and growth of the economy as it is the cheapest mode of transport compared to road or rail (World Bank, 2007).

Table 9: Comparison of cargo tariffs by transport mode

		Dhaka-Chittagong	**Dhaka-Sylhet**
ROAD	Distance	264 km	346 km
	Tariff per 5-ton truck	Tk 6,000	Tk 7,500
	Tariff/ton-km	Tk 4.50	Tk 4.34
RAIL	Distance	340 km	230 km
	Tariff per ton	Tk 930	Tk 870
	Tariff/ton-km	Tk 2.74	Tk 3.78
IWT	Distance	304 km	410 km
	Tariff per ton	Tk 300	Tk 260
	Tariff/ton-km	Tk 0.99	Tk 0.63

IWT costs are below the road and rail tariffs above, confirming that IWT is the cheapest mode of transport. The following graph compares costs per ton-km based on various assumptions of loading rate and distance of journey. IWT is profitable for a distance of 300 km with a 60 percent loading rate or higher and for a distance of 400 km with an 80 percent loading rate or higher (World Bank, 2007)



So there is no doubt that the use of water ways is the cheapest option for any type of goods transport. The following table describes the Comparison of Cost-Performance among Container Carrier, Open Truck and Water barge.

Table: Economic Analysis for the use of Water Barge

Cost item		Container Carrier (3 ton)	Open Truck (3 ton)	Water barge
Initial Cost	vehicle*	3464500	2675400	20,000,000
	container	58,000	none	none
	service life	10 years	15 years	10 years
	no. of containers	3 per truck		
	allocated			
	container life	3 years		
	cost per year	352250	178360	2000000
running cost	fuel cost	42 liters @Tk 51 x 365 days:	21 liters @Tk 51 x 365 days:	28 liters@ Tk 51x 365
		781830	390915	521220
	driver	1 person @Tk 10,000 x 12	1 person @Tk 10,000 x 12	1 person @ 10000x12 month
		120000	120000	120000
	cleaner	1 person @Tk 6,000 x 12 =	3 person @Tk 6,000 x 12 =	30 person @9000x12=
		72000	72000	3240000
total cost	per year	1326080	761275	5881220
Performance item				
collection volume	daily	3 ton x 3 trips x 80%: 7.2 t/d	3 ton x 1.5 trips x80%: 3.6 t/d	300ton x 1 trip x 80% : 240 t/d
	annual	7.2 t/d x 365 day: 2,628 t	3.6 t/d x 365 days: 1,314 t	96.8t/dx365 days: 35332 t
unit cost		Tk 505/t	Tk 579/t	Tk 67/t

6. CONCLUSION:

The water ways route presented in this study is efficient for waste transfer using water barges in the available navigation facility. Incorporation of the transfer stations increases waste collection efficiency of existing collection trucks situation. Recycling and sorting of waste material can be possible in the proposed transfer station. More details study is required for the final construction of transfer stations near river side.

The use of Water ways will reduce the pressure on the road traffic system. Since water transportation is comparatively cheap so this will be economic and it will reduce the problem of pollution by the open truck mostly used by DCC. Grab cranes may be used to off-load the incoming waste from barges and fill up site trucks.

ACKNOWLEDGEMENTS

The authors are thankful to ITN-BUET for funding this project

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PRODUCTION OF CITRIC ACID FROM OIL PALM EMPTY FRUIT BUNCHES THROUGH SOLID STATE BIOCONVERSION IN TRAY FERMENTOR

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ABSTRACT

Semi-pilot scale production of citric acid in tray fermentor was optimized considering aeration and substrate thickness as process parameters. Other process parameters like moisture content, incubation temperature and initial pH were adopted from previous study at optimum level. The optimization experiments were carried out for citric acid production by following traditional one-factor-at-a-time (OFAT) method and response surface methodology (RSM). The possible optimum level of aeration and substrate thickness were found from the OFAT study to be 2 L/kg/min and substrate thickness of 3 cm, respectively. The highest production of citric acid of 384.21 g/kg-EFB was obtained by further optimizing the parameters by CCD under RSM at aeration of 2.2 L/kg/min and substrate thickness of 3.2 cm. The analysis of variance (ANOVA) of the statistical optimization showed that the quadratic regression model demonstrated the model was highly significant ($P_{model} > F = 0.0001$). It was observed that square term of both parameters (A^2 and B^2) were the most significant over the other model term as *p*-value <0.0001. Furthermore, the *p*-value of the linear terms, aeration (A) and substrate thickness (B) as well as the interactive term (AB) were lower than 0.05 indicated the significance at 95% confident interval. The lack of fit of the model was found to be insignificant (p>0.05) which is strongly desirable for the adequate model.

Keywords: citric acid; process condition; optimization; solid state bioconversion; tray fermentor and *Aspergillus niger*.

1. INTRODUCTION

Citric acid is an organic acid that has been observed to be a commercially valuable microbial metabolite which is produced mainly through submerged (liquid state) fermentation of starch or sucrose based media, using the filamentous fungus *Aspergillus niger* (Lofty et al., 2007a,b; Barrington and Kim, 2008). Because of its manifest use, citric acid is considered one of the most important fermentation products. Considerable amount of citric acid is required in several industrial processes (Jianlong, 2000). The food and beverage industries consume about 70% of total citric acid produced, while pharmaceutical industries consume about 12%, and the remaining 18% are consumed by other industries (Pandey et al., 2001; Soccol and Vandenberghe, 2003). As the global demand for citric acid is growing faster than its production while the cost of existing raw materials is increasing thus requiring an alternative economical process with new substrates to be an attractive over the submerged fermentation (SmF) process.

Solid-state fermentation (SSF) is an alternative method to submerged fermentation for citric acid production using agro-industrial residues as reported by several researchers (Barrington and Kim, 2008; Imandi et al., 2008; Narayanamurthy et al., 2008). It offers significant advantages in the treatment and value addition of these residues over the submerged fermentation (Brand et al., 2002). This process has lower energy requirements, less wastewater generation, less risk of bacterial contamination, easily manageable, and is environment friendly as it resolves the problem of solid wastes disposal by microbial treatment (Imandi et al., 2008).

A variety of agro-industrial residues and by-products have been investigated using SSF techniques for their potential to be used as substrates for citric acid production. A cost effective production of citric acid has been achieved by using less expensive lignocellulosic substrates, such as sugarcane bagasse (Kumar et al., 2003), wheat bran (kumar et al, 2003), cassava bagasse (Prado et al., 2005a,b), carob pod (Roukas, 1999), coffee husk (Vandenberghe et al., 2000). Due to similar reasons, oil palm empty fruit bunches (EFB), which is abundantly produced (17.08 million tons/year) by oil palm industries in Malaysia (Chew and Bhatia, 2008), would be the potential alternative renewable raw material for citric acid production by solid state bioconversion with *Aspergillus niger*.

The accumulation of citric acid is strongly affected by fermentation conditions such as initial pH, moisture content, incubation temperature, aeration and thickness of substrate (Lofty et al. 2007b). Solid state fermentation is relatively a new technology and tray type fermentor might be used for this purpose. The development of semi-pilot scale production process is essential as an initial stage of large stage production. So far our knowledge goes, there is a lack of information on optimum fermentation conditions for citric acid production through solid state bioconversion in the literature. Hence, the aim of this study is to develop a laboratory scale production process in tray fermentor. In the present study, the production of citric acid was optimized in Erlenmeyer flasks and in tray fermentor by following one-factor-at-a-time (OFAT) method followed by central composite design (CCD) to develop a model which helped to determine the optimum process parameters for citric acid production from EFB through solid state bioconversion.

2. MATERIALS AND METHODS

2.1 Major Substrate

The major substrate oil palm empty fruit bunches (EFB) was collected from Seri Ulu Langat Palm Oil Mill in Dengkil, Selangor, Malaysia and stored in a cold room at 4^0C to avoid the unwanted bio-degradation by any microorganisms. The EFB samples were prepared by grinding to 0.5 mm down-grade particle size after washing vigorously with tap water. The ground EFB was dried at 60^0C for 48 hours to get constant dry weight for the experimental study.

2.2 Preparation of Inoculums

The cultures of local isolate of *Aspergillus niger* IBO-103MNB (IMI396649) were grown on PDA plates at 32^0C for 4 days and washed with 25 ml sterilized distilled water to prepare the inoculum. Spore suspension was collected in 100 ml Erlenmeyer flask by filtering with Whatman No. 1 filter paper. A haemocytometer was used to maintain the spores density of 1×10^8 spores/ml. The fungal strain was maintained on 3.9% w/v of potato dextrose agar (PDA, Mark) slants, sub-cultured once in a month and stored at 4^0C.

2.3 Experimental Procedure for Solid State Bioconversion

Two kilograms substrate (wet basis) was prepared for the experiment in semi-pilot scale tray fermentor with required percentage of major substrate – EFB (particle size≤0.5 mm), 6.4% (w/w) sugar (sucrose), 2% methanol and 9% mineral solution containing 0.09 g/l $ZnSO_4.7H_2O$, 0.1 g/l $CuSO_4.5H_2O$, 0.4 g/l $MnSO_4$ and 5 g/l $MgSO_4.7H_2O$, which was optimized by previous study (Bari et al., 2009). Moisture content was adjusted with mineral solution, inoculum, methanol and distilled water. Methanol and inoculum were added after sterilization of media by autoclaving at 121^0C for 15 minutes.

2.4 Harvesting and Extraction of Citric Acid

Harvesting and extraction of citric acid was carried out after 7 days of bioconversion by taking five samples of 20 g fermented substrate from different points of tray fermentor into 250 ml Erlenmeyer flask. The fermented substrate was thoroughly mixed with 50 ml distilled water and was shaken for 1 hour at 150 rpm at room temperature (28 ± 1^0C) in a rotary shaker (Tran et al. 1998). The supernatant was collected by filtering with Whatman no. 1 filter paper and immediately analyzed to determine the content of citric acid.

2.5 Determination of Citric Acid

The concentration of citric acid in extract was determined by Waters HPLC instrument equipped with a refractive index detector (RID), Shodex RSpak KC 811 column (inner dia. 8×300 mm, Shodex, Japan). The eluent used for this analysis was 0.1% phosphoric acid solution. HPLC analysis was carried out under the following operation conditions: pump flow, 1 ml/min; column temperature, 40^0C; sample volume 5 μl; integration method and peak area. Concentrations were automatically calculated by Breeze software (version 3.3), Waters Corporation, U.S.A. The production of citric acid was expressed as g/kg of dry solid substrate (EFB).

2.6 Experimental Design

The optimum production conditions of moisture content, incubation temperature and initial pH of substrate were adopted from previous study (Bari et al. 2010). The production conditions in semi-pilot scale tray fermentor were optimized by following traditional one-factor-at-a-time (OFAT) method and central composite design (CCD) under response surface methodology (RSM). The aeration and substrate thickness were considered as parameters in tray fermentor production. Substrate thickness was varied from 1 to 5 cm and aeration from 0 to 4 L/kg/min in case of OFAT method.

The central composite design (CCD) under response surface methodology (RSM) was employed in order to illustrate the nature of the response surface in the experimental region and to elucidate the optimal concentrations of the most significant independent variables. Two variables namely, aeration and substrate thickness were included in this model. According to the CCD for two variables, 13 experimental runs with 5 runs at center point were executed and their observations were fitted to the following second order polynomial model:

$$Y = \beta_0 + \beta_1 A + \beta_2 B + \beta_{11} A^2 + \beta_{22} B^2 + \beta_{12} AB \qquad (1)$$

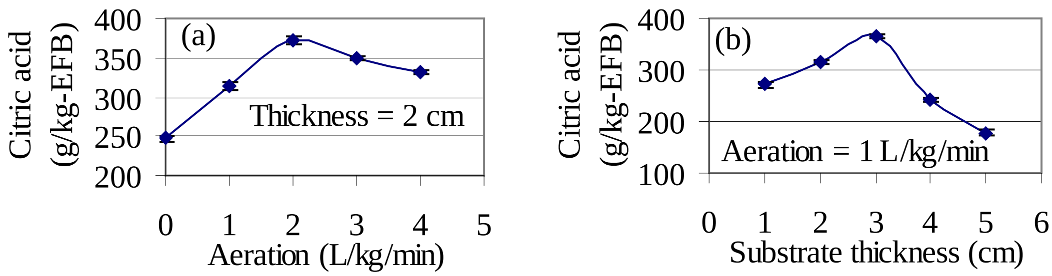
where, Y is the dependent variable (citric acid productivity); A and B are the independent variable (aeration and substrate thickness); β_0 is the regression coefficient at center point; β_1 and β_2 are the linear coefficients; β_{11} and β_{22} are the quadratic coefficients and β_{12} is the second order interaction coefficient.

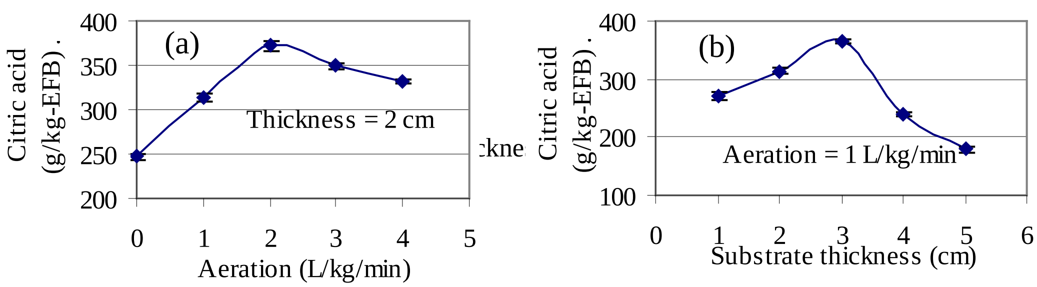
The developed regression model was evaluated by analyzing the values of regression coefficients, ANOVA (analysis of variance), *p*-values and *F*-vales. The quality of fit of the polynomial model equation was expressed by the coefficient of determination, R^2. The statistical software package Design-Expart®6.0.8 was used to generate a regression model to predict the effect of the operating parameters on citric acid production. A final experiment was designed to validate the CCD model prediction.

3. RESULTS AND DISCUSSION

3.1 Optimization of Citric Acid Production in Tray Fermentor by One-Factor-at-a-Time (OFAT) Method

The probable optimum level of aeration and substrate thickness were determined by following OFAT method. The effect of aeration on citric acid production in tray fermentor was examined with the varying supply of saturated air from 0 to 4 L/kg/min. Air was passed through 0.2µm air filter to sterilize before entering into the humidifying chamber. Sterilized distilled water with 2% methanol was poured into the humidifying chamber. The result shows that the highest production of citric acid (372 g/kg-EFB) was observed with 2 L/kg/min air supply while substrate thickness was maintained of 2 cm (Fig. 1a). The lower production of citric acid with lower and higher supply of air might be due to high and low partial pressure of CO_2, respectively, which probably create unfavourable environment for citric acid synthesis (Prado et al., 2005a).





On the other hand, substrate thickness was varied from 1 to 5 cm and observed its effect on citric acid production. Fig. 1b shows the variation of citric acid production with the variation of substrate thickness in tray fermentor. Result shows that the highest citric acid of 364.88 g/kg-EFB was obtained for 3 cm substrate thickness while air supply was maintained at 1 L/kg/min. Low thickness of substrate allowed more air transfer and reduced the entrapment of CO_2 which enhance the fungal growth. However, growth limitation is important for higher accumulation of citric acid which is extremely desirable (Prado et al., 2005a). Furthermore, higher thickness provides more compaction of substrate, which reduces the production of citric acid by hampering the mass and heat transfer. From the above discussion, the probable optimum level of aeration and substrate thickness would be 2 L/kg/min and 3 cm, respectively that need to be further optimized by using CCD under response surface methodology (RSM).

3.2 Optimization of Citric Acid Production in Tray Fermentor by Central Composite Design (CCD)

The citric acid production in tray fermentor was optimized considering aeration and substrate thickness as independent variables from EFB. The initial pH, moisture content and incubation temperature were maintained at optimum level obtained from laboratory scale production optimization (Bari et al. 2010). The design matrix of the coded and actual values of the variables according to CCD together with the experimental and predicted (using Eq. 1) results for citric acid production are presented in Table 1.

Table 1: The experimental and predicted citric acid production in tray fermentor by CCD

Run no.	Aeration	Thickness	Citric acid (g/kg-EFB)	
			Experimental	Predict
1	0 (-1)	1 (-1)	206	208.98
2	4 (+1)	1 (-1)	273	262.98
3	0 (-1)	5 (+1)	258	265.64
4	4 (+1)	5 (+1)	265	259.64
5	0 (-1)	3 (0)	324	313.38
6	4 (+1)	3 (0)	322	337.38
7	2 (0)	1 (-1)	285	292.05
8	2 (0)	5 (+1)	321	318.71
9	2 (0)	3 (0)	391	381.45
10	2 (0)	3 (0)	384	381.45
11	2 (0)	3 (0)	385	381.45
12	2 (0)	3 (0)	383	381.45
13	2 (0)	3 (0)	388	381.45

For predicting the optimal values of citric acid produced within the experimental constrains, a second order polynomial model was fitted to the experimental results for the citric acid yield by the Design Expert software. The model developed is as follows:

$$Y = 99.72 + 73.32A + 128.27B - 14.02A^2 - 19.02B^2 - 3.75AB \qquad (2$$

The regression equation for the optimization of medium constituents showed that citric acid production (Y, g/kg of dry EFB)) is a function of the aeration (A, L/kg/min) and substrate thickness (B, cm).

The value of R^2 was 0.9828 of developed second order quadratic model for the citric acid production. This value indicates a high degree of correlation between the experimental and the predicted values. The value of R^2 indicates that 98.28% of the variables: aeration and substrate thickness were supported by the response. The value of the adjusted coefficient of determination was also very high (97.06%) to indicate a high significance of the model. The value of adequate precision (signal to noise ratio) of 24.75 is very high compared to desirable value (greater than 4) which indicates this model can be used to navigate the design space.

The corresponding analysis of variance (ANOVA) is presented in Table 2. The ANOVA of quadratic regression model demonstrated the model was highly significant, as an evident from the Fisher's *F*-test with a very low probability value ($P_{model} > F = 0.0001$). It was observed that square term of both parameters (A^2 and B^2) were the most significant over the other model term as p-value <0.0001. Furthermore, the p-value of the linear terms, aeration (A) and substrate thickness as well as the interactive (AB) were lower than 0.05 indicated the significance at 95% confident interval. Linear, quadratic and interactive effects of parameters were significant, which means that they could act as limiting conditions and little variation in their magnitude would alter either growth rate or the product formation rate or both to a considerable extent (Imandi et al. 2008). Along with the significance of all model terms, lack of fit of the model was found to be insignificant (p>0.05) which is strongly desirable for the adequate model.

Table 2: Analysis of Variance (ANOVA) for Response Surface Quadratic Model in laboratory scale production

Source	Sum of Squares	DF	Mean sq.	*F*-value	*P*-value > F
Model	42180.86	5	8436.17	80.20	< 0.0001
Aeration (L/kg/min), A	864	1	864	8.21	0.0241*
Substrate thickness (cm), B	1066.67	1	1066.67	10.14	0.0154*
A2	8682.68	1	8682.68	82.54	< 0.0001**
B2	15981.73	1	15981.73	151.92	< 0.0001**
AB	900	1	900	8.56	0.0222*
Lack of Fit	605.17	3	201.72	6.15	0.0559
R-square	0.9828				
Adj R-square	0.9706				
Adequate Precision	24.75				

** p < 0.01 indicate the model terms are highly significant

* p < 0.05 indicate the model terms are significant

The 3D and 2D plots for the interaction between two variables are presented in Fig. 2. An elliptical response surface in the entire region was found from the second order quadratic equation for citric acid production with interaction of aeration and substrate thickness (Fig. 2). The maximum production of citric acid of 382.50 g/kg-EFB was predicted at aeration level of 2 L/kg/min and substrate thickness of 3 cm within the given ranges of both parameters. Only one numerical solution suggested by using Design Expert software within the experimental range of parameters for the maximum production of citric acid at the desirability level of 95.4%. The numerical solution of the developed model predicts the highest production of citric acid of 382.55 g/kg of dry EFB at 2.2 L/kg/min aeration and 3.2 cm substrate thickness. The validation experiment of the developed second order quadratic model shows the highest citric acid production of 384.21 g/kg-EFB was obtained in optimum conditions, which is slightly higher than the predicted value.

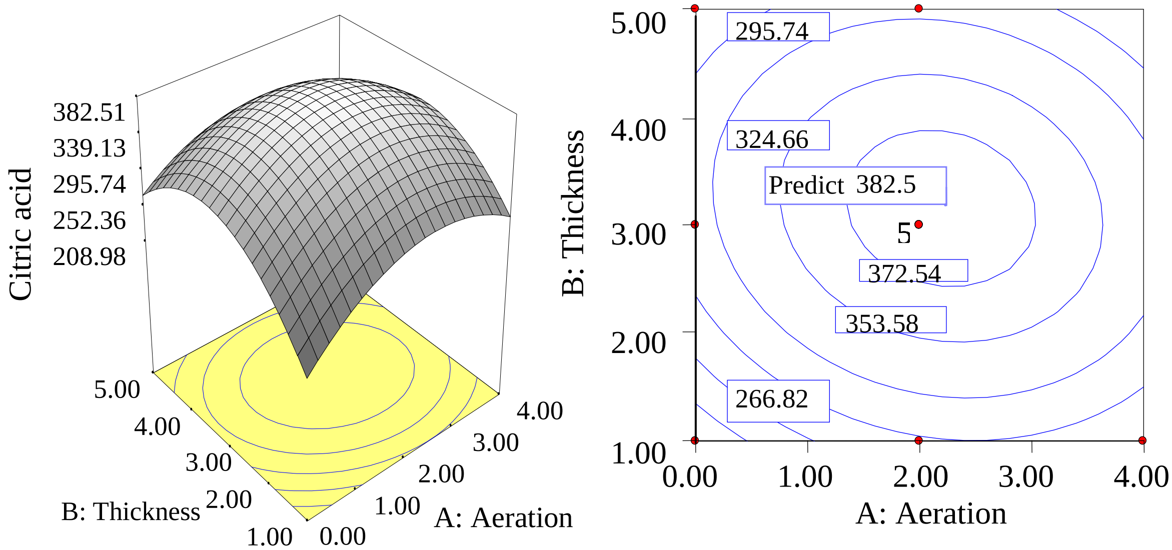


Fig. 2: 3D response surface and 2D contour plots show the effect of aeration (L/kg/min) and substrate thickness (cm) on the production of citric acid (g/kg of dry EFB)

Prado et al. (2005a) observed only the effect of substrate thickness on citric acid production and found highest production of citric acid of 263 g/kg from treated cassava bagasse. So far our knowledge goes; there is no report on statistical optimization of citric acid production in tray fermentor considering aeration and substrate thickness as parameters. Therefore, it is observed from the above discussion that a significant amount of citric acid production was achieved from EFB through solid state bioconversion in tray fermentor with optimum of process conditions.

4. CONCLUSIONS

The possible optimum level of aeration and substrate thickness were determined with the OFAT method. The CCD under the RSM was employed for further optimization of aeration and substrate thickness through the development of second order regression model. The high adequacy of the developed second order regression model was proven by fitting the experimental and predicted values. The variables were tested for the correlation between their level and the production of citric acid, and both variables showed a significant influence on the production. The predicted maximum production of citric acid from EFB was validated to be 384.21 g/kg of dry EFB for optimum aeration of 2.2 L/kg/min and 3.2 cm substrate thickness at the optimum process conditions of moisture content, 70.3% (v/w); incubation temperature, 33.1^{0}C and initial pH, 6.5 in semi-pilot scale production with tray fermentor.

ACKNOWLEDGEMENTS

The authors are grateful to the Ministry of Science, Technology and Innovation (MOSTI), Malaysia, for financing the research project (No. 02-01-08-SF0050) under e-Science research grant and to the Department of Biotechnology Engineering, International Islamic University Malaysia for their support.

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Structural Engineering

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE OF ADOBE BLOCKS RANDOMLY REINFORCED WITH NATURAL FIBERS

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ABSTRACT

Earthen house is one of the traditional housing types in Bangladesh, which is environmentally friendly and cost effective. Earthen houses are so vulnerable to dynamic loading because of their low mechanical properties and brittleness. In this study, jute fibers were used as reinforcing materials to improve the performances of adobes. A series of uniaxial compression tests were executed to investigate the effectiveness of the proposed natural reinforcing fibers with soil matrix for different combinations of adobe blocks. The results obtained from compression tests were outlined as stress-strain relationship to compare both of the strength and ductility properties of each combination. Test results revealed that jute is effective to improve both the strength and ductility of adobe blocks.

Keywords: *Adobe, Fiber reinforced soil, Jute, Compressive strength, Ductility*

1. INTRODUCTION

Earth as a construction material, has been using historically for thousands of years in the world in different cultures and countries because of its availability, cheapness and environmentally friendliness. Around 30% of the world's population still lives in earth made constructions (Ren and Kagi, 1995). Among different types of earth made constructions, adobe is one of the oldest and most widely used building materials in the world. Adobe is a sun-dried earth blocks which are commonly used in some regions of the world such as Latin America, Africa, the Indian subcontinent and other parts of Asia, the Middle East, and southern Europe because of their many folds technical advantages (Blondet and Garcia, 2003).

Earthen house is one of the traditional housing types in Bangladesh which has been practicing for more than 200 years in almost all rural areas by low income people because of its simplicity and low cost (Das et al., 2007). From the cost analysis among Mud, Masonry, and R.C.C houses, it was found that the construction cost (excluding roof) of Mud house (42′x11′x10′) per square foot is only 1.26 US $ (Bhuiyan and Kayser, 2009). In Bangladesh, earthen houses like adobe and rammed earth both of those are locally called as Mati-Ghor (Mati means earth and Ghor means house). From the housing pattern in Bangladesh, it is found about 18% houses are Mud house (HIES, 2005). According to Chowdhary's (1995) report, the division wise distributions of mud houses in Bangladesh are, 50-60% in Rajshahi (Northwestern part), 40-50% in Rangpur (Northernmost part), 20-30% in Khulna and Chittagong (South-west and Southernmost parts respectively), 10-15% in Dhaka and Sylhet (Central and North-eastern parts respectively), 1-5% in Barisal (South-central part). From this report it can be said that mud house predominates in some parts of Bangladesh due to influence of environmental and climatologic factors of those regions.

In the recent time, considerable researches have been carried out for the improvement of adobe houses and most of these works have been focused on the improvement of its physical and mechanical properties using different stabilizers, binders (natural fibers) and retrofitting techniques. Smith and Redman (2009) reported different techniques of retrofitting to improve seismic resistance of adobe houses. Although, these techniques were found to be effective for seismic resistance but they are not suitable to adobe houses in respect to cost effectiveness and sustainability. Feasibility of using stabilizers such as cement, lime, bitumen, cow-dung, gypsum and phosphogypsum were investigated by different investigators such as Ngowi (1997), Isik and Tulbentci (2008), and Degirmenci (2008). These studies were mainly focused on strength, water resistance, and thermal conductivity of adobe. It was also found that stabilizers increase the production cost of adobe and environmental impact (Chen, 2009; Isik and Tulbentci, 2008).

The present study investigates the performance of natural fibers in soil matrix without addition of stabilizers. Jute fibers were used as natural fibers due their affordability and locally availability in Bangladesh. As an agricultural country, Jute is one of the major cultivation crops in Bangladesh. According to the Ministry of Agriculture and Bangladesh Bureau of Statistics, the annual production (2007-08) of jute is 4.62 million bales (1 bale =180 kg).

In the current research, a series of uniaxial compression tests were executed on various combinations of blocks systems to evaluate the performance of the selected reinforcing fibers. Stress-strain relationship graphs were also drawn to compare the effectives of fibers in terms of ductility and strength.

2. MATERIALS AND PROPERTIES

For the adobe production, a locally available lateritic soil (typically red in color and rich in mineral) was collected from Purbachol site in Narayanganj district because of the availability of mud houses and it was also near to the Dhaka city. To characterize the selected adobe soil, the physical property tests were executed such as Atterberg limits, grain size distributions by hydrometer analysis, standard proctor test, and specific gravity (Lambe, 1951). The index properties of the selected adobe soils are summarized in Table 1.

Jute was adopted herein as natural reinforcing fibers to improve the ductility of earthen houses because of their availability, low-price, and environmentally friendliness. Figure.1 presents the chopped fibers of 3.5 cm in length.

Table 1: Index properties of adobe

Property		Value
Atterberg limits	Liquid limit LL (%)	49.00
	Plastic limit PL (%)	22.00
	Plasticity index (%)	27.00
Grain size distribution	Sand (%)	31.00
	Clay (%)	22.00
	Silt (%)	47.00
Specific gravity		2.73

Figure 1: Chopped jute fibers

3. SPECIMEN PREPARATION

To investigate the effectiveness of fiber in adobe, cylindrical block specimens were prepared. Each specimen composed of processed clay and fiber only. At first, clay was dried in a controlled way under sunlight, and then reduced into powder. A controlled amount of water (about to liquid limit) was added in powder clay and this clay-water mixture was kept for 1-2 days for the complete soaking of the clay particles which make the soil paste more homogeneous. Later than the fibrous materials were added (2% jute by weight of soil) thoroughly by kneading and mixing was continued to ensure uniform consistency of the fibers in the soil matrix. The mixture was then poured into a cylindrical steel mold (8”x4”) as a lump, and compacted by wooden bar to reduce the void in the paste. After sufficient hardening, the produced specimens were taken out from the mold and placed in open tin shed room for 30-45 days to dry (Islam et al., 2008). To ensure uniform drying of the prepared specimens were turned over 2 times per day. For the test execution, all laboratory test specimens were leveled by using rubbing mosaic stone (#80) and level meter.

4. EXPERIMENTAL METHOD

The main focus of this study was to evaluate the performance of the selected reinforcing materials (i.e., jute) in terms of compressive strength. To find out compressive strength, a series of uniaxial compressive experiments were conducted of adobe specimens with and without fibers. The compressive strength tests were executed on cylindrical block specimen of size of 4 inch in diameter and 8 inch in height. Three specimens were tested for each combination to check the repeatability of results. The cylindrical block specimens were tested by CBR machine of load capacity 10 KN. Rate of strain was calculated using elapsed time and it was around 1-1.50% per minute.

5. RESULTS AND DISCUSSIONS

Stress-strain relationships of the tested unreinforced and jute reinforced blocks are reported in Figures 2 and 3 respectively. From Figure 2, it is observed that plain adobe (unreinforced blocks) is brittle in behavior. On the contrary, jute reinforced adobe shows significant ductile behavior (Figure 3).Table 2 outlines the physical properties of the tested specimens. It is seen that the ultimate compressive strength (q_u) of reinforced blocks are about 20-68% higher than plain blocks. Due to fiber resistant, it can also be noticed that the failure strain (3.90-4.10%) of reinforced adobe is much higher than unreinforced adobe. The same strain behavior has previously been observed by (Islam et al., 2006). In soil matrix jute acting as a binder and tensile element when block tries to expand under compression load which increases the energy absorption capacity as well as ductility. Therefore, the strength and failure strain of jute reinforced blocks are higher than unreinforced blocks.

From the failure pattern (Figure 4) of unreinforced and reinforced specimens, it was seen that the unreinforced block failed in shear and failure occurred suddenly (Figure 4a). On the other hand, reinforced block also failed in shear but showed ductile behavior (Figure 4b). It indicates that jute is effective to improve ductility and compressive strength. It was also viewed that jute has the great control to minimize shrinkage cracks.

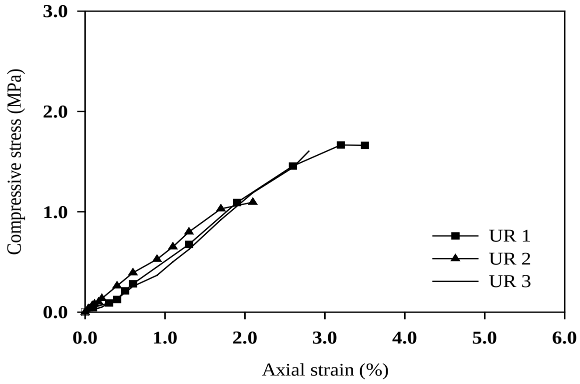

Figure 2: Stress-strain relationships of unreinforced (UR) cylindrical blocks

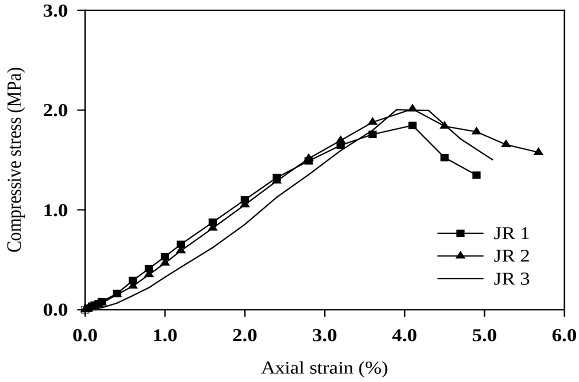

Figure 3: Stress-strain relationships of jute reinforced (JR) cylindrical blocks

Table 2: Characteristics of cylindrical blocks

Specimens combination	Final water content (%)	Ultimate comp. Strength q_u (MPa)			Failure strain ε_f (%)			Failure pattern
		Min.	Max.	Avg.	Min.	Max.	Avg.	
UR	0.94-1.12	1.10	1.67	1.39	2.10	3.20	2.65	Shear & brittle
JR	1.92-2.38	1.85	2.01	1.93	3.90	4.10	4.00	Shear &ductile

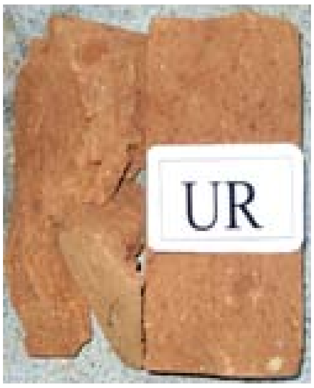

Figure 4: Failure pattern of unreinforced (left) and jute reinforced block (right)

6. CONCLUDING REMARKS

Jute has the abundance in Bangladesh. Jute was used in the present research as reinforcement to improve adobe performance. Effectiveness of jute fibers in soil matrix was studied by conducting a series of uniaxial compression tests. The results obtained from uniaxial compression tests can be recapped as follows:

1. Strength of unreinforced block is much lower than jute reinforced block and failure strain also. Jute reinforced block shows significant ductile behavior while unreinforced block is brittle.

2. It can be said that jute as a reinforcing fiber is effective one with respect to strength and strain criteria. The properties of jute such as flexibility and strong coherence to soil may be responsible for its better performance in soil matrix.

7. RECOMMENDATIONS

In today's world sustainable development is one of the most important issue to maintain ecological balance. Adobe is a good alternative for sustainable development, where locally available environmental friendly raw materials are used for its construction. To adopt, fiber reinforced adobe as a building materials it is needed more details study about durability of fiber-soil composite and softening in water contact. Moreover, a microstructure study is also required to realize bond characteristics within fiber reinforced soil.

ACKNOWLEDGEMENTS

The financial support for this study was provided by a grant from Department of Civil Engineering, BUET awarded to the second author.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STUDY THE ROLE OF BUILDING SETBACK AND HEIGHT, IN TERMS OF INDOOR TEMPERATURE DUE TO THE DIRECT COMPONENT OF SOLAR RADIATION ON WEST FAÇADE OF RESIDENTIAL BUILDINGS: COMPARING 1996 AND 2008 'IMARAT NIRMAN BIDHIMALA'

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ABSTRACT

New buildings are being constructed under the 2008 building construction act, termed as 'Dhaka mohanagar imarat nirman bidhimala 2008' which results more open space, provides more green and the result of applying FAR increased the building height to accommodate the required floor area through providing more open spaces. Increase in building heights with certain setbacks resulted increase in possibilities of change in natural indoor lighting at the lower level of the structures, as a result use of artificial lighting in indoor spaces may be increased which may result change in fossil fuel consumption for power generation. Beside the indoor day lighting condition the question arises what would be the situation in terms of indoor temperature due to building to building setback; is there any impact of indoor temperature due to the setback provided in the construction rules and the building height achieved in this regard? The International Energy Agency projected that global final energy consumption in buildings would grow by 30 percent from 2007 to 2030 if prevailing practices and trends continued. Most of that increase is expected to come from fast growing developing countries (Liu,Meyer and Hogan,2010). So, on the basis of above discussion the focus of the paper will be trying to investigate and find out whether there are any impact in indoor temperature at different floor levels facing 'west' considering only the direct component of solar radiation; due to building setback and height of neighborhood structures in terms of the present building code for residential buildings comparing the previous 1996 building by-law.

Key words: *indoor temperature, building code, building setback, building height, floor levels,*

1. INTRODUCTION

A country develops through expanding its urban areas day by day. Bangladesh is one of those developing countries of which the capital city Dhaka is expanding which resulted as one of the mega cities of the world. Naturally numbers of buildings are increasing to accommodate the increasing population staying and penetrating this mega city. Regulations for energy efficiency in buildings in developing countries, seek to improve comfort and to reduce the dramatic increase in energy consumption (Laustsen,2008). Energy is used in buildings for various purposes: heating and cooling, ventilation, lighting and the preparation of hot sanitary water are among them. Use of Fossil Fuels has resulted in emission of huge quantity of carbon dioxide causing serious environmental damages. There is still a considerable potential for reducing energy consumption by adopting energy efficiency measures at various sectors of our country (Sayeed, 2005). In 1961 Denmark established one of the first building codes which systematically regulated energy consumption. Since then, building codes have been updated several times, including major changes in 1972, 1979, 1997, and in 2006 (Laustsen,2008) Mandatory energy efficient design requirements for buildings were first introduced in Europe and North America in the late 1970s and have proven to be an effective policy instrument. Several developing countries began similar efforts in the 1990s,and many more joined the pursuit in the last decade (Liu,Meyer and Hogan,2010) Today, mandatory minimum energy efficiency requirements in the form of building codes or standards exist in nearly all OECD countries(Organisation for Economic Co-operation and Development).
As global warming is one of the burning issues now-a-days, energy efficient building code is necessary for a country to sustain at present world, but research work and analysis regarding the present building code comparing other codes focusing energy efficiency is required to investigate. Some selected objectives in terms of 'energy sources' and 'sustainable development' given by the CIBSE (Chartered Institute of Building Services) are given below: (CIBSE, 2004)

Energy sources:

- to mitigate the demands placed on the world's reserves of fossil fuels
- to reduce the consumption of fossil fuels, in order to reduce pollution
- to promote the use of renewable and sustainable energy sources and passive design solutions

Sustainable development:

-Consider within the design of systems and products how, through design, to reduce energy use, waste and pollution in construction, transportation, installation and use.

2. PROBLEM STATEMENT

The previous 1996 building by-law which existed from 1996 to 2006 resulted many buildings under the law; as mandatory open space, except the set-back were not present in that law, closely spaced buildings resulted bulk of concrete structures in the city, using more and more fossil-fuels to generate power for mitigating darker interior of the lower levels of the structures which resulted the necessity of the new building code, with a provision of mandatory open space to penetrate daylight and to provide more green spaces through applying floor area ratio (FAR). As green spaces decreased day by day due to 1996 by-law, that resulted climate change year after year and increased the temperature of different areas of urban Dhaka. The urban building stock in developing countries is expected to more than double by 2030.Demand for energy services in buildings in developing countries will rise substantially in the next two decades, driven by population growth, urbanization, and increased and expanded wealth (Liu,Meyer and Hogan,2010). Building energy standards and codes have been developed and used in many countries to provide a degree of control over building design and to encourage awareness and innovation of energy conscious design in buildings (Hui, 2002). , but research work and analysis regarding the present building code comparing other codes in terms of lighting and thermal performance are required to investigate So, the focus of the paper will be trying to investigate and find out whether there are any impact in indoor temperature at different floor levels of the structure having west façade, considering only the direct component of solar radiation; due to building setback and height of neighborhood structures in terms of the present building code for residential buildings comparing the previous 1996 building by-law.

3. OBJECTIVE

Thus the objective of the paper will be to investigate and find out whether there are any impact in indoor temperature at different floor levels facing ‘west’, considering only the direct component of solar radiation; due to building setback and height of neighborhood structures in terms of the present building code for residential buildings comparing the previous 1996 building by-law.

4. METHODOLOGY

To asses the conditions regarding internal temperature comparing two building codes (1996 and 2008 imarat nirman bidhimala) several equations were taken to see the situation as this study is based on theoretical analysis rather than field survey nor simulation. As the focus of this paper is to compare the two codes in identical situation, several values were considered to be same because of the identical manner. Among three components of solar radiation only the direct component were taken into consideration. Five katha (335 sq m) plot were taken for the study as in planned residential area such as Uttara model town lying towards the north of the main city comprises five katha plots. While considering this size of plot, floor levels were considered from ground floor to 5^{th} floor as in the 1996 building construction rule the structures were made of six stories, that is- ground + 5 floors. Regarding orientation west side of the building is considered as this part achieves the highest amount of radiation at the later hours of the day. Similar studies can be done with other orientations as well.
Two scenarios are considered in this study:

1. Two six storey buildings standing side by side having west façade, constructed under 1996 law
2. Two buildings standing side by side were one building constructed under 1996 law and the other building under the new law that is 2008 imarat nirman bidhimala

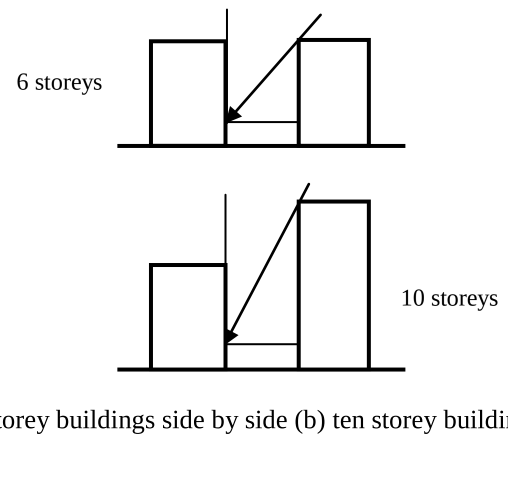

Fig 1(a): two six storey buildings side by side (b) ten storey building beside six storey

As building side to side and rear to rear having two different setbacks, this study also tries to see the difference due to the change in setback assuming both the situations are facing west in different situations. Through this, the change in level of obstruction of the neighboring structures due to the difference in building codes along with change in setback and building height can be observed while calculating the indoor temperature. As there are different parameters while calculating the indoor temperature such as orientation, different floor levels, building to building gap, building height, different plot sizes (as there is a relationship with plot size and building set back), in this study the focus will be given for residential buildings facing west, considering ground to fifth floor and buildings setback in terms of side to side and rear to rear for 1996 and 2008 building codes. Considered walls are of west facing, opposite having a building i) constructed under 1996 law, ii) constructed under 2008 building code (ten storey). At each floor the direct solar radiation are considered to enter at sill level which is at 0.6m from each floor level. Each floor height is considered 3m. The direct solar radiation received at west façade is considered in the month of April, as April is being identified most representative month of hot-dry summer (Ahmed, 1994). The average amount of solar radiation received at west façade is 47 Wh/sq.m (Ahmed 1994). The average high outdoor temperature is considered 36 deg C in the month of April (wikipedia).

As there are three components of the global radiation received at any point on the earth's surface which is as follows:

G(s) = B(s) + D(s) + R (s)……………… (i); where, B(s) = direct (beam) component, D(s) = diffuse component and R(s) = ground reflected component. As this paper focuses only on direct component, B(s) will be considered only; while comparing the two codes in identical situation.

To calculate the direct component irradiation on a place normal to the direction of beam (Bn), we know: *B = Bn/SinALT*, considering the irradiation will be on a plane normal to the direction of beam we get:
Bn = B SinALT…………………….(ii) (ALT being the altitude angle)
In terms of mean heat gains from all sources we get:

Qt = Qs+Qc in watts; where

i) mean solar gain, Qs from
Qs = Se G Ag in watts, therefore, *Qs = G(Se Ag)……………….(iii)*; where Se=mean solar gain factor, G=mean total radiation on relevant face (w/sq m), Ag=area of glazing in sq.m.

As the situation and the materials are assumed to be same in both the situations (1996 and 2008 building construction rules), the only variable will be 'total radiation' (G).

ii) mean casual gain Qc:
Qc = [(qc1xh1) x (qc2xh2)………..]/24 in watts; where qc1 qc2 = instantaneous casual gain in watts, h1 h2= duration of individual casual gains in hours, for same reason Qc will not be applicable for this study.
In terms of mean internal environmental temperature Tei we get:
Qt = (∑Ag Ug + Cv) (Tei-Tao) + ∑ Af Uf (Tei-Teo)………… (iv); where Teo=mean sol-air temperature in degC, Cv=ventilation exchange dependent on ventilation rate, Tao=mean outside air temperature in degC, ∑A= total area of the surfaces bounding the enclosure in sqm.

As, Qt=Qs+Qc, or Qt= [G (SeAg)] +Qc……………. (v),

From (iv) and (v) we get,
[G (Se Ag)]+Qc = (∑AgUg+Cv) (Tei-Tao) + ∑AfUf(Tei-Teo)
or, [G(SeAg)] + Qc = (Tei-Tao) [(∑AgUg+Cv)+(∑AfUf)] [due to vertical surface]
or, [G(SeAg)] = (Tei-Tao) [(∑AgUg+Cv)+(∑AfUf)] [because of identical situation]
or, (Tei-Tao) = [G(SeAg)]/[(∑AgUg+Cv)+(∑AfUf)]
Therefore we get,
Tei = G [(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+Tao…………………..(vi)

5. 2008 IMARAT NIRRMAN BIDHIMALA

'Dhaka Mohanagar Imarat Nirman Bidhimala 2008' basically came into action in the year 2006, later there were some corrections in the code which now is in practice as 2008. According to the present building code (2008) different setbacks (rear and side to side of the buildings) in respect to different plot sizes are given below:

5.1 Building setback

Site area	Setback	
	Rear Side of the Building	Both Side of the Building
Up to 134 sqm	1m	0.8m
134sqm-201sqm	1m	1m
201sqm-268sqm	1.5m	1m
268sqm-335sqm	2m	1.25m
335sqm-402sqm	2m	1.25m
402sqm-469sqm	2m	1.25m
469sqm-536sqm	2m	1.25m
536sqm-603sqm	2m	1.25m
603sqm-670sqm	2m	1.25m
670sqm-804sqm	2m	1.25m
804sqm-938sqm	2m	1.25m
938sqm-1072sqm	2m	1.25m
1072sqm-1206sqm	2m	1.25m
1206sqm-1340sqm	2m	1.25m
Over 1340sqm	2m	1.50m

(Source: 'Dhaka Mohanagar Imarat Nirman Bidhimala 2008')

Table 1: Rear and both side of building setbacks according to different plot sizes : 'Dhaka Mohanagar Imarat Nirman Bidhimala 2008'

5.2 Building height

According to 'Dhaka Mohanagar Imarat Nirman Bidhimala 2008' the focus has been given on FAR (floor area ratio), M.G.C. (maximum ground coverage) and the front road width of the respective plots rather than building heights. The chart given above has been restricted to 10 storey buildings. So this paper focuses buildings up to 10 storey structures.

6. 1996 BUILDING CONSTRUCTION BY-LAW

The building construction by-law that was in practice from 1996 to 2006 was known by 'Imarat Nirman Bidhimala, 1996'. Some of the basic data of Rajuk rules, which were in practice during that time, are given in this section.

6.1 Building setback

Site area	Setback	
	Rear Side of the Building	Both Side of the Building
Up to 134 sqm	1m	0.8m
134sqm-200sqm	1m	1m
200sqm-268sqm	1.5m	1m
More than 268sqm	2m	1.25m

(Source: Imarat Nirman Bidhimala, 1996)

Table 2: Rear and both side of building setbacks according to different plot sizes : 'Imarat Nirman Bidhimala, 1996'

6.2 Building height

According to this law the height of the buildings were depended upon the front road width. If the front road width were 75' or above then the height of the building could be unlimited or else in most cases the height of the buildings, specially the residential buildings were restricted up to six storey (5+1) structures.

7. INDOOR TEMPERATURE DUE TO DAYLIGHT INTRUSION (CONSIDERING DIRECT COMPONENT OF SOLAR RADIATION)

While calculating the indoor temperature due to daylight intrusion, considering only the direct component of solar radiation, building set back due to side to side and rear to rear according to the two different codes calculated separately, which are given below.

7.1 Building side to side

The set back considered in terms of building side to side is 1.25x2=2.5m according to 2008 and the same goes for 1996 building construction rule for 5 katha plots, which is the considered plot size for this study.

7.1.1 Temperature at different floors after applying 2008 building construction code

First of all required angles were calculated, after that respective indoor temperature equation is used to see the difference of temperature at different floors. From equations (ii) and (vi) we get the following calculations:

Ground floor:	$G=47x\sin 4.8=3.93;$	$Tei=3.93[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
1st floor:	$G=47x\sin 5.33=4.37;$	$Tei=4.37[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
2nd floor:	$G=47x\sin 6=4.91;$	$Tei=4.91[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
3rd floor:	$G=47x\sin 6.89=5.64;$	$Tei=5.64[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
4th floor:	$G=47x\sin 8.04=6.57;$	$Tei=6.57[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
5th floor:	$G=47x\sin 9.72=7.94;$	$Tei=7.94[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$

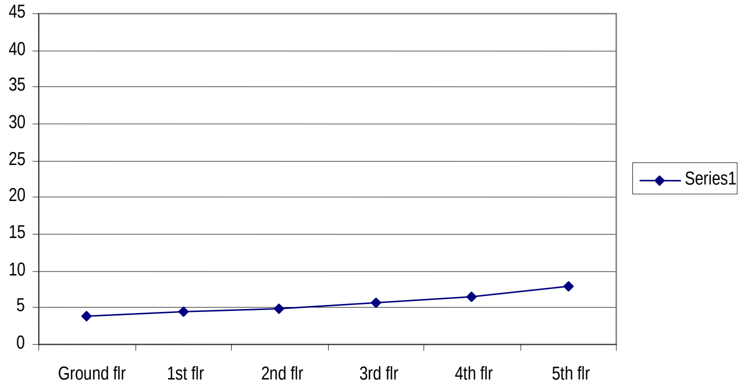

Fig 2: 2008 building code (building side to side)-floor wise comparison

7.1.2 Temperature at different floors after applying 1996 building construction rule

First of all required angles were calculated, after that respective indoor temperature equation is used to see the difference of temperature at different floors. From equations (ii) and (vi) we get the following calculations:

Ground floor:	$G=47x\sin 8.08=6.61;$	$Tei=6.61[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
1st floor:	$G=47x\sin 9.7=7.92;$	$Tei=7.92[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
2nd floor:	$G=47x\sin 12.26=9.98;$	$Tei=9.98[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
3rd floor:	$G=47x\sin 16.39=13.26;$	$Tei=13.26[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
4th floor:	$G=47x\sin 24.44=19.45;$	$Tei=19.45[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$
5th floor:	$G=47x\sin 46.17=33.91;$	$Tei=33.91[(SeAg)/(\sum AgUg+Cv)+(\sum AfUf)]+36$

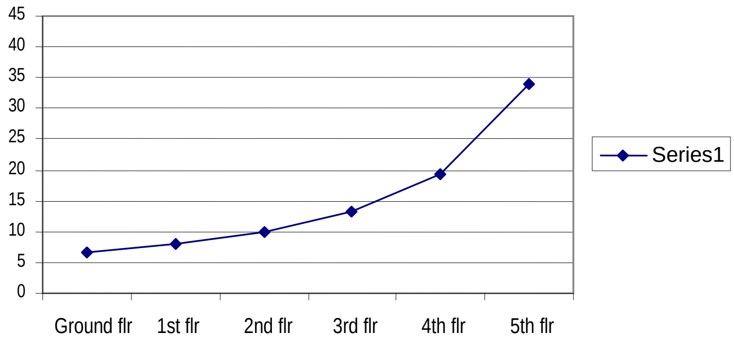

Fig 3: 1996 building code (building side to side)-floor wise comparison

7.2 Building rear to rear

The set back considered in terms of building rear to rear is 2x2=4m according to 2008 and the same goes for 1996 building construction rule for 5 katha plots, which is the considered plot size for this study.

7.2.1 Temperature at different floors after applying 2008 building construction code

First of all required angles were calculated, after that respective indoor temperature equation is used to see the difference of temperature at different floors. From equations (ii) and (vi) we get the following calculations:

Ground floor:	*G=47xsin7.65=6.26;*	*Tei=6.26[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
1st floor:	*G=47xsin8.49=6.94;*	*Tei=6.94[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
2nd floor:	*G=47xsin9.54=7.79;*	*Tei=7.79[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
3rd floor:	*G=47xsin10.94=8.92;*	*Tei=8.92[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
4th floor:	*G=47xsin12.7=10.33;*	*Tei=10.33[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
5th floor:	*G=47xsin15.32=12.42;*	*Tei=12.42[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*

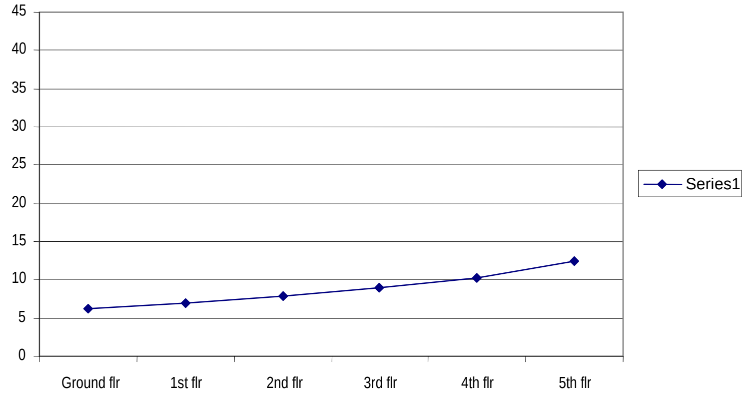

Fig 4: 2008 building code (building rear to rear)-floor wise comparison

7.2.2 Temperature at different floors after applying 1996 building construction rule

First of all required angles were calculated, after that respective indoor temperature equation is used to see the difference of temperature at different floors. From equations (ii) and (vi) we get the following calculations:

Ground floor:	*G=47xsin12.8=10.41;*	*Tei=10.41[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
1st floor:	*G=47xsin15.32=12.42;*	*Tei=12.42[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
2nd floor:	*G=47xsin19.18=15.44;*	*Tei=15.44[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
3rd floor:	*G=47xsin25.2=20.01;*	*Tei=20.01[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
4th floor:	*G=47xsin36=27.63;*	*Tei=27.63[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*
5th floor:	*G=47xsin59=40.29;*	*Tei=40.29[(SeAg)/(∑AgUg+Cv)+(∑AfUf)]+36*

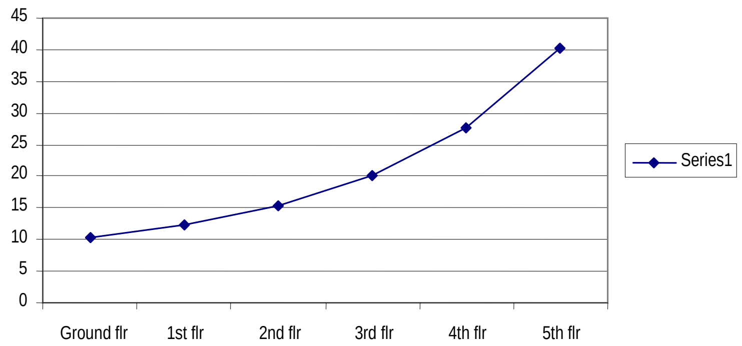

Fig 5: 1996 building code (building rear to rear)-floor wise comparison

8. COMPARATIVE ANALYSIS (COMPARING 2008 AND 1996 BUILDING CONSTRUCTION RULES)

For comparing the two codes, it has been divided into two parts for better understanding in terms of building to building gap, as there are two aspects in setbacks, which are building side to side and rear to rear setback; but the situations were different, because all time the considered façade of building was selected to be west facing, as the study focuses on west facing facades only. As the situation was considered identical in terms of building material, size of the openings, quantity of glazing materials, environment and so on, only building to building gap and floor levels were considered for both the codes as mentioned above; for comparing, only the variables were considered in the graphs to see the differences for better understanding in terms of indoor temperature.

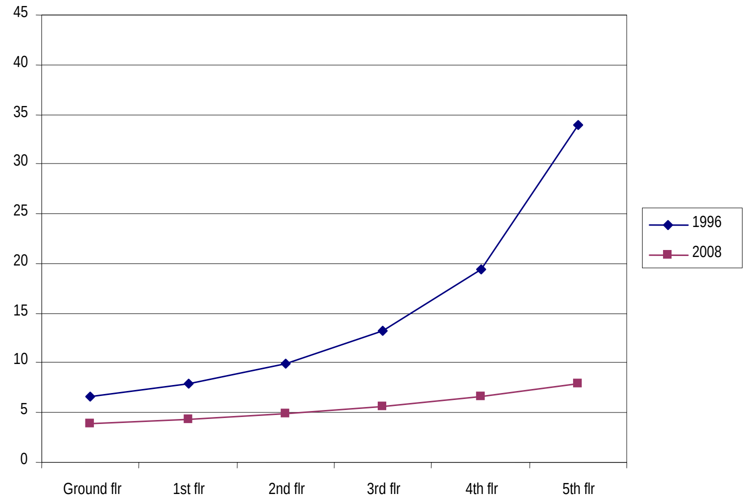

Fig 6: Comparing 1996 and 2008 building code (building side to side) Comparative study

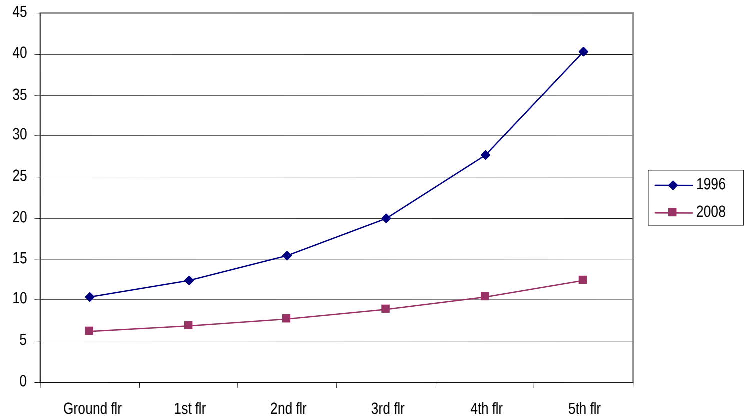

Fig 7: Comparing 1996 and 2008 building code (building rear to rear) Comparative study

9. FINDINGS

It is quiet clear from the graphs above, after comparative study between 1996 and 2008 building code that, as the setback increases the amount of solar radiation intrusion indoor increases which results increase in indoor temperature. More over due to increases in building height after applying 2008 building code, affects the adjacent building in terms of indoor temperature in a huge level. The steepness of the lines in the graph can be observed that, in terms of 1996 building code shifts rapidly from 2nd floor to 3rd floor and above; where as, this remains more over close to each other in terms of applying 2008 building code. Another feature can be observed, in terms of 'building side by side' calculation, there is a sudden jump in indoor temperature from 4th floor to 5th floor when applying 1996 building code, where as in terms of 'building rear to rear' the shift of indoor temperature from 4th floor to 5th floor is not the same.

10. CONCLUSION

From the above study it is clear that there are changes in indoor temperature between the studied building construction codes. The relative indoor temperature changes due to the differences in building set back and the height of the building and the change in the two codes are quiet high in terms of comparative indoor temperature. Further detail studies are required to analyze the indoor temperature at different floors of the buildings constructed under 2008 building construction rules. This study was focused on west facing façade only. Similar studies can be done in other orientations with relationship to different floor levels. Various plot sizes can be considered as well, as there is a relationship between plot size and building set back.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ASSESSMENT OF SEISMIC VULNERABILITY USING DIFFERENT SITE CONDITIONS

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ABSTRACT

With rapid population growth over the past 30 years and frequent seismic activity during this period, the unchecked developments of the built environment in Bangladesh have been resulted in toughness to earthquake. Identifying structures that have high vulnerability is of critical importance for both reliable loss estimation as a result of an expected strategic earthquake and setting priority criteria for strengthening of those buildings. In this study building assessment allowing for different site conditions based on statistical method as demonstrated in Istanbul, Turkey has been considered to evaluate seismic vulnerability of reinforced concrete structures. In order to extrapolate this risk assessment procedure to other regions that are likely to be subjected to major earthquakes, a Cut off value is determined which depends on the soil condition. The cutoff values recommended are considered to be valid for damaging earthquakes and the regions that have similar "distance to source" and site conditions to that of 1999 Duzce earthquake. To apply the procedure to the sites, which have different distance to source and soil properties than Duzce, modifications to the cutoff values have been computed. The sites are classified based on shear wave velocity of the soil. Different soil sites were considered using different shear wave velocity of the soil. Finally vulnerability of RC structures was determined for different site conditions based on shear wave velocity of soil.

Keywords: *RC structures, vulnerability, discriminant analysis, shear wave velocity, cutoff value*

1. INTRODUCTION

Earthquake occurs when energy stored within the earth, usually in the form of strain in rocks suddenly releases. The energy is transmitted to the surface of the earth by waves. Most earthquakes are minor tremors, while larger earthquakes usually begin with slight tremors, rapidly take the form of one or more violent shocks and end in vibrations of gradually diminishing force called aftershocks. Earthquake is a form of energy of wave motion, which originates in a limited region and then spreads out in all directions from the source of disturbance. Seismic waves generated by an earthquake source are commonly classified into three main types. The first two, the P and S waves are propagated within the earth, while the third consisting of Love and Rayleigh waves, is propagated along its surface. P waves are the first to reach any point on the earth's surface. At all distances from the focus, the mechanical properties of the rocks, such as incompressibility, rigidity and density, play an role in the speed with which the waves travel and the shape and duration of the wave trains. The layering of rocks and the physical properties of surface soil is also affecting these characteristics of the waves. In most cases, elastic behavior occurs in earthquakes, but the shaking of surface soils from the incident seismic waves sometimes results in nonelastic behavior, including slumping and the liquefaction of sandy soil. When a seismic wave encounters an interface or boundary that separates rocks of different elastic properties, it undergoes reflection and refraction. There is a special complication if a conversion between the wave types occurs at such a boundary: either an incident P or S wave can yield in general reflected P and S waves and refracted P and S waves. Boundaries between structural layers also give rise to diffracted and scattered waves. These additional waves are in part responsible for the complications observed in ground motion during earthquakes.

The earthquake risk at any location depends on the seismic hazard as well as the vulnerability of its structure. The seismic hazard evaluation considers the likelihood of earthquake of a particular magnitude or intensity affecting a site. Seismic vulnerability, on the other hand, depends on the construction practice in the area and quality of building and soil condition. Every damaging earthquake reaffirms the importance of seismic hazard and risk analysis for estimating the consequences of an earthquake. Here hazard means a threatening event, or the probability of occurrence of a potentially damaging phenomenon within a given time period and area. Risk means expected losses due to a particular hazard for a given area and reference period. Based on mathematical calculations, risk is the product of hazard and vulnerability.

In view of the earthquake hazard mitigation, various organizations in the earthquake threatened countries have come up with documents which serve as guidelines for the assessment of the expected performance and safety of existing building as well as for carrying out the necessary rehabilitation. In USA, stepwise seismic vulnerability evaluation process is proposed. The evaluation steps are namely rapid visual screening, evaluation in detail and engineering evaluation. Ohkubo and Otani describe Japanese standard for the evaluation of the seismic vulnerability of the existing low-rise reinforced concrete buildings. The screening procedure in Japanese standards is mainly based on examination of story shear of columns and structural walls. Hasan and Sozen proposed a simplified method for the classification of low-rise monolithic reinforced structures in a given region according to their seismic vulnerability. The main theme of this method is to identify buildings that have high probability of severe damage during a strong motion. The ranking process is basically depends on the total floor area of the building and cross-sectional areas of columns, shear walls and masonry walls. Gulkan and Sozen also proposed a methodology in order to estimate the seismic vulnerability of reinforced concrete frame buildings with masonry infill. The method requires only total floor area, cross sectional dimensions of columns and masonry infill walls. Most of the methodology requires only structural configuration to evaluate the seismic vulnerability. Seismic risk assessment procedure given by Yucemen et al consider the effect of different site conditions for vulnerability. This method is applied considering variable site conditions for a typical reinforced concrete structures and analyzed in this paper.

2. SEISMIC RISK ASSESSMENT PROCEDURE

In many instances statistical analysis based on the observed damage and significant building attributes would provide reliable and accurate results for regional assessments. Yucement et al employed the discriminant analysis technique to develop a preliminary evaluation methodology for low to medium rise RC buildings in Turkey. This procedure relies on the damage cutoff values developed using a statistical approach based on the damage data compiled from Duzce in 1999 earthquakes. Some selected building attributes are entered into a relation obtained from the discriminant analysis to compute a damage score. The analyses are conducted for both "Life Safety Performance Level" and "Immediate Occupancy Performance Level". In Life Safety Performance Level, the main objective is to identify the buildings, which have a tendency to suffer severe damage or to collapse under the design earthquake. However, Immediate Occupancy Performance Level, the main concern is to identify the buildings that are suitable for habitation immediately after the design earthquake. The main structural parameters are number of stories, minimum normalized lateral stiffness index, minimum normalized lateral strength index, normalized redundancy score and soft story index. The basic equations are given below:

$$DI_{LS} = 0.563 * n - 0.082 * mnlsti - 0.161 * mnlsi - 0.502 * nrs + 0.443 * ssi + 0.201 * or \quad (1)$$

$$DI_{IO} = 0.720 * n - 0.112 * mnlsti - 0.095 * mnlsi - 0.488 * nrs + 0.070 * ssi + 0.285 * or \quad (2)$$

In the above equation, DI_{LS}: damage index corresponds to Life Safety Performance Level, DI_{IO}: damage index corresponds to Immediate Occupancy Performance Level, n: number of story, mnlsti: minimum normalized lateral stiffness index (indicate lateral rigidity of ground story), mnlsi: minimum normalized lateral strength index (indicate base shear capacity of the most critical story), nrs: normalized redundancy score, ssi: soft story index, or: overhang ratio.

This damage score is then compared with a cutoff value, which identifies the buildings as "High Risk", "Low Risk" or "Moderate Risk". The cutoff values recommended are considered to be valid for damaging earthquakes and the regions that have similar "distance to source" and site conditions to that of 1999 Duzce earthquake. To apply this procedure to the sites, which have different "distance to source" and "soil properties" than Duzce, modifications to the cutoff values have been computed. The effects of distance to source and soil type are the two main parameters that should be taken into account. The sites are classified by distance to source (ds) and soil type. Five distances to source were selected in view of the variation in the response spectra with the distance. Soil types were determined based on the shear wave velocity of the soil. Twenty different site classes were obtained from the combination of distance to source and soil types, which are illustrated in the following table.

Table 1: Site classification

Soil Type	Shear wave velocity, v_s (m/sec)	Distance to Source (km)				
		0-4	5-8	9-15	16-25	>26
A	0-200	A1	A2	A3	A4	A5
B	201-400	B1	B2	B3	B4	B5
C	401-700	C1	C2	C3	C4	C5
D	701+	D1	D2	D3	D4	D5

The cut off modification coefficient value for different types of soil with different distance to source and for different magnitude are shown in the table 2.

Table 2: Cutoff Modification Coefficients (CMC)

Magnitude of Earthquake, M=6.0						
Soil Type	Shear wave velocity, v_s (m/sec)	Distance to Source (km)				
		0-4	5-8	9-15	16-25	>26
A	0-200	1.505	1.886	2.802	3.552	3.616
B	201-400	1.920	2.181	2.927	3.752	3.874
C	401-700	2.479	2.840	3.604	5.063	5.238
D	701+	3.053	3.509	4.477	6.340	6.56
Magnitude of Earthquake, M=6.5						
Soil Type	Shear wave velocity, v_s (m/sec)	Distance to Source (km)				
		0-4	5-8	9-15	16-25	>26
A	0-200	0.871	1.012	1.250	1.674	2.469
B	201-400	1.115	1.411	1.809	2.577	4.033
C	401-700	1.349	1.727	2.396	3.461	5.471
D	701+	1.577	2.116	2.964	4.309	6.832

cutoff values for each performance levels is then calculated using the following equations:

$$CV_{LS} = LS_{CVR} + |LS_{CVR}| * (CMC - 1) \quad (3)$$

$$CV_{IO} = IO_{CVR} + |IO_{CVR}| * (CMC - 1) \quad (4)$$

Where, CV_{LS}: cutoff values corresponds to Life Safety Performance Level, CV_{IO}: cutoff values corresponds to Immediate Occupancy Performance Level. The damage index for both performance levels are computed and compared with the cut off values to identify the structural vulnerability.

3. ANALYSIS OF TYPICAL STRUCTURE

In this analysis, a typical reinforced concrete frame structure is considered having 5 spans in X-direction and 4 spans in Y-direction and shown in Figure 1. Column size is taken as 300 x 300 mm for exterior column and 300x450 mm for interior column. All the structural parameters (mnlsti, mnlsi, nrs, or) have been calculated and by using the equations. Damage index corresponds to the “Life Safety Performance Level” and “Immediate Occupancy Performance Level” are calculated. After that, cutoff values relating to different soil type is calculated for two earthquake magnitude M=6.0 and M=6.5. Finally vulnerability of the structure is determined.

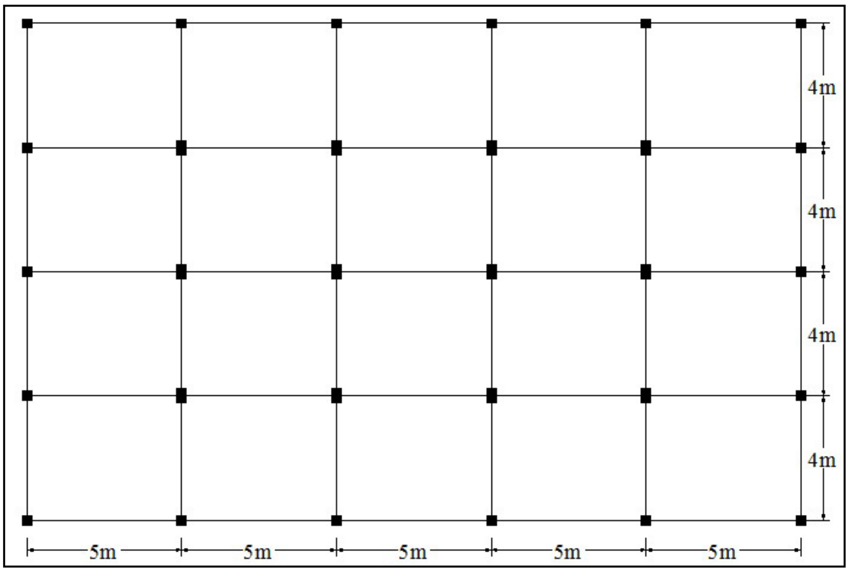

Figure 1: Plan view of a typical RC structure

4. RESULT

Seismic vulnerability assessment of a typical RC structure for different types of soil and different distance to source has been analyzed and shown in the following figure 2 to figure 11

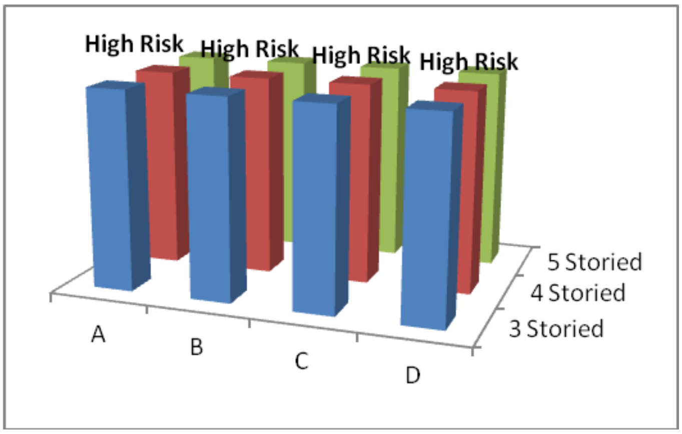

Figure 2: Vulnerability for different soil types (Distance to source: 0-4 km, M=6.0)

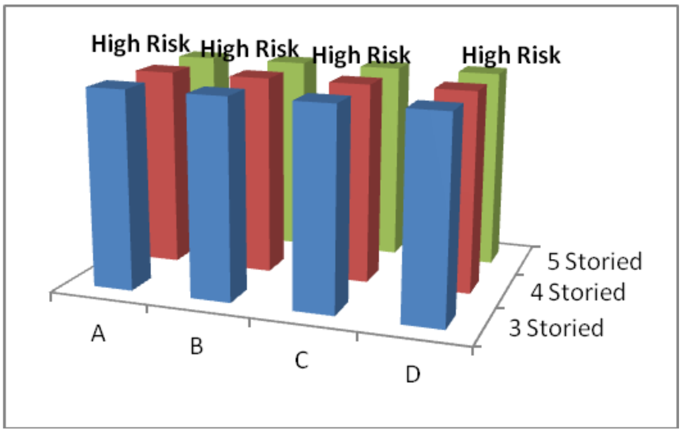

Figure 3: Vulnerability for different soil types (Distance to source: 5-8 km, M=6.0)

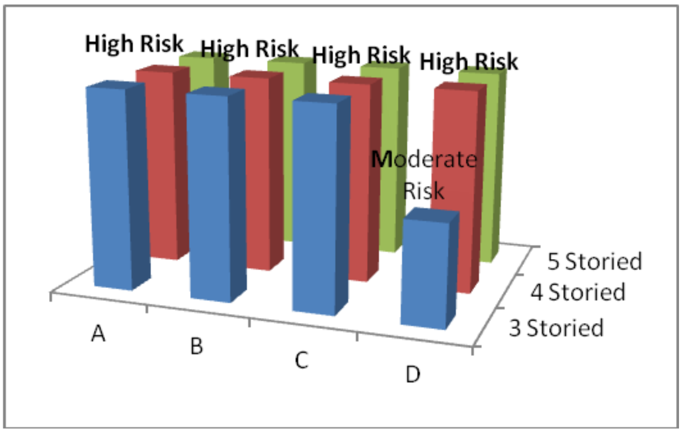

Figure 4: Vulnerability for different soil types (Distance to source: 9-15 km, M=6.0)

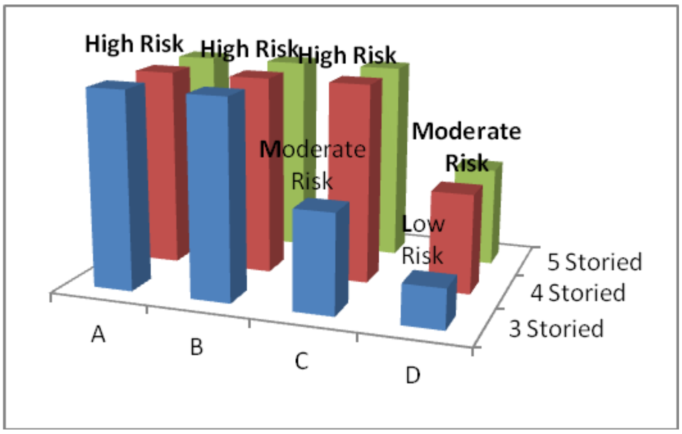

Figure 5: Vulnerability for different soil types (Distance to source: 16-25 km, M=6.0)

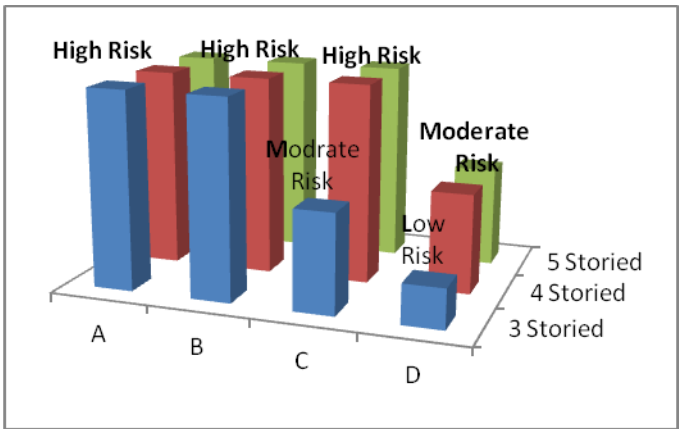

Figure 6: Vulnerability for different soil types (Distance to source: >26 km, M=6.0)

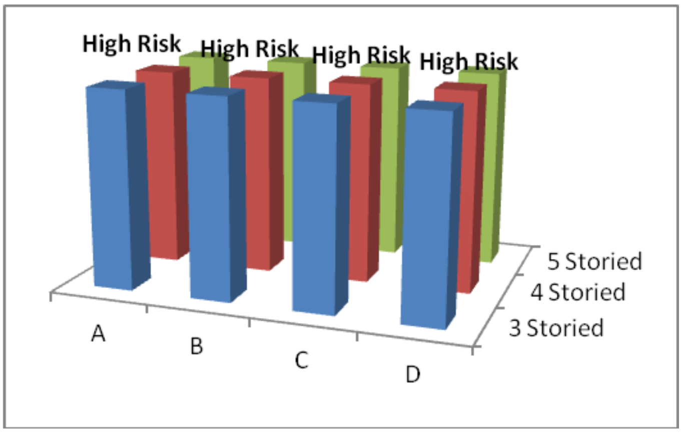

Figure 7: Vulnerability for different soil types (Distance to source: 0-4 km, M=6.5)

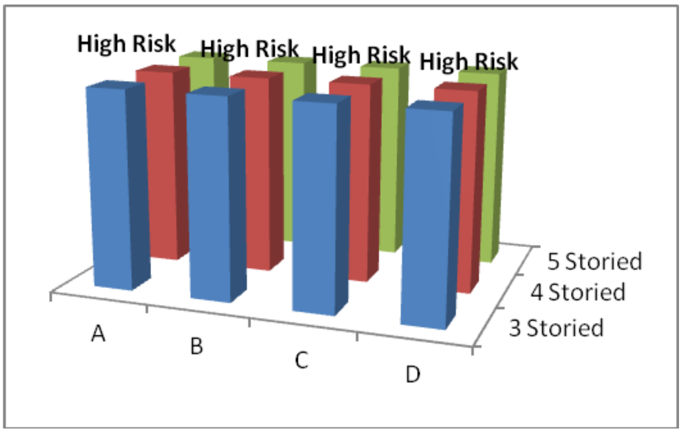

Figure 8: Vulnerability for different soil types (Distance to source: 5-8 km, M=6.5)

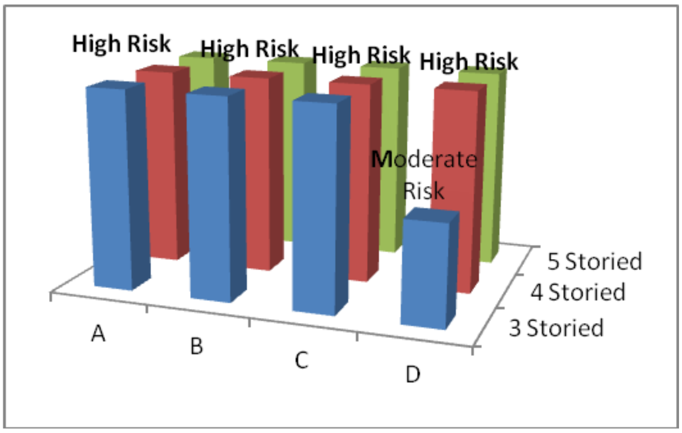

Figure 9: Vulnerability for different soil types (Distance to source: 9-15 km, M=6.5)

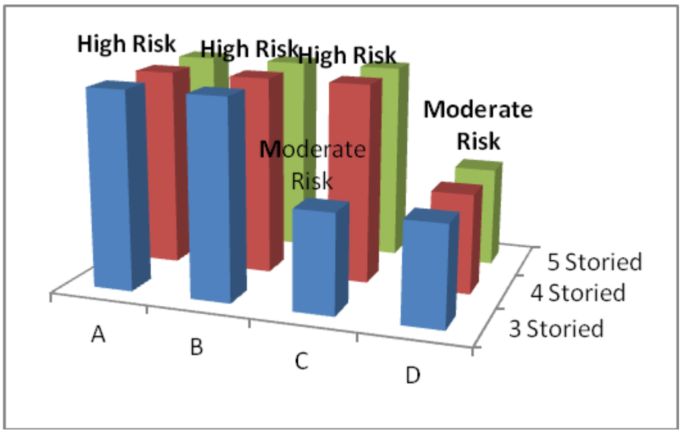

Figure 10: Vulnerability for different soil types (Distance to source: 16-25 km, M=6.5)

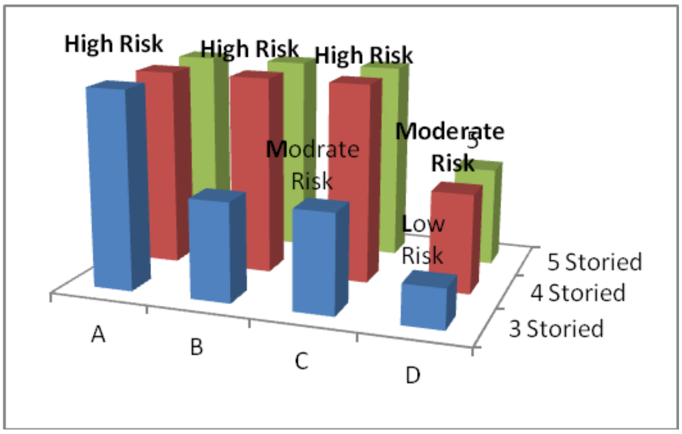

Figure 11: Vulnerability for different soil types (Distance to source: >26 km, M=6.5)

From the above analysis it is shown that, structure lies in High Risk group for the distance to source up to 5-8 kilometers for magnitude 6.0. Only a five storied structure built on D type soil lies in Moderate risk group in case of 9-15 kilometer distance to source. Any low rise structures can be treated as High Risk group built on A & B type soil when distance to source is greater than 15 kilometers. For C type soil, 4 storied and 5 storied buildings lies in High Risk group. Structure can be treated as Moderate Risk group as they are constructed on D type soil. Furthermore, only the structures build on D type soil and distance to source greater than 16 km has been grouped as Moderate risk group.

5. CONCLUSION

Typical reinforced concrete structures from three to five storied has been considered in the analysis using different soil condition. Soil is classified based on shear wave velocity. Structures grouped as high risk when it is near the source of earthquake. It can be categorized from high risk to moderate risk group with the increase of distance to source of excitation and shear wave velocity of soil. However, the above described seismic risk assessment procedure have completed within limited facilities and short period. Future extensive research can be carried out considering dynamic structural analysis, detailed soil classification and also considering soil liquefaction.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SUSTAINABLE DEVELOPMENT IN DRIFT CONTROL OF TALL BUILDINGS DUE TO WIND LOAD: CRITICAL ANALYSES OF THE STRUCTURAL SYSTEMS.

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ABSTRACT

The concept of sustainability should be included in consideration of structural systems during the design and construction of tall buildings (more than 10 storied). The provision of adequate lateral stiffness is also major consideration during selecting proper structural systems for tall buildings. A tall flexible structure subjected to lateral deflection due to the action of fluctuating wind resulting oscillatory movements may induce mild discomfort to acute nausea to the occupants. In this regard, this paper provides results from critical analyses of the structural systems to control the drift (lateral deflection) of tall buildings using STAAD.Pro 2006. Framed system, shear wall-frame system, braced-frame system, tubular system, outrigger system and also hybrid structural systems are analyzed in this research. Braced system and shear wall system are efficient in reduction of drift. Hybrid shear wall-bracing system and large-scale diagonal bracing system are found to be most effective in controlling the drift of tall buildings.

Keywords: *Sustainable development, tall buildings, structural systems, wind loads and drift control.*

1. INTRODUCTION

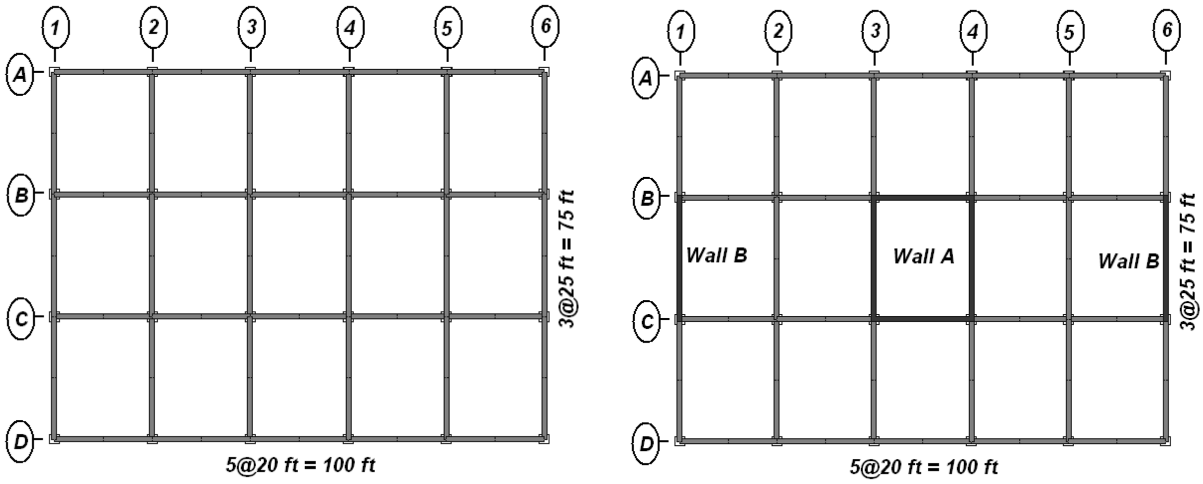
Sustainable development of drift control refers to maintain the lateral deflection of tall building at sufficiently low level to allow the proper functioning of nonstructural components (elevators, escalators etc.) and to avoid distress in the structure, to prevent excessive cracking due to deflection and consequent loss of stiffness. The adopted structural system should be sufficiently stiff to prevent dynamic motions (due to wind and seismic loading) becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken, affect sensitive equipment and to avoid any redistribution of load to non-loadbearing portions or infills. One simple parameter to estimate the lateral stiffness of a building is the drift index, defined as the ratio of maximum deflection at the top of the building to the height due to lateral forces (Smith & Coull, 1991). As per ASCE 7-05 (2005), the limit for drift index is below 0.0025. For conventional structures the preferred acceptable range is 0.0015 to 0.003 and sufficient stiffness must be provided to ensure that the top deflection does not exceed the value under extreme loading condition (Smith & Coull 1991, Islam et. al. 2011). The shear walls are often parts of the elevator and service cores, while the frames are arranged in plan, and they are linked by floor slabs such that the building will deflect as a structure with rigid section. When a wall-frame structure is loaded laterally, the wall deflects in a flexural mode with concavity downward and a maximum slope at the top, and the frame deflects in a shear mode with concavity upward and a maximum slope at the base. Accordingly, the deflected shape of the whole structure has a flexural profile in the lower part and a shear profile in the upper part. The interacting forces cause the wall to restrain the frame near the base and the frames to support the wall at the top, and thus reduce the lateral drift of the structure. The major advantages of a wall-frame structure depend on the amount of horizontal interaction, which is governed by the relative stiffness of the walls and frames, and the height of the structure. The key idea in limiting the wind drift in a tall building is by changing the structural form of the building into something more rigid and stable to confine the deformation and increase stability. The stiffness (rigidity) and stability requirements become more important as the height of the structure increases, and they are often the dominant factors in the design (El-Leithy et. al. 2011).

An important problem associated with wind induced motion of buildings is concerned with human response to vibration and perception of motion. At this point it will suffice to note that humans are surprisingly sensitive to vibration to the extent that motions may feel uncomfortable even if they correspond to relatively low levels of stress and strain. Therefore, for most tall buildings serviceability considerations govern the design and not strength issues (Mendis et. al. 2007).

The major factors that has to be taken into account in selecting the appropriate structural system includes the internal planning, the material and method of construction, the external architectural treatment, the planned location and routing of the service systems, the nature and magnitude of horizontal loading and the height and proportions of the building. Common structural systems are framed system, braced-frame system, rigid-frame system, infilled-frame system, flat-plate and flat-slab system, shear wall and coupled-shear wall system, shear wall-frame system, framed-tube (tubular) system, tube-in-tube and bundled-tube system, braced-tube system, outrigger-braced system, suspended structure system, core structure system, space structure system, hybrid structure system etc. In this paper, the structural systems are critically analyzed to identify the sustainable option to control the drift of tall buildings.

2. EXPERIMENTAL PLANS AND NUMERICAL MODELING

Six major structural systems are selected to investigate the drift pattern due to lateral loading; they are framed system, braced-frame system, tubular system, shear wall-frame system, outrigger-braced system and hybrid system. A 20 storied frame system is considered as a prototype model for the analysis. The plan of framed system and shear wall-frame system are shown in Figure 1(a) and (b). Table 1 provides the short description of the structural system analyzed in this research work.



(a) (b)

Figure 1: (a) Plan of the framed system tall building, (b) plan of the shear wall-frame system tall building.

The tall buildings of different structural system are modeled considering 20 stories with story height 12.5ft using STAAD.Pro 2006, which is a structural analysis software. The thickness of the shear walls are considered 12in. The sizes of the beams are 12in x 20in and columns are 24in x 24in. The thickness of the shear walls are considered 12in, the cross sections of all the bracings are considered 12in x 12in and cross section of the outrigger beam is considered 12in x 48in. The wind load is applied as per BNBC (1993) considering exposure condition A and wind velocity 210 km/hr. Thus the lateral deflection due earthquake is much less compared to the wind load so wind load is considered as the governing lateral load for drift analysis in this research. Fixed support is applied at the base of the structures.

Table 1: Description of the structural systems

Type of structural system	I.D.	Drift index	Description	Figure
Framed system	F1	0.0103	Rigid frame system	2(a)
	F2	0.0029	Rigid frame system (beam size 2x)	
	F3	0.0069	Rigid frame system (column size 2x)	
Braced-frame system	B1	0.0024	Large-scale diagonal bracing (only at Ext. frame)	2(b)
	B2	0.0014	Large-scale diagonal bracing (Ext. & Int. core frame)	
	B3	0.0034	Diagonal bracing	2(c)
	B4	0.0029	K bracing	2(d)
	B5	0.0024	Double K bracing	2(e)
Tubular system	T1	0.0059	Tube-frame system	2(f)
	T2	0.0052	Tube-in-tube system	
	T3	0.0039	Tube-in-tube system (column size 1.5x)	
Shear wall-frame system	S1	0.0029	Shear wall-frame system (1 core and 2 plain)	2(g)
	S2	0.0019	Shear wall-frame system (beam size 1.5x)	
	S3	0.0027	Shear wall-frame system (column size 1.5x)	
Outrigger-braced system	O1	0.0033	Outrigger truss with braced core	2(h)
	O2	0.0062	Outrigger beam with column core	2(i)
Hybrid system	H1	0.0009	Shear wall-bracing (diagonal) system	2(j)
	H2	0.0027	Shear wall-outrigger beam with shear wall core	2(k)

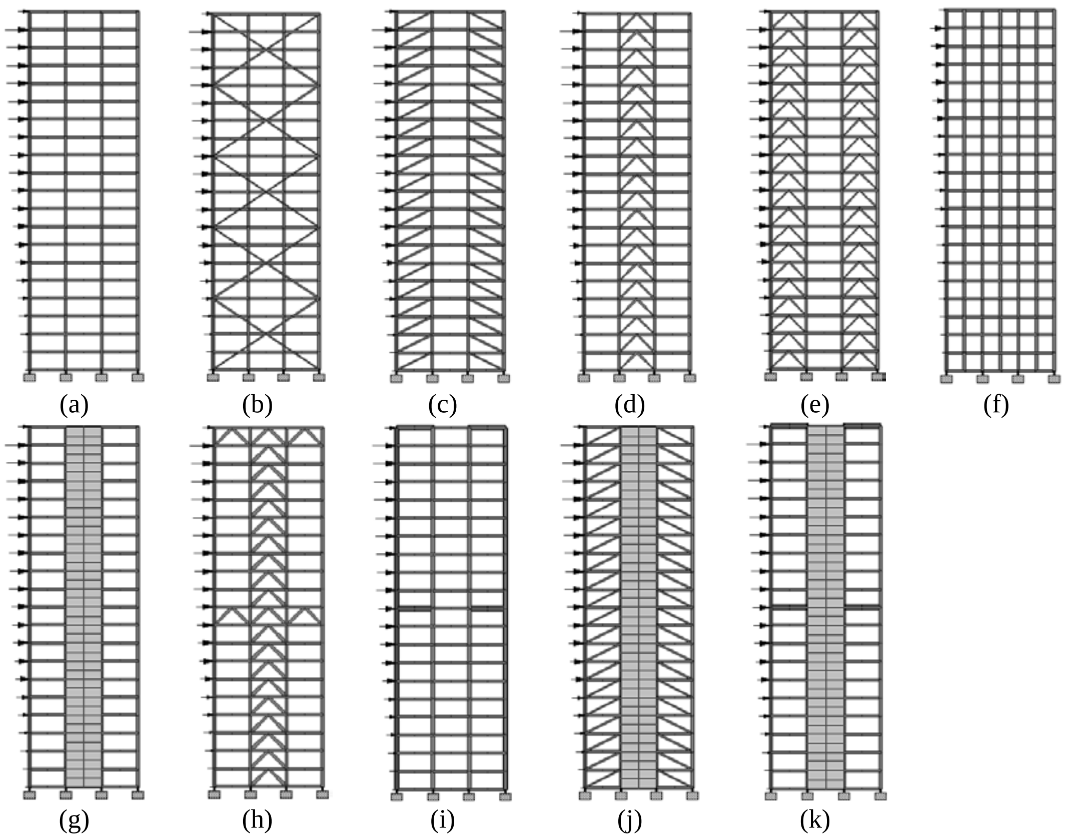
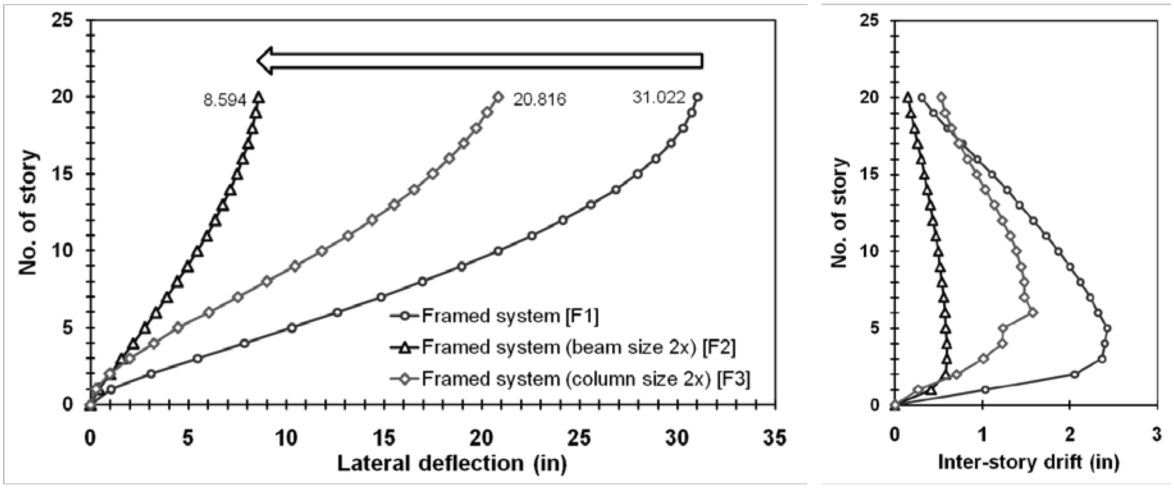


Figure 2: (a)-(k) Structural systems as in Table 1.

3. NUMERICAL RESULTS AND DISCUSSION

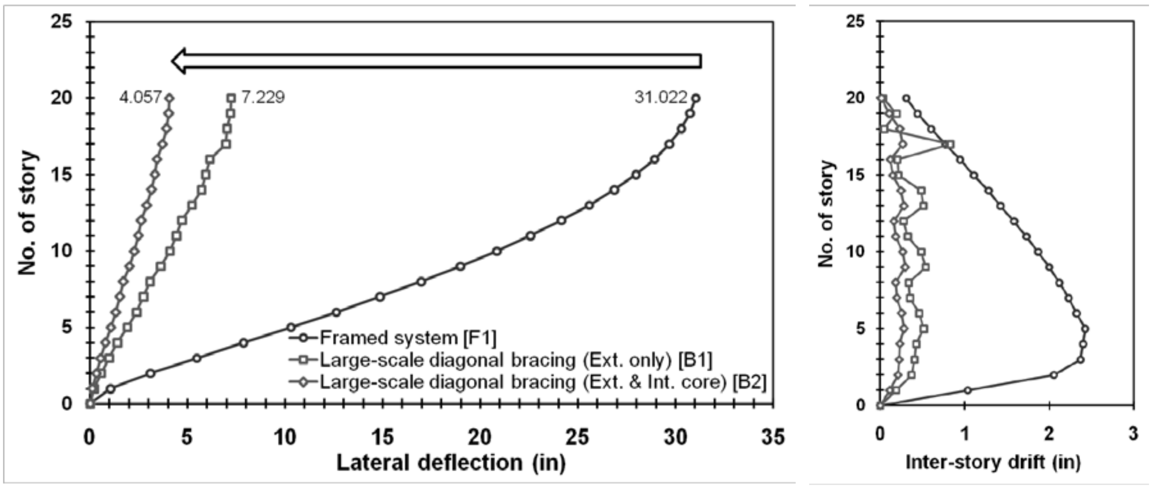
Critical numerical analyses are done to investigate the efficiency of different structural systems for tall buildings in reducing the drift (lateral deflection) due to lateral loading. Figure 3(a) shows the effects of framed system. Drift can be significantly reduced by increasing the moment of inertia (i.e. size) of the beams & columns, and it is interesting to note that the reduction of drift is more with increasing the moment of inertia of beams compared to columns which is also described in Islam et. al. 2011. Figure 3(b) shows that the inter-story drift is higher from 3rd story to 10th story. Increase the size of beams reduced the inter-story drift at 3rd floor as a result the drift is lowered.



(a) (b)

Figure 3: Effect of framed system (a) lateral deflection (b) inter-story drift.

In case of large-scale bracing in Figure 4(a), the reduction of drift is extensively higher compared framed system, even the bracing at the external frame reduced the drift of the whole structure most efficiently. The inter-story drift in Figure 4(b) shows that the reduction of drift started from the 2nd floor and lower at the connection floors of the bracings. For diagonal bracing, K bracing and double K bracing in Figure 5(a) & (b), the reduction is quite similar to large-scale bracing at exterior frame.

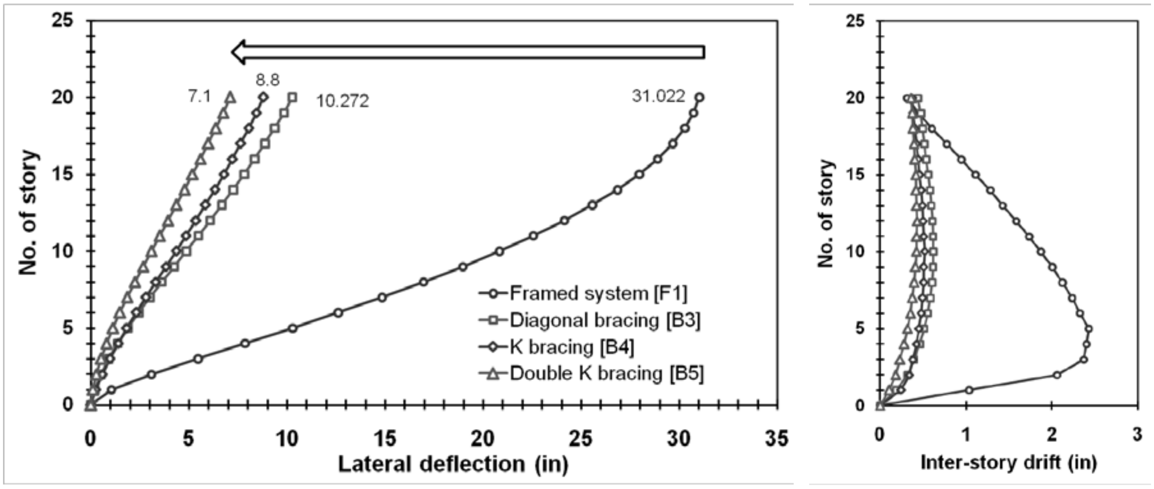


(a) (b)

Figure 4: Effect of large-scale bracing system (a) lateral deflection (b) inter-story drift.

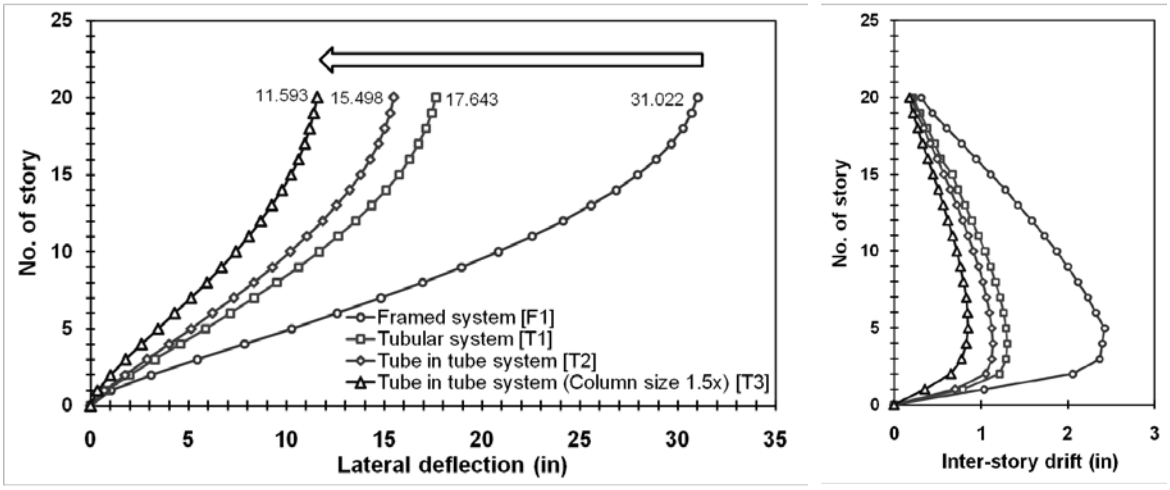
According to tubular system in Figure 6 (a), the reduction of drift is significant but not so satisfactory compared to large-scale bracing system. Figure 6(b) shows that in tubular and tube-in-tube system if unable to reduce the inter-story drift at low story level that results less reduction of drift.

Shear wall-frame system is found to be capable of reducing the drift in significant way (Figure 7a) as described in Islam et. al. 2011. Figure 7 (b) shows that the inter-story drift is considerably reduced by the shear walls at the lower stories (from 2nd story). The increase the moment of inertia of column is less effective compared to beam in shear wall-frame system to reduce the drift. Figure 8(a) shows the effect of outrigger-braced system. Two types of outrigger-braced system are analyzed here. The outrigger truss and braced must/core system showed considerable reduction of drift compared to outrigger beam and column core system. The inter-story drift is lowered more efficiently by the outrigger truss and braced core system (Figure 8b).



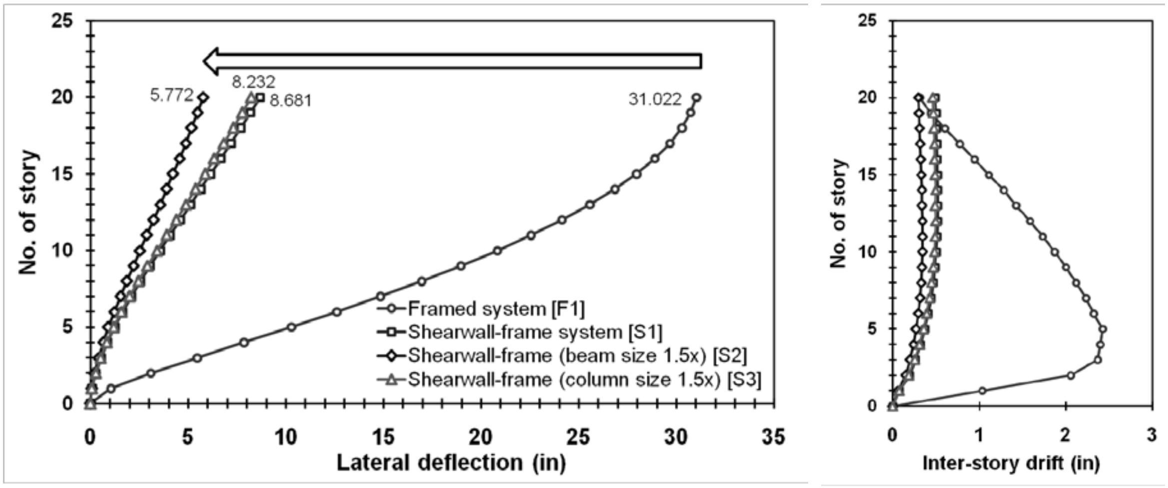
(a) (b)

Figure 5: Effect of braced-frame system (a) lateral deflection (b) inter-story drift.



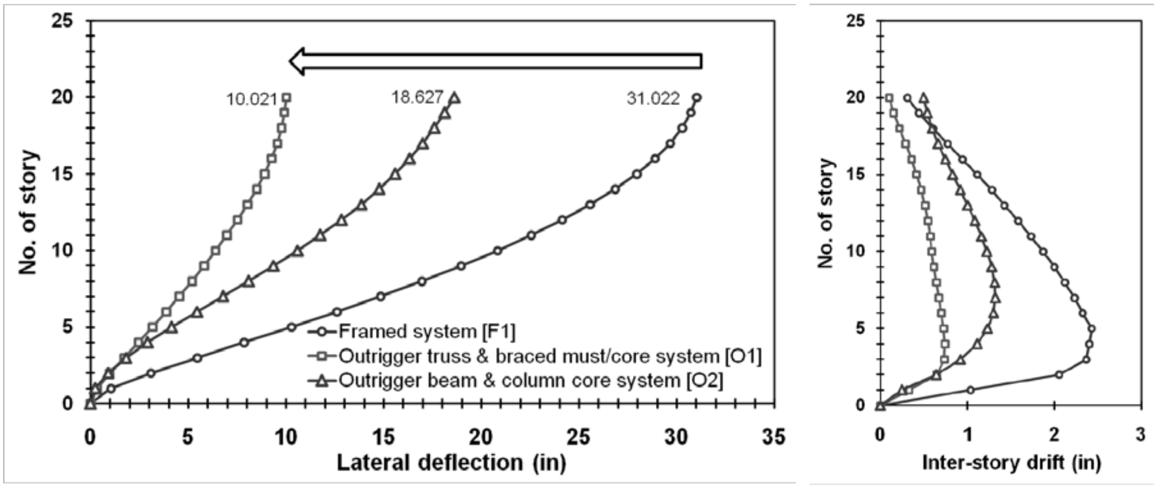
(a) (b)

Figure 6: Effect of tubular system (a) lateral deflection (b) inter-story drift.



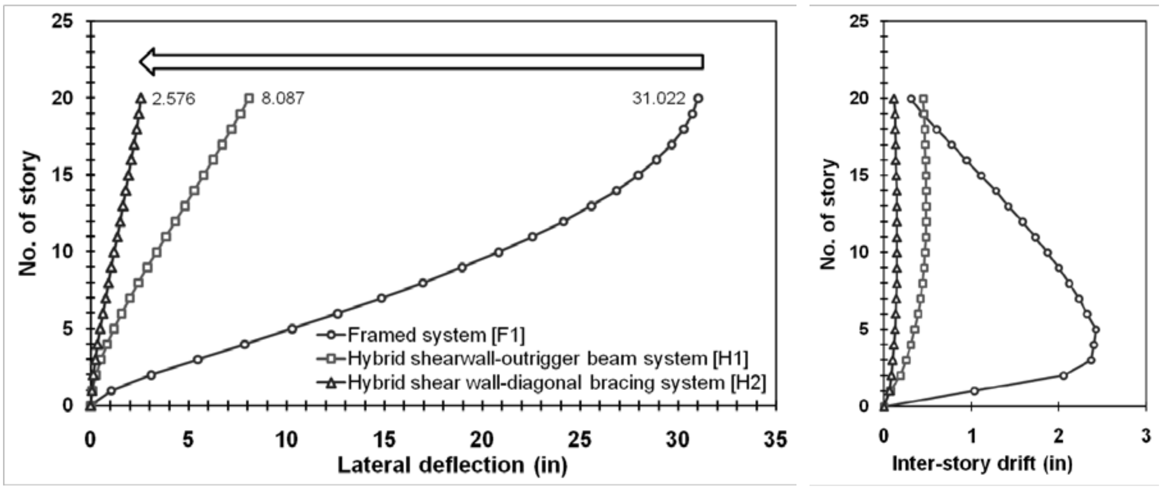
(a) (b)

Figure 7: Effect of shear wall-frame system (a) lateral deflection (b) inter-story drift.



(a) (b)

Figure 8: Effect of outrigger-braced system (a) lateral deflection (b) inter-story drift.



(a) (b)

Figure 9: Effect of hybrid system (a) lateral deflection (b) inter-story drift.

Hybrid shear wall-diagonal bracing system is found to be the most efficient structural system in reducing the drift of the tall buildings (Figure 9a). The efficiency of hybrid shear wall-outrigger beam system is more or less similar to shear wall-frame system. The inter-story drift of the hybrid shear wall-diagonal bracing system is also low from 2nd story which results in the lesser lateral deflection (Figure 9b). The drift indexes are calculated for all the structural systems and are shown in Figure 10. It is clearly visible that the drift index is the minimum for hybrid shear wall-diagonal bracing system (H1) and is followed by large-scale diagonal bracing system (B2) and both lie below 0.0015. With these two systems, B1, B5 and S2 also lie below the limit of ASCE 7-05 (2005) i.e. drift index below 0.0025. Again F2, B4, S1, S3 and H2 structural systems are found to be lower drift index below 0.003 as for the requirement for conventional tall buildings. Tubular system and outrigger system are not found effective in reduction of the drift of tall buildings for above considering specifications. Following expression shows the comparative situation of the structural systems with respect to drift index:

H1 < B2 < S2 < B5 < B1 < H2, S3 < F2, S1, B4 < O1 < B3 < T3 < T2 < T1 < O2 < F3 < F1

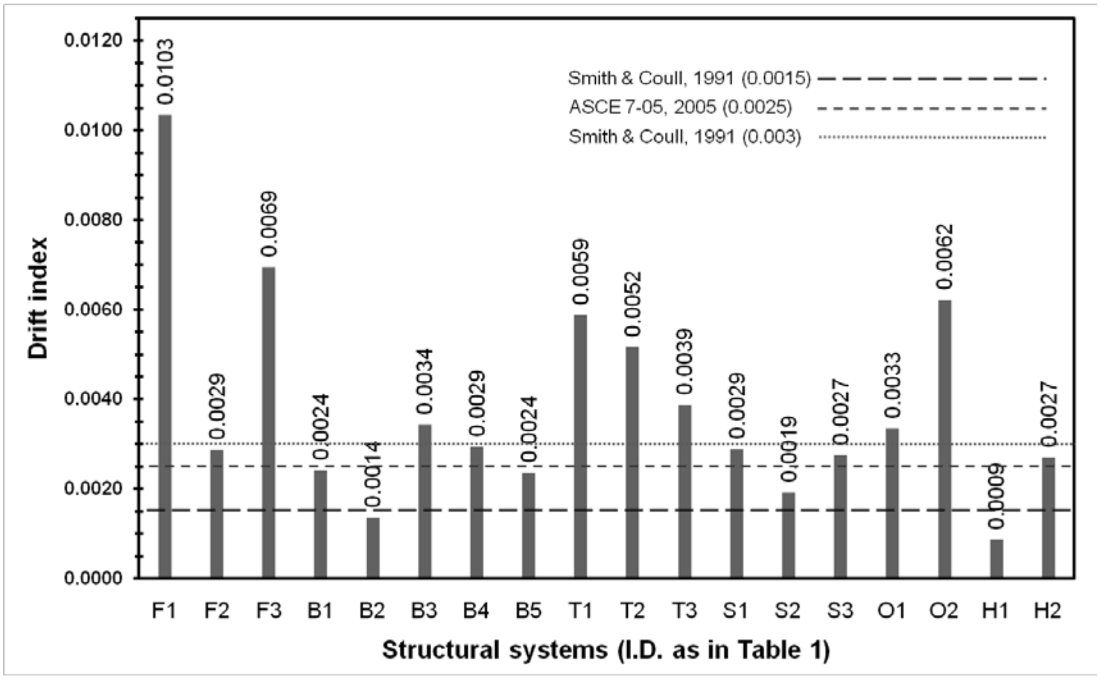


Figure 10: Drift index of the structural systems.

4. CONCLUSION

Considering all the above structural systems, the hybrid shear wall-bracing system (H1) is found to be the most effective structural system for drift control of tall buildings and this system is followed by large-scale diagonal bracing system (B2). Shear wall-frame system with increased moment of inertia (1.5x) of beam (S2) is found to be efficient next to H1 & B2 and is followed by double K bracing system (B5) & large-scale diagonal bracing system at only exterior panel (B1). The performances of hybrid shear wall-outrigger beam system (H2) and shear wall-frame system with increased moment of inertia (1.5x) of column (S2) are quite similar though their inter-story drift pattern is different. It is interesting to note that, in reduction of the drift of tall building, when wind load is governing parameter over earthquake load, the increase of moment of inertia of beam is found to be more efficient instead of column for both the cases of framed system (i.e. F2) and shear wall-frame system (i.e. S2). Tubular system and outrigger braced system are not found much efficient in drift reduction as per the structural considerations.

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COMPARISON OF CREEP BEHAVIOUR OF CONCRETE MADE BY BRICK CHIP & STONE CHIPS IN BANGLADESH

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ABSTRACT

In Bangladesh, Brick Chips are extensively used in concrete but Stone Chips are used to make Bridge, Flat Plate structure and some major components of building structure. As creep is influenced due to many factors including the constituents' materials, relative humidity and temperature, it is essential to examine the effect of the characteristic of coarse aggregate (Brick Chip and Stone Chip) on creep behavior of concrete in Bangladeshi environment. In Bangladesh, research done on creep's effect in both stone & brick chips is very low. The main objective of this research was to find out the effect of creep on both type of coarse aggregates & compare their relative differences. In this work, investigation on the behavior of creep in concrete of different grade (i.e. 2500 psi & 4000 psi) made of brick chips & stone chips under 3(Three) stage loading in a creep test rig with coil spring loading system for a total of 48 days was done. With these investigations, a comparison was done regarding the effect of creep between concrete made of two different coarse aggregate in Bangladeshi Environment. From these comparisons, a percentage difference in creep due to the property of coarse aggregate in concrete for Bangladeshi environment is established.

Keywords: *Bangladesh, Brick Chip, Coarse Aggregate, Creep, Stone Chip.*

1. INTRODUCTION

Concrete is a commonly used construction material for Bangladesh construction industry since centuries ago. One of the important behaviour of concrete is it undergoes volumetric changes throughout its service life. These changes are a result of creep and shrinkage, which are time-dependent deformation of concrete. According to Vincent, Edward C., (2003), creep is defined as the time-dependent deformation resulting from a sustained load. Creep without moisture loss is referred as basic creep whereas with moisture loss is referred as drying creep. Therefore, total creep strain comprises of basic creep and drying creep. An example is 53 storeys Chifley tower, constructed in 1992 Sydney, Australia. This building exploits the composite construction along with the use of belt truss and outrigger system for deflection control. It has central steel braced frame core and outriggers placed at two levels along the building height. In practice, local engineers predict the creep strains by using standard codes or other available prediction models which were developed in temperate country. These prediction models consist of Eurocode 2 (EC2), ACI 209 model (developed by American Concrete Institute), CEB-FIP 90 model (developed by Euro-International Concrete Committee and International Federation for Prestressing), B3 model (developed by Z. P. Bazant and S. Baweja), GL2000 model (developed by N. J. Gardner and M. J. Lockman). These prediction models were derived by empirical approach, in which time functions were determined by curve fitting of test results. It is well documented that creep are influenced by various factors such as constituent materials, temperature and relative humidity of environment. According to S. Ishtiaq Ahmed & S. Roy (2009), creep for Bangladeshi concrete made with brick chips have different magnitude from the predicted values by those prediction models. However, local experimental works are scarcely available to study the effects of the influencing factors for Bangladeshi concrete made with different coarse aggregate i.e. stone & brick chips. In addition, the degree of difference in creep value in brick chips made concrete and stone chips made concrete is never significantly verified. Hence, it is difficult for local design engineer to predict creep related behaviour such as deflection and prestress loss with confidence Based on the above factors the main objectives of this research is,

1. To investigate creep variation between brick chips made concrete and stone chips made concrete in Bangladeshi environment.
2. To investigate creep variation with various strength of concrete.
3. To investigate creep between different grade concrete made with brick chips & stone chips.

2. METHODOLOGY

To investigate creep behaviour for concrete made with brick chips and stone chips with respect to same compressive strength and with respect to different stages of loading, ¾ inch down grade stone and brick chips was used in casting of concrete cylinder. Several tests have been performed to predict the engineering properties of concrete such as compressive strength test, sieve analysis, absorption test, and moisture content test. Two different grade of concrete for brick chips and stone chips were prepared. The grades are 2500 psi & 4000 psi. During the casting the humidity was 65% and the laboratory temperature was 30^0 C.

2.1 Compressive Strength Test

Cylinder specimens with 6 inch in diameter and 12 inch in height were prepared for compressive strength test. The specimens prepared were the same as specimens prepared for creep test and were tested at the age of 10 days & 28 days. The compressive test was performed in accordance with ASTM C 39 (1993). The specimens were tested using a compression machine and with the rate of loading of 3.0 kN/sec. The load applied continuously and without shock until the specimen fails and the maximum load carrying by the specimen during the test was recorded. The cylinder samples were cure for 10 days after casting. The results strength test are give in the following table.

Table 1: Compressive strength of concrete made with brick chips

Sample	Specimen1 (psi)	Specimen 2 (psi)	Specimen 3 (psi)	Average (psi)	10 days data (psi)	28 days data (psi)
BC-1	1671	1930	1910	1837	1809	2585
BC-3	2943	2454	2969	2789	2747	3924

Table 2: Compressive strength of concrete made with stone chips

Sample	Specimen 1 (psi)	Specimen 2 (psi)	Specimen3 (psi)	Average (psi)	10 days data (psi)	28 days data (psi)
SC-1	1810	2012	1839	1887	1859	2656
SC-2	2437	3275	2971	2894	2851	4073

Due to strength similarities, the creep values of sample BC-1 will be compared to the sample SC-1 and the creep values of sample BC-3 will be compared to the sample SC-2.

2.2 Creep Test

Creep tests were carried out for the period of 48 days in a creep test rig with coil spring loading system. The specimens were placed in the loading frame. The centre point of each plate was determined and the specimens were placed with caution to avoid eccentricity. The actual load applied was monitored using a load cell. The load was measured every time before each strain reading was taken to ensure the correct value of loading was applied. The strain reading was taken immediately before and after loading, two to six hours later, and then daily for 1 week, weekly until the end of one month and monthly until the end of the testing. Direct reading of the strain was obtained by multiplying the reading shown on the DEMEC gauge by a calibration of 0.002 mm.

2.2.1 Three (3) Stage Loading

The load applied on the samples was done in three (3) stages. At first stage, 10000lb load was applied on the samples for the first 10 days. At second stage, 20000lb load was applied on the samples for day 11 to day 20. And at third stage, 30000lb load was applied from day 21 to day 48.

3. ILLUSTRATIONS

3.1 Equations

(A) Total Creep

The total creep strain was obtained by subtracting the instantaneous elastic strain and from the total deformation strain as given in the following equation.

$C\ (t_1, t_o) = (\varepsilon_t\ (t_1) - \varepsilon_{ie}\ (t_o))*M$ ………………….. (1)

Where:
$C\ (t_1, t_o)$ = total creep at time t_1 due to a stress applied at t_o
$\varepsilon_t\ (t_1)$ = total deformation at t_1
$\varepsilon_{ie}\ (t_o)$ = instantaneous elastic strain at time t_o
M = coefficient of DEMEC gauge.

(B) Creep Coefficient

After the creep value obtained from above Equation the creep coefficient was obtained as a ratio of creep to the instantaneous elastic strain at any age.

$\Phi\ (t_1, t_o) = C\ (t_1, t_o) / \varepsilon_{ie}\ (t_o)$ ………………….. (2)

Where:
$\Phi\ (t_1, t_o)$ = creep coefficient at t_1 due to a stress applied at t_0
$C\ (t_1, t_o)$ = total creep at time t_1 due to a stress applied at t_o
$\varepsilon_{ie}\ (t_o)$ = instantaneous elastic strain at time t_o.

3.2 Figures & Graphs

In this section, datas of creep behavior affecting by the different stages of loading with same compressive strength for concrete made with brick chips and stone chips are shown.

Table 3: Creep strain for 10000 lb loads

Load	Time	Creep Strain (micron)			
(lb)	(days)	BC-1A	SC-1	BC-3	SC-2
10000	0	141.95	97.03	122.43	91.66
10000	0.042	140.43	97.38	124.71	92.85
10000	0.083	138.72	98.01	125.24	94.56
10000	0.125	141.23	98.85	126.89	96.33
10000	0.208	142.19	99.75	127.77	99.48
10000	1.083	148.01	100.06	128.53	100.02
10000	2.083	141.54	100.91	129.36	100.60
10000	5.083	143.72	101.63	131.26	101.30
10000	5.938	137.84	102.33	133.56	102.03
10000	6.938	130.57	103.66	134.25	102.99
10000	7.938	136.99	103.97	135.87	103.54

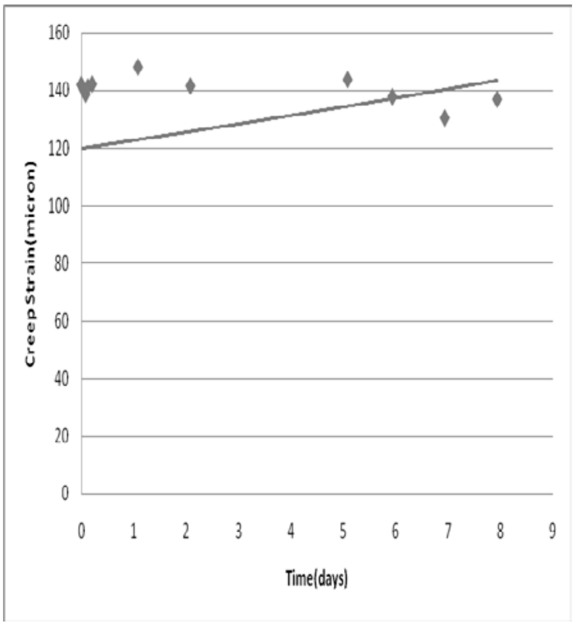

Figure 1: Graph for creep strain vs. time of BC- 1 specimen at 10000 lb loading

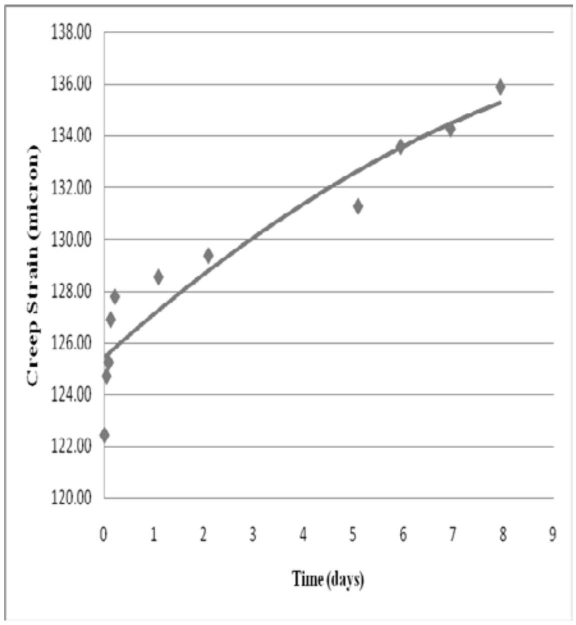

Figure 2: Graph for creep strain vs. time of BC-3 specimen at 10000 lb loading

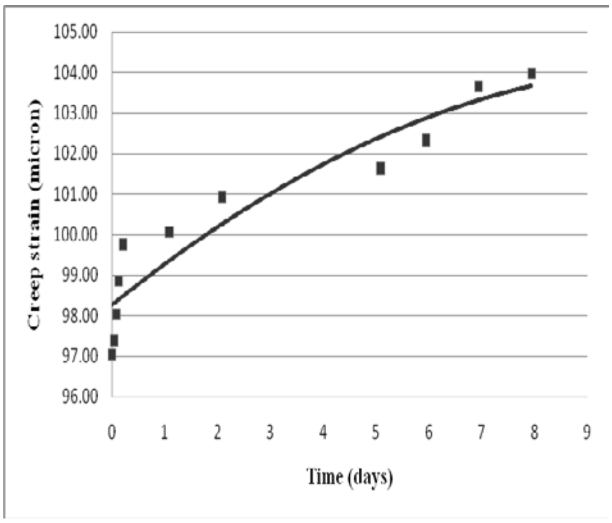

Figure 3: Graph for creep strain vs. time of SC-1 specimen at 10000 lb loading

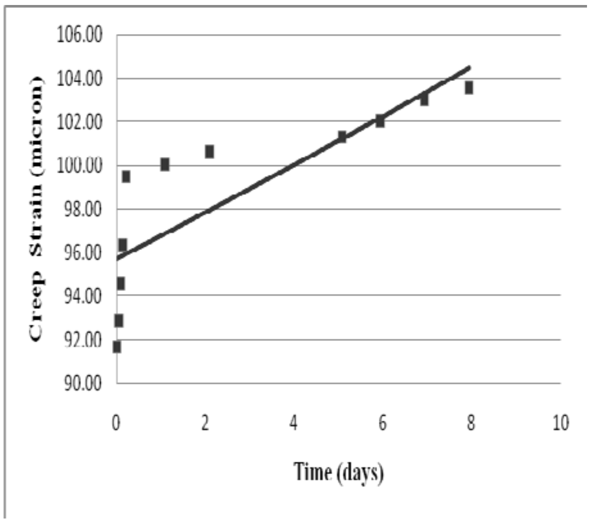

Figure 4: Graph for creep strain vs. time of SC-2 specimen at 10000 lb loading

From Figure 1, Figure 2, Figure 3 and Figure 4, it can be seen that for same applied loading of 10000 lb the creep strain for brick chip concrete is more than stone chip concrete because brick chips are not as stronger as concrete chips. On the other hand for the same material concrete like stone chip concrete and brick chip concrete, creep strain is higher for lower grade concrete made with same materials. The compressive strength of BC-1 and SC-1 are same around 2500 psi and the compressive strength of BC-3 and SC-2 are same around 4000 psi.

Table 4: Creep strain for 20000 lb loads

Load	Time	Creep Strain (micron)			
(Ib)	(days)	BC-1	SC-1	BC-3	SC-2
20000	11.9375	137.14	89.81	113.30	83.33
20000	11.979167	134.93	96.59	113.98	86.30
20000	12.020833	128.95	86.27	115.89	88.56
20000	13.0625	124.61	88.5	116.81	89.63
20000	14.0625	117.57	96.07	117.19	90.23
20000	15.0625	120.33	84.57	119.75	91.23
20000	16.020833	133.1	95.87	121.84	92.45
20000	17.020833	134.44	80.03	124.37	93.74
20000	18.020833	147.36	100.16	126.86	95.14
20000	19.020833	147.31	99.57	128.42	96.08
20000	20.104167	150.04	94.85	130.13	100.23

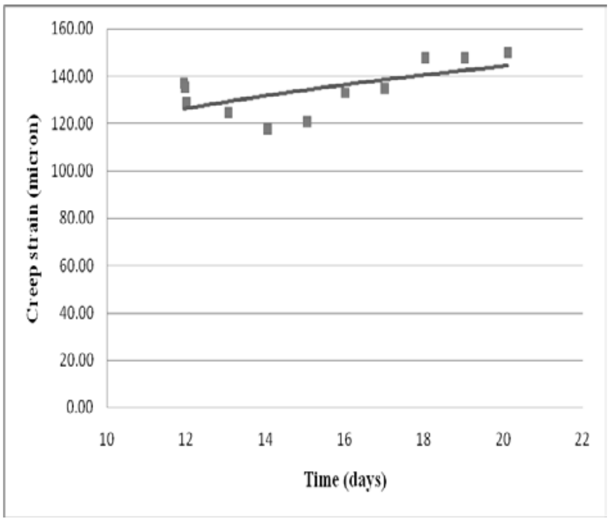

Figure 5: Graph for creep strain vs. time of BC- 1 specimen at 20000 lb loading

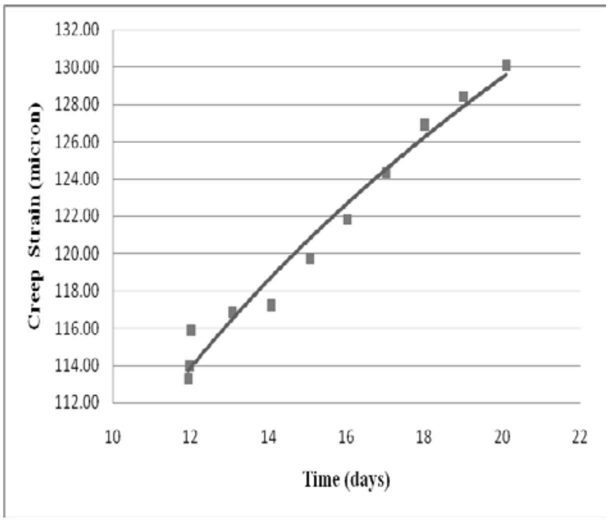

Figure 6: Graph for creep strain vs. time of BC-3 specimen at 20000 lb loading

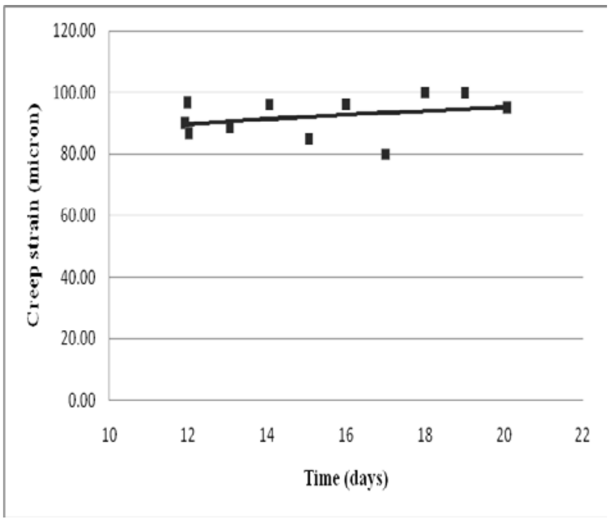

Figure 7: Graph for creep strain vs. time of SC-1 specimen at 20000 lb loading

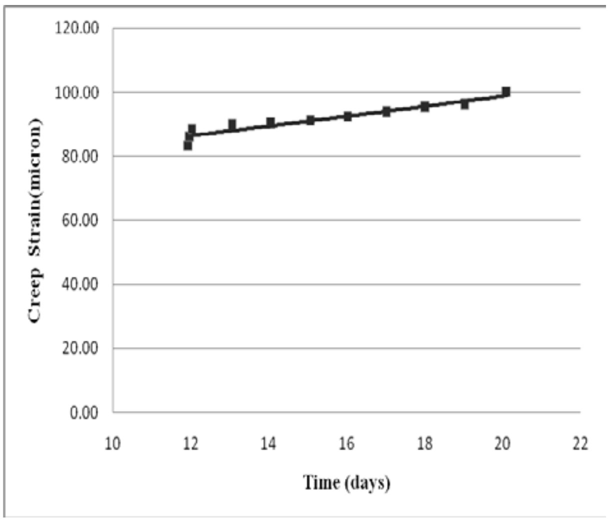

Figure 8: Graph for creep strain vs. time of SC-2 specimen at 20000 lb loading

From Figure 5 and Figure 6 we see that the variation of creep strain for BC-1 specimen is more than the variation of creep strain for BC-3 specimen because the compressive strength of BC-1 specimen is less than the compressive strength of BC-3 specimen. There is little variation in creep strain for concrete made with stone chips shown in Figure 7 and Figure 8. But the increment of creep strain is not linear for both stone chip concrete as well as brick chip concrete with respect to the increment of the amount of loads.

Table 5: Creep strain for 30000 lb loads

Load	Time	Creep Strain (micron)			
(Ib)	(days)	BC-1	SC-1	BC-3	SC-2
30000	20.104	146.77	93.90	138.21	98.15
30000	20.1665	140.19	94.20	138.56	99.51
30000	20.249833	149.54	96.45	138.66	101.09
30000	20.333167	148.53	98.69	137.96	103.33
30000	23.062333	146.36	101.33	134.23	104.56
30000	24.062333	139.45	105.48	134.98	107.00
30000	25.062333	146.19	109.63	144.1	110.23
30000	26.062333	140.46	114.60	136.26	112.44
30000	27.062333	130.33	117.25	132	114.98
30000	34.062333	144.56	125.47	140.16	116.71
30000	41.062333	147.74	129.36	131	117.63
30000	48.062333	134.53	133.25	139.21	118.00

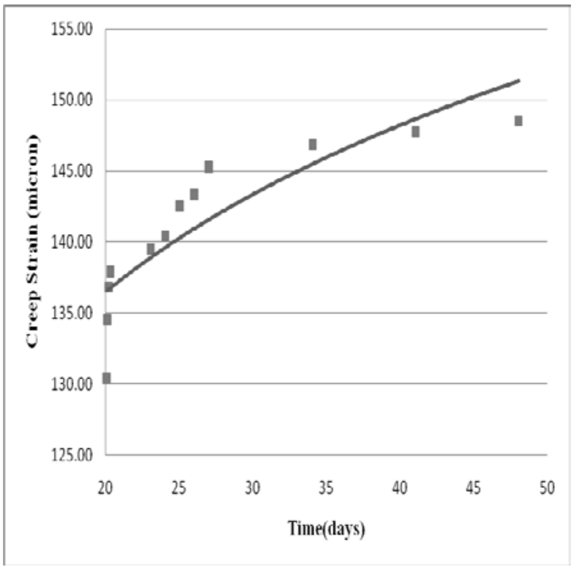

Figure 9: Graph for creep strain vs. time of BC- 1 specimen at 30000 lb loading

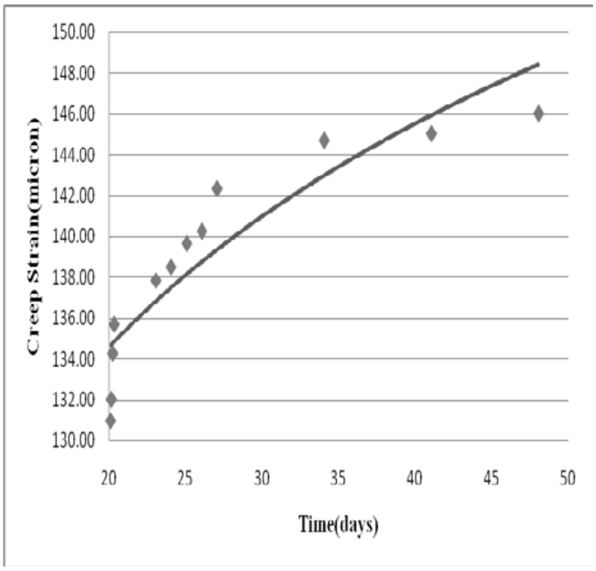

Figure 10: Graph for creep strain vs. time of BC-3 specimen at 30000 lb loading

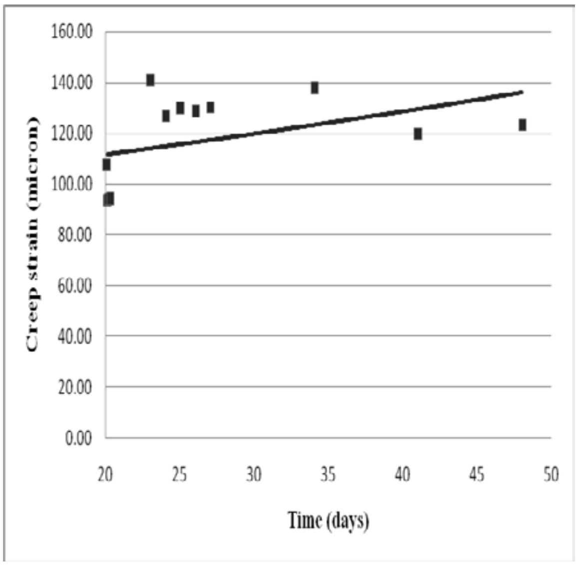

Figure 11: Graph for creep strain vs. time of SC-1 specimen at 30000 lb loading

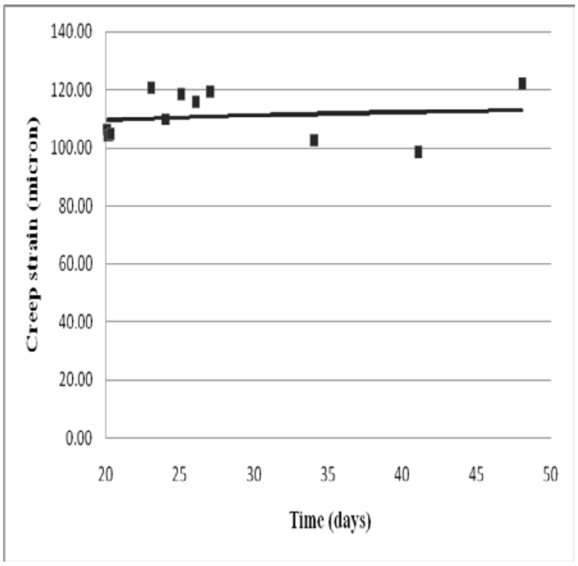

Figure 12: Graph for creep strain vs. time of SC-2 specimen at 30000 lb loading

From Figure 11 and Figure 12 it can be seen that the variation of creep strain for stone chip concrete is not significant. But from the observation of Figure 9 and Figure 10 it can be said that the variation of creep strain for brick chip concrete is significant and the variation pattern of creep strain for brick chip concrete is almost same. From the observation of Figure 11 and Figure 12 it can be also said that creep strain for SC-1 specimen is more than that of SC-3 specimen as the compressive strength of SC-2 specimen is more than that of SC-1 specimen so that the resistance capacity of SC-2 specimen against deformation is more than the resistance capacity of SC-1 specimen against deformation. For SC-2 specimen the variation pattern is almost linear.

3.3 Tables

As per we know that stone chip is stronger than brick chip. So the resistance capacity of stone chip concrete against deformation is more than the resistance capacity of brick chip concrete against deformation. So the creep Strain is less than for stone chip concrete. But it is more for brick chip concrete.

Table 6: Variation of Creep Strain Between BC-1 & SC-1

Time (days)	Creep Strain			
	BC-1	SC-1	Variation From SC-1 (%)	Average Variation (%)
0	141.95	110.85	28.06	
0.042	140.43	97.38	37.66	
0.083	138.72	100.77	41.41	
0.125	141.23	99.87	45.05	
0.208	142.19	98.03	47.10	
1.083	148.01	100.62	40.60	
2.083	141.54	100.67	40.96	
5.083	143.72	101.96	44.05	
5.938	137.84	95.69	44.92	
6.938	130.57	90.10	41.18	
7.938	136.99	97.03	41.18	
11.938	136.99	97.03	52.70	
11.938	137.14	89.81	39.69	
11.979	134.93	96.59	49.47	
12.021	128.95	86.27	40.80	
13.063	124.61	88.50	22.38	
14.063	117.57	96.07	42.28	
15.063	120.33	84.57	38.83	36.58
16.021	133.10	95.87	67.99	
17.021	134.44	80.03	47.12	
18.021	147.36	100.16	47.95	
19.021	147.31	99.57	58.19	
20.104	150.04	94.85	36.33	
20.104	146.77	107.66	49.62	
20.167	140.19	93.70	58.75	
20.250	149.54	94.20	58.18	
20.333	148.53	93.90	4.19	
23.062	146.36	140.48	9.94	
24.062	139.45	126.84	12.77	
25.062	146.19	129.63	9.37	
26.062	140.46	128.43	0.08	
27.062	130.33	130.23	5.11	
34.062	144.56	137.53	5.11	
41.062	147.74	119.75	23.37	
48.062	134.53	123.43	8.99	

Table 7: Variation of Creep Strain Between BC-3 & SC-2

Time (days)	Creep Strain			
	BC-3	SC-2	Variation From SC-2 (%)	Average Variation (%)
0	143.92	91.66	57.02	26.91
0.042	135.84	104.75	29.68	
0.083	142.29	103.27	37.78	
0.125	141.60	103.96	36.21	
0.208	142.79	104.16	37.09	
1.083	131.04	110.09	19.03	
2.083	133.01	108.18	22.95	
5.083	126.89	102.60	23.67	
5.938	122.43	109.69	11.61	
6.938	137.83	101.94	35.21	
7.938	124.71	96.12	29.74	
11.938	124.71	96.12	29.74	
11.938	127.95	110.24	16.06	
11.979	128.50	112.86	13.86	
12.021	123.98	101.33	22.35	
13.063	122.47	111.72	9.62	
14.063	132.84	93.90	41.47	
15.063	115.89	86.33	34.24	
16.021	116.81	86.65	34.81	
17.021	129.25	88.06	46.77	
18.021	113.30	94.63	19.73	
19.021	113.30	95.08	19.16	
20.104	130.13	105.28	23.60	
20.104	138.21	106.41	29.88	
20.167	138.56	104.04	33.18	
20.250	138.66	104.48	32.71	
20.333	137.96	104.93	31.48	
23.062	134.23	120.32	11.56	
24.062	134.98	109.84	22.89	
25.062	144.10	118.70	21.40	
26.062	136.26	115.98	17.49	
27.062	132.00	119.34	10.61	
34.062	140.16	102.49	36.75	
41.062	131.00	98.15	33.47	
48.062	139.21	121.81	8.99	

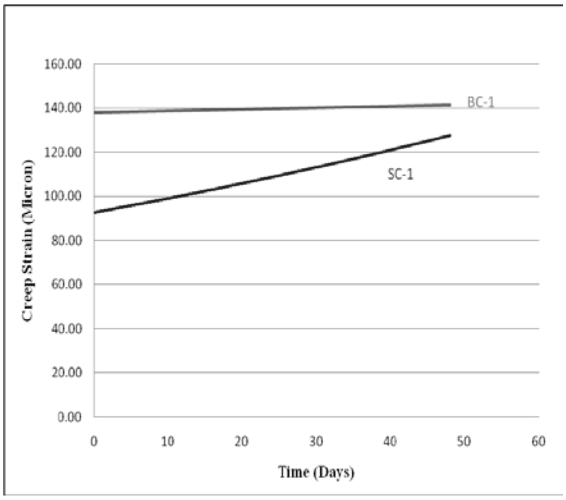

Figure 13: Graph for creep strain vs. time of BC-1 specimen and SC-2 specimen

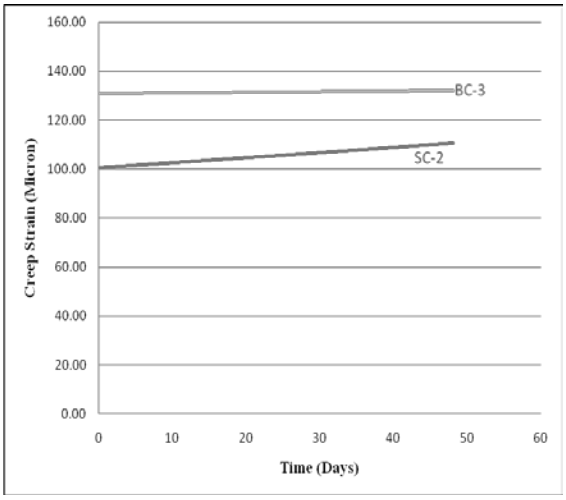

Figure 14: Graph for creep strain vs. time of BC-3 specimen and SC-2 specimen

From Table-6, It can seen that BC-1 has about an average of 36.58% higher creep strain than that of SC-1 though they are of same grade of 2500psi. And from Table-7, it can be seen that BC-3 has about an average of 26.91% higher creep strain than that of SC-2 though they are of same grade of 4000psi. Figure-13 & Figure-14 shows the variations in creep for the concrete samples of Stone aggregate & Brick aggregate.

Table 8: Variation of Creep Strain Between BC-1 & BC-3

Time (days)	Creep Strain			
	BC-1	BC-3	Variation From BC-3 (%)	Average Variation (%)
0	141.95	143.92	1.37	7.57
0.042	140.43	135.84	3.38	
0.083	138.72	142.29	2.51	
0.125	141.23	141.60	0.26	
0.208	142.19	142.79	0.42	
1.083	148.01	131.04	12.95	
2.083	141.54	133.01	6.41	
5.083	143.72	126.89	13.26	
5.938	137.84	122.43	12.59	
6.938	130.57	137.83	5.27	
7.938	136.99	124.71	9.85	
11.938	136.99	124.71	9.85	
11.938	137.14	127.95	7.18	
11.979	134.93	128.50	5.00	
12.021	128.95	123.98	4.01	
13.063	124.61	122.47	1.75	
14.063	117.57	132.84	11.50	
15.063	120.33	115.89	3.83	
16.021	133.10	116.81	13.95	
17.021	134.44	129.25	4.02	
18.021	147.36	113.30	30.06	
19.021	147.31	113.30	30.02	
20.104	150.04	130.13	15.30	
20.104	146.77	138.21	6.19	
20.167	140.19	138.56	1.18	
20.250	149.54	138.66	7.85	
20.333	148.53	137.96	7.66	
23.062	146.36	134.23	9.04	
24.062	139.45	134.98	3.31	
25.062	146.19	144.10	1.45	
26.062	140.46	136.26	3.08	
27.062	130.33	132.00	1.27	
34.062	144.56	140.16	3.14	
41.062	147.74	131.00	12.78	
48.062	134.53	139.21	3.36	

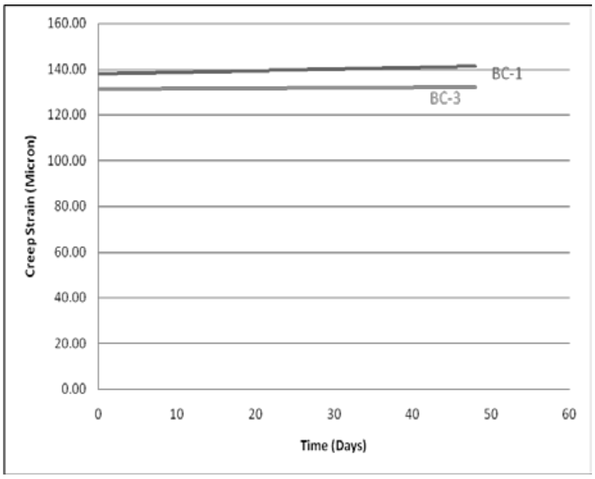

Figure-15: Graph for creep strain vs. time of BC-1 specimen and BC-3 specimen

From Table-8, it can be seen that, BC-1 has an overall average 7.57% higher creep strains than that of BC-3. This due to its lower compressive strength of 2500psi than that of 4000 psi of BC-3. From Figure-15, it can be seen that the variation of creep strain pattern of concrete made with same coarse aggregate but has different compressive strength. The compressive strength of BC-1 specimen and BC-3 specimen are around 2500 psi and 4000 psi respectively. So the creep strain for BC-1 specimen is more than creep strain of BC-3 specimen. This phenomenon is observed from the above graph.

4. CONCLUSIONS

Based on the experiment, the following conclusions can be made:

1. Creep of concrete decreases as the concrete strength increases. However, they are not solely affected by a single factor but a combination of intrinsic and extrinsic factors.
2. It is seen that, 2500psi grade concrete made with brick chips has 7.57% higher creep values than 4000psi grade concrete made with brick chips.
3. Creep of concrete made with brick chips are always higher than the creep of concrete made with stone chips. This is because of the lower strength of brick chips than the stone chips.
4. For 2500psi grade, concrete made with brick chips has 36.58% higher creep values than concrete made with stone chips.
5. For 4000psi grade, concrete made with brick chips has 26.91% higher creep values than concrete made with stone chips.
6. It is seen that deviation of creep values between brick chip made concrete and stone chips made concrete decreases with the increase in strength.
7. Creep is the time-dependent deformation of concrete and the creep test will take a very long time to measure the behavior of creep with sufficient accuracy.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

DAMAGE IDENTIFICATION IN RC COLUMN USING LONG-GAGE FBG SENSORS

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ABSTRACT

Damage identification in structures is an important and active field of research and in last few years, a large number of methods and/or techniques have been proposed for this purpose. Concerning civil structures, some critical issues are presently facing the development of damage identification techniques including: (1) large size of the structure, (2) the damage is usually localized and (3) structural responses are affected by the variation in the environmental parameters as well as the operational conditions. In this paper, modal macro strain ratio based damage identification strategy was applied to identify damage in RC column. Long-gage fiber Bragg grating (FBG) sensors was deployed to capture the response of a column under base excitation. Different loading scenario was applied on the column to investigate the capability of the sensing system as well as the damage identification technique. Modal macro strain (MMS) was extracted for different sensing zone for different load cases and the regression analysis was applied to identify damage . Damage was clearly identified from the observation of the change of the slope of the fit lines.

Keywords: *Damage identification, long-gage sensor, fiber Bragg grating, modal macro strain, strain ratio.*

1. INTRODUCTION

Damage identification in structures is an important and active field of research and in last few years, a large number of methods and/or techniques have been proposed in the field of aeronautics, civil and mechanical engineering for this purpose. The existing approaches proposed in this area can be classified into two major categories-the static identification methods using static test data and the dynamic identification methods using dynamic test data. Compared with the static identification techniques, the dynamic ones have been developed more maturely and the corresponding literature are quite extensive. A detail review on the dynamic identification techniques can be found in Doebling et al. (1998), Carden and Fanning (2004), Sohn et al. (2004) and Montalovao et al. (2006). In most of the vibration based damage identification techniques, structural modal parameters such as modal frequencies and damping ratios, mode shapes etc., are used. One major difficulties with the vibration based damage identification techniques is that the structural properties such as modal frequencies and damping ratios are susceptible to the change of the environmental temperature and damage identification result may be false positive or negative (Serker and Wu, 2009a). Similarly, most of the static test based damage identification techniques utilize static displacement and strain (Hajela and Soerio, 1989; Hjelmstad and Shin,1997;) and the health status of the intact structure is a prerequisite. Therefore, different types of data such as accelerations, velocities, displacements and strains are measured under various excitations for damage identification. Moreover, conventional measurements such as accelerations, velocities and displacements are essentially "point" measurements at translational DOF. Among these measurements, strain may be the most sensitive to local damage and can be a good candidate measurement for detecting a local damage. One of the significant limitations of the point measurement is that it may not reflect a local damage unless the area where the sensor is fixed exactly covers the damaged region. Since the traditional foil strain gages have very small gage length compared to the length of the structure to be monitored, it will be unwise and expensive to use a large number of sensors to cover the whole structure or important parts. Sensors having distributed sensing capability can be a good choice to overcome this limitation in detecting damage or monitoring large civil structures.

In recent days, fiber optic sensing technology has opened the door of distributed sensing with a gage length up to several meters (Li and Wu 2007). Among the fiber optic sensors, fiber Bragg grating based strain sensors are most suitable with its special features of high precision level, stable sensing capacity, reliability and so on. Adewuyi et. al. (2009) showed that the distributedly measured strain responses has better damage identification capability over the point measurement data with the same damage identification algorithm. Li and Wu (2007) developed a long-gage FBG sensor which can be used to measure the structural response distributedly by placing the sensors in series. Serker and Wu (2009b) proposed a damage detection technique which focuses the application of the distributed static strain response. Another problem of the static response based damage identification method is with the application of the static loading which requires involvement of heavy equipment, cost and may need temporary closing of the bridge. This paper focuses on the application of the distributed macro-strain response to detect and localize damage in RC column structures. A modal macro-strain based damage identification technique presented by Serker et al. (2010) was applied to this purpose. With this technique, damage detection can be accomplished with no requirement for an analytical model and/or health condition of the intact or undamaged structure.

2. DISTRIBUTED LONG-GAGE FIBER OPTIC SENSING SYSTEM

Long-gage FBG sensors developed at the Structural Engineering and Dynamics laboratory of Ibaraki University, Japan was deployed as the sensing device for the distributed strain sensing, as shown in Figure 1. In spite of high precision and excellent sensing ability, the ordinary FBG faces an unfavorable problem in that its inherent gauge length is around 1-2 cm, which makes FBG work as a traditional "point" strain gauge and difficult for distributed placement. After special packaging the gage length was extended to 1.0 meter by using a tube to sleeve the optical fiber and then fixing at two ends of the tube. For a general long-gage sensor, the in-tube fiber has the same mechanical behavior and hence the strain transferred from the shift of Bragg center wavelength represents the average strain or the macro-strain over the sensor gauge length. A FBG sensors array for distributed macro-strain measurements can be achieved after connecting the long-gage sensors in series as shown in Figure 1(b). The conventional and commonly used transducers, such as accelerometer velocimeter and displacement transducer essentially provide some kinds of measurements in translational degrees of freedom whereas the long-gage FBG sensors are related with the translational degrees of freedom as described later.

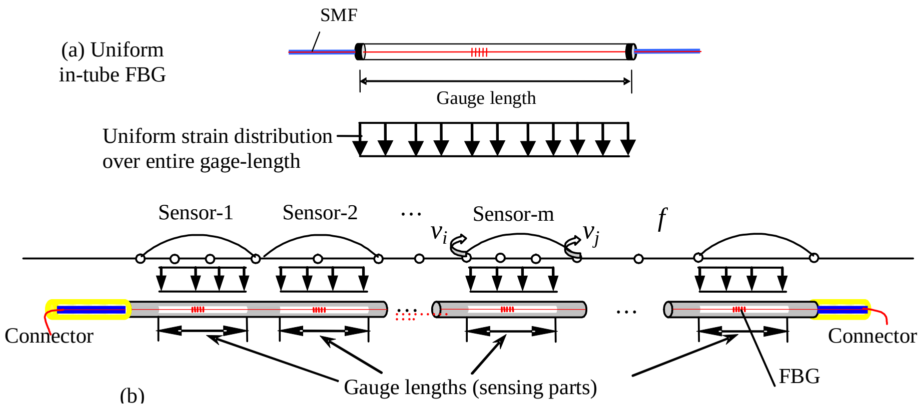


Figure 1: Long-gage FBG sensors array (Li and Wu, 2007)

3. NON-PHYSICS BASED APPROACH FOR DAMAGE IDENTIFICATION

3.1 Damage Detection using the Modal Macro Strain-ratio (MMSR)

For a beam like structure at intact condition and under a certain loading configuration as shown in Figure 2(a), it can be shown that the ratio of the strains between a target and a reference location is constant and can be expressed as the ratio of the distances of the corresponding sections measured from the same reference point as:

$$\frac{\varepsilon_m}{\varepsilon_R} = \frac{x_m}{x_R} \tag{1}$$

Similarly, the strain ratio between a damaged section and the undamaged reference section can be written as

$$\frac{\varepsilon_m^*}{\varepsilon_R} = \frac{1}{\beta}\frac{x_m}{x_R} \tag{2}$$

where ε_m^* is the strain at section x_m after damage, β $(0 \le \beta \le 1)$ is the ratio of the effective flexural rigidity at damaged condition and intact condition. It is apparent from Equations (1) and (2) that the damage can be identified from the change in the strain ratio. In addition, the location of the sensors can be used as a reference for damage identification. The above relationship is valid for a single point time invariant loading. Since every structure has a certain deflected shape under some loading configuration, the above idea can also be applied with a generalized loading. Serker and Wu (2009b) showed the potentials of this approach for damage identification and application for structural health monitoring (SHM) in beam-like structures.

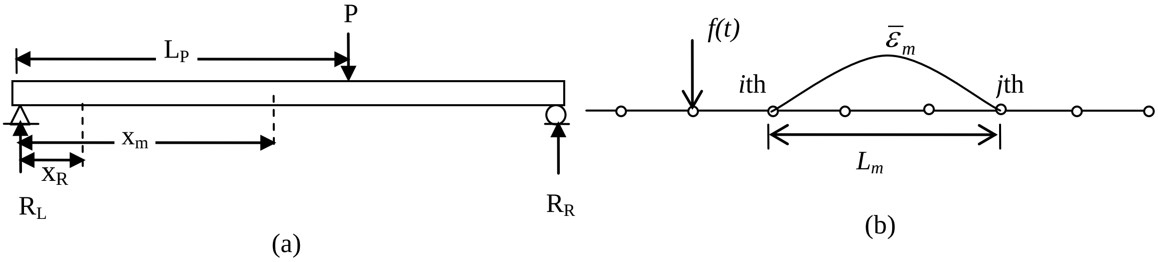


Figure 2. (a) Fundamentals of the strain-ratio approach (b) beam structure model

Although the static test methods have some important features for damage identification, these techniques are not suitable for continuous monitoring of a structure and dynamic measurement is essentially needed. Structure modal properties, such as mode shape, modal frequency etc. are the intrinsic properties and expected to be the same under any condition provided that the structural properties has not been changed. The concept of damage detection presented in the above section can also be applied using the flexural mode shapes. However, mode shape itself is not very sensitive to local damage and mode shape derivatives of different orders have been proposed for damage identification (Pandey et al., 1991). Since strain is very sensitive to local damage, strain mode shapes may also provide excellent damage detection capability.

Strain mode shapes can be obtained from the flexural mode shapes using the well known moment-curvature relationship

$$\varepsilon = \frac{Mh}{EI} = \frac{1}{\rho(x)}h \tag{3}$$

where $\dfrac{1}{\rho(x)}$ is the curvature and h is the distance measured from the neutral axis of the beam to the location where strain will be measured. M and EI have their usual meaning. The curvature of the beam bending can be obtained from the 2nd order differentiation of the displacement mode shape (Pandey *et al.*, 1991) as:

$$\frac{1}{\rho(x)} = \frac{f_{m+1} - 2f_m + f_{m-1}}{l^2} \tag{4}$$

where f_m is the displacement at section m, l is the distance between two measurement points. Combining Equations (3) and (4)

$$\varepsilon = h.\frac{f_{m+1} - 2f_m + f_{m-1}}{l^2} \tag{5}$$

The traditional way of obtaining the displacement mode shape is the use of accelerometers which essentially correspond to the global behavior of the structure. Another problem with the mode shape derivatives is the high sensitivity to noise. Recently, Li and Wu (2008) presented a robust technique to obtain the strain mode shapes using the macro-strain measurements from the long-gage FBG sensors. A summary of the macro-strain modal analysis is presented here.

Consider a beam-like structure installed with a series of long-gage FBG sensors distributed over the length and each sensor covers at least one element (Figure 2(b)). The average strain or the macro-strain, $\bar{\varepsilon}$, over any sensor with a gage length L_m, can be obtained from the rotational displacement with a reasonable assumption that at each element the distance from the inertia axis to the bottom of the beam where sensors are to be installed is the same.

$$\bar{\varepsilon}_m(t) = \frac{h_m}{L_m}[\theta_i(t) - \theta_j(t)] \tag{6}$$

or, in the frequency domain,

$$\bar{\varepsilon}_m(\omega) = \frac{h_m}{L_m}[\theta_i(\omega) - \theta_j(\omega)] \tag{7}$$

where h_m is the distance measured from the neutral axis of the beam to the sensor location. θ_i and θ_j are the rotational displacements of the mth sensor at ith and jth node respectively.

The macro-strain FRF, $H_{mp}^{\bar{\varepsilon}}$, between the measurement from the mth sensor and the excitation at the pth DOF can be achieved by

$$H_{mp}^{\bar{\varepsilon}}(\omega) = \frac{\bar{\varepsilon}_m(\omega)}{P_p(\omega)} \tag{8}$$

Submitting Equation (7) into Equation (8) with $\eta_m = h_m / L_m$

$$H_{mp}^{\bar{\varepsilon}} = \eta_m . \frac{\theta_i(\omega) - \theta_j(\omega)}{P_p(\omega)} = \eta_m \left(H_{ip}^d(\omega) - H_{jp}^d(\omega) \right) \tag{9}$$

where H_{ip}^d and H_{jp}^d are the displacement FRFs at the ith and jth DOF. Equation (9) can be further expressed as:

$$H_{mp}^{\bar{\varepsilon}} = \sum \frac{{}_r A_{mp}^{\bar{\varepsilon}}}{\omega_r^2 - \omega^2 + 2 j \xi_r \omega_r \omega} \tag{10}$$

in which the macro-strain modal constant can be written as

$${}_r A_{mp}^{\bar{\varepsilon}} = \frac{\varphi_{pr}}{M_r} \delta_{mr} \tag{11}$$

and $\delta_{mr} = \eta_m \left(\varphi_{ir} - \varphi_{jr} \right)$ (12)

is the modal macro-strain at mth sensor under the rth mode. By comparing with the traditional displacement FRF, some important conclusions can be drawn about the macro-strain FRF

$$\frac{{}_r H_{lp}^d(\omega)}{{}_r H_{mp}^{\bar{\varepsilon}}(\omega)} = \frac{{}_r A_{lp}^d}{{}_r A_{mp}^d} = \frac{\varphi_{lr}}{\eta_m \left(\varphi_{ir} - \varphi_{jr} \right)} \tag{13}$$

Macro-strain FRF is spatially related with the displacement FRF and not dependent on the load and frequency. Another important feature of the MMS is that there exists excellent correlation among the MMS extracted from various sensors installed on a structure (Li and Wu, 2008). In addition, for a certain condition of the structure and for a particular mode or combination of several modes, MMSR between a target and a reference location will be a constant.

$$\frac{\delta_{ar}^{t_1}}{\delta_{Rr}^{t_1}} = \frac{\delta_{ar}^{t_2}}{\delta_{Rr}^{t_2}} = ... = \frac{\delta_{ar}^{t_m}}{\delta_{Rr}^{t_m}} \tag{14}$$

If more than one mode is used

$$\frac{\sum_{r=1}^{N}\delta_{ar}^{t_1}}{\sum_{r=1}^{N}\delta_{Rr}^{t_1}} = \frac{\sum_{r=1}^{N}\delta_{ar}^{t_2}}{\sum_{r=1}^{N}\delta_{Rr}^{t_2}} = ... = \frac{\sum_{r=1}^{N}\delta_{ar}^{t_m}}{\sum_{r=1}^{N}\delta_{Rr}^{t_m}} \tag{15}$$

where $\delta_{ar}^{t_m}$ and $\delta_{Rr}^{t_m}$ are the MMS obtained from the measurement of macro-strain data at any time t_m at a target and a reference location respectively. N is the number of mode.

3.2 Application of the MMSR for SHM in Changing Environmental Conditions

The above mentioned concept of damage detection will be applied considering that the target structure is instrumented with a series of long-gage distributed sensors. The damage detection technique is presented graphically in Figure 3. In this approach, features (MMS) will be extracted from the measurement over a period of time. Next, the extracted feature of a target location will be plotted against that of a reference location. It is obvious from Equation (1) that all the points of the feature plot will lie on a same line for every condition of the structure and any point lying above the line can be treated as a critical event. However, every structure experiences the varying operational and environmental condition during its life which may cause some variation in the extracted features. It is considered that only the structural damages can cause a significant and permanent change in the local as well as global behavior of the structure. Therefore, for a damaged section the strain ratio line will shift to a new position and continue shifting for a progressive damage. Statistical approaches can be incorporated to account for the variability in the measured responses due to changing environmental condition.

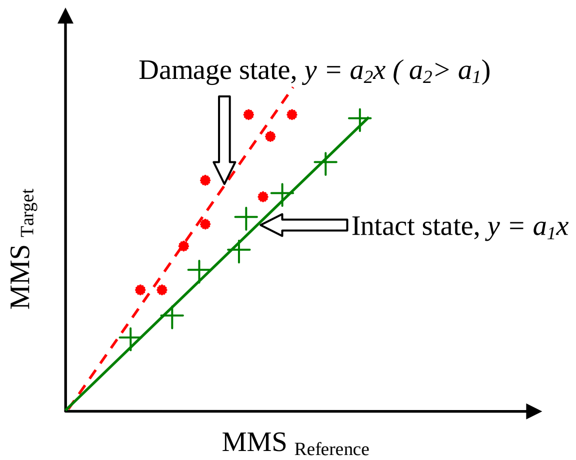


Figure 3. SHM in a changing operational and environmental condition

Let the extracted features of a target zone and the reference zone for a set of measurements follow a linear variation and can be expressed as

$$M_{Target} = a_1 * M_{Reference} \tag{16}$$

where a is the slope of the regression, M_{Target} and $M_{Reference}$ are the extracted features from a target zone and a reference zone respectively. Let the statistical model for another set of measurement for a target zone be

$$M_{Target} = a_2 * M_{Reference} \tag{17}$$

From equation (2) it is obvious that damage causes an increase in the value of the strain ratio. Therefore, damage is likely to be present if the regression line shift to new position with an increased slope i.e. $a_2 > a_1$.

4. DAMAGE IDENTIFICATON IN RC COLUMN

4.1 Experimental Set-up and Data Collection

To investigate the damage identification capability of the proposed technique due to base excitation, for example earthquake excitations, shaking table test on a short column was carried out. A squared section PC column of 1.40m length as shown in Figure 4 was used for the experimentation. Base excitations of different levels with sine and sine sweep waves of different levels were applied. In addition, an earthquake excitation, the 2007 Niigata Chuetsu-Oki earthquake, was applied. Description of the load steps are listed in Table 1. Six FBG sensors were installed to capture the response due to the applied excitations. Details of the sensor installation and the orientation of the column on the shaking tables are shown in Figure 5. The dynamic responses from FBG sensors have been recorded at a sampling rate of 500Hz. FBG sensors are attached on the principal direction of vibration marked as face A as shown in Figure 5. Typical excitations applied to the column are shown in Figure 6.

4.2 Damage Identification

Wave length data recorded during the experiment were processed using a PC. Typical macro-strain responses measured on the column are shown in Figure 7. The first two responses are measured on the intact column and the third response is from the damaged column. At first the reference model for damage identification was constructed by using the response of the column before cracking. Magnitudes of the applied excitations were kept low so that the developed stress remains within the cracking strength of concrete. The first crack was developed by applying the sine sweep excitation with a frequency of 5~25 Hz.

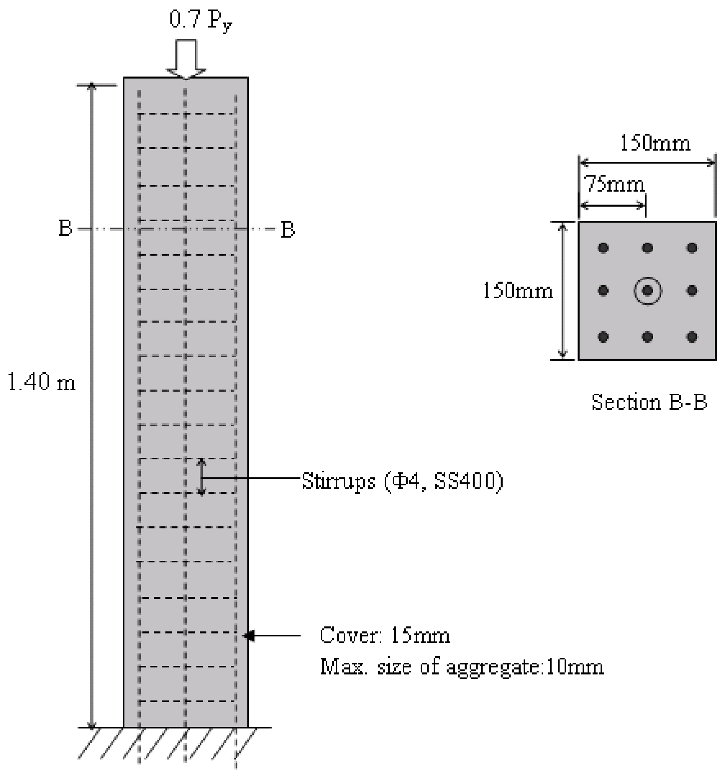
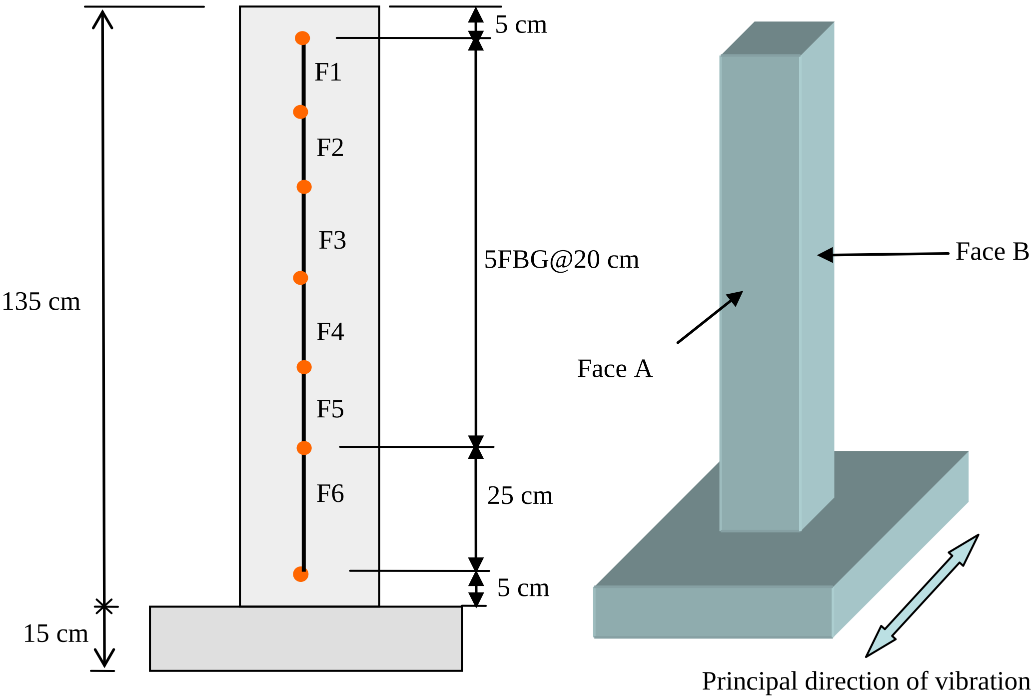


Figure 4: Details of the column specimen

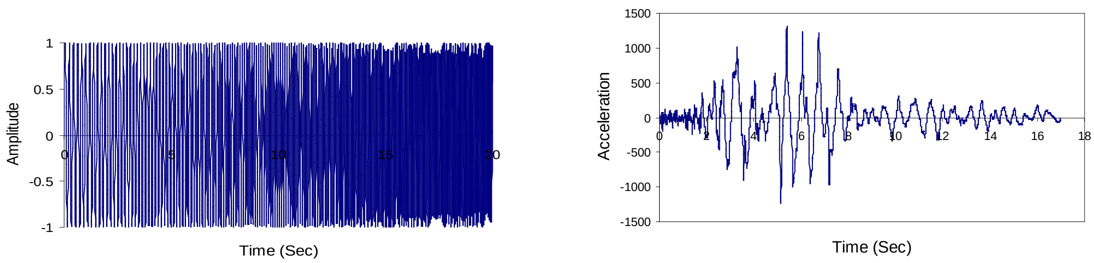


(a) (b)

Figure 5: (a) Sensor configuration (b) Orientation of the shake table

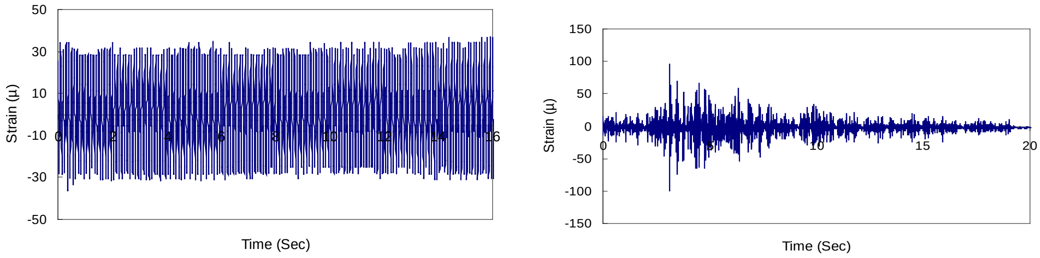
Table 1 List of excitations applied to the column specimen

Load steps	Excitations	
	Before damage	After damage
1	*Sin (2π*5)t*	*Sine* Sweep 5~25 Hz
2	*2Sin (2π*5)t*	*Sine* Sweep 5~20 Hz
3	*3Sin (2π*7)t*	*3Sin (2π*15)t*
4	*3Sin (2π*10)t*	*3Sin (2π*20)t*
5	2007 Niigata Earthquake	*3Sin (2π*20)t*
6	*Sine* Sweep 5~10 Hz	*Sine* Sweep 5~15 Hz
7	*Sine* Sweep 5~20 Hz	

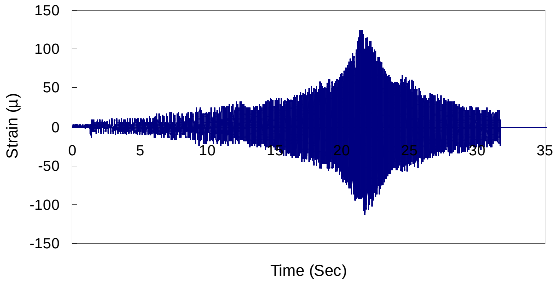


(a) (b)

Figure 6: Typical excitations (a) sine sweep (b) 2007 Niigata Chuetsu-Oki earthquake



(a) (b)

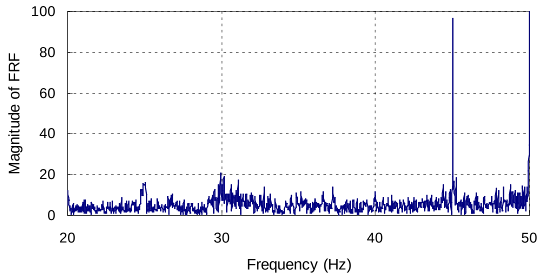


(c)

Figure 7: Typical response data due to (a) sine wave (b) earthquake wave (c) sine sweep wave

Modal macro strain was extracted from the strain responses through the FRF technique. Typical FRFs from the intact and damaged column are shown in Figure 8. The first mode of vibration is used for damage identification. The first mode frequency of the intact column was obtained as 45.01 Hz. The first mode frequency of the cracked column was obtained as 40.04Hz. The column specimen after cracking is shown in Figure 9. The first crack was developed very close to the base of the column and supposed to be captured by FBG6 located in this zone. Unfortunately, performance of FBG6 and FBG1 were not good. Therefore, damage produced in zone

(a)



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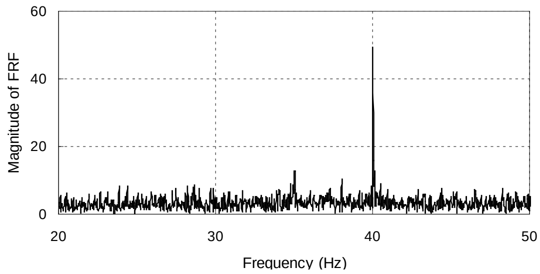


Figure 8: Typical FRFs (a) intact column (b) damaged column

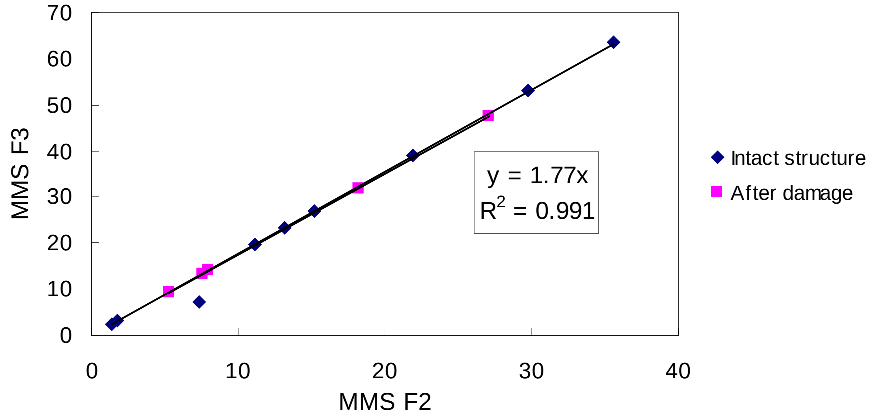
FBG6 can not be detected. FBG2 was selected as the reference sensor for damage identification. Through the visual inspection it was confirmed that the zone of FBG2 was not cracked.

Scatter plot along with the corresponding fit lines are shown in Figure 10. The slope and R^2 values are also shown in Figure 10. R^2 values obtained for all sensing zones are very close to unity. Therefore, the extracted MMS from different sensors are in well agreement. It is obvious from Figure 10 that the fit lines of FBG3 and FBG4 do not have any significant change after the cracking of column. Damage in FBG5 can easily be identified from the change in slope of the corresponding fit line.

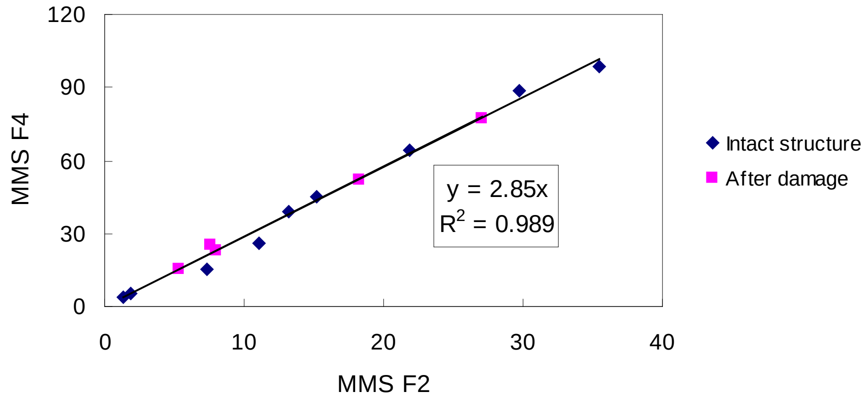
Figure 9: Test column after cracking

5. CONCLUSIONS

In this paper, modal macro strain ratio based damage identification technique was applied with the deployment of long-gage FBG sensors to capture the macro strain response of a RC column under different types of base excitations. From the experimental results it is obvious that the long gage FBG sensors are capable of capturing the response under different excitations. Also, the MMSR technique can detect and localize the damage. It is also important to note that the location of the damage can easily be found from the sensor configuration. In addition, the proposed damage detection technique does not involve any physics based model, therefore, environmental temperature effect on the geometry, boundary conditions etc. may not affect the damage identification process. However, for better damage identification result the reference model should be calibrated using a large number of data.



(a)



(b)

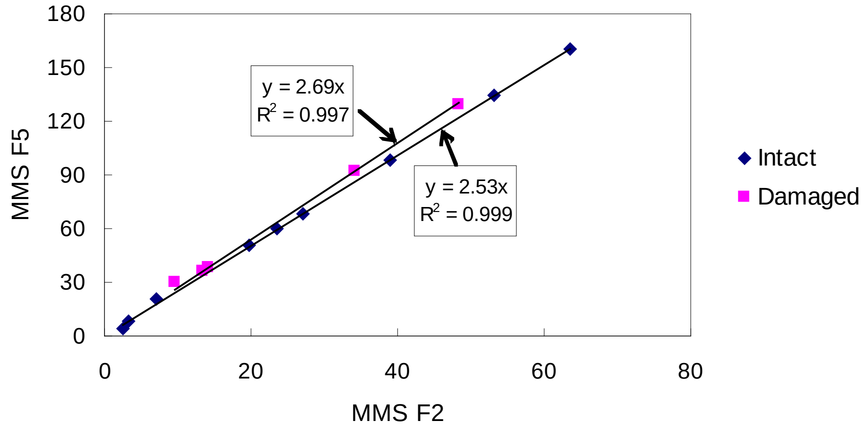


Figure 10: Damage identification results

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STRESS DISTRIBUTION OF CFRP STRENGTHENED STEEL HOLLOW SECTIONS UNDER TENSION

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ABSTRACT

Carbon fibre reinforced polymer (CFRP) sheets have many outstanding properties such as high strength, high elastic modulus, light weight and good durability which are made them a suitable alternative for steel in strengthening work. This paper describe the ultimate load carrying capacity of steel hollow sections at effective bond length in terms of its cross sectional area and the stress distribution within bond region for different layers CFRP. It was found that depending on their size and orientation of uni- directional CFRP layers, the ultimate tensile load was different. Along with these tests, non linear finite element analysis was also performed to validate the ultimate load carrying capacity depending on their cross sections. The predicted ultimate loads from FE analysis are found very close to the laboratory test results. The validated model has been used to determine the stress distribution at bond joint for different orientation of CFRP. This research shows the effect of stress distribution and suitable wrapping layer to be used for the strengthening of steel hollow sections in tension.

Keywords: *CFRP, steel hollow section, wrapping layer, stress distribution*

1. INTRODUCTION

A composite material is one that attains its physical and mechanical characteristics through the integration of other materials. Generally, a composite material combines the most desirable characteristics of its constituents to create a superior material. A well-known example of a composite material is CFRP (carbon fibre reinforced polymer). This advanced composite material provides greater strength at lighter weights than traditional construction materials thus offering distinct advantages in many engineering applications. The use of carbon fibre reinforced polymer in civil infrastructure for repair and strengthening of reinforced concrete structures and also for new construction has become common practice [Teng et al, 2001]. With the introduction of CFRP materials, the possibility for providing a solution to the ongoing problem of infrastructure deterioration may be extended to steel structures as well.

Conventional welding used to repair cracks in steel structures by adding new material to the crack area will typically lead to poor fatigue performance. Welding can also cause metallurgical changes to the parent material, resulting in premature failure. These conservative methods are less effective and could increase maintenance costs [Seica et al, 2007]. Thus there is a need for more efficient and reliable retrofit and restoration methods. CFRP strengthening is attractive for steel bridges and structures since it avoids field welding, particularly overhead welding.

Bond stresses may be much more critical for steel structures than for concrete structures since more strengthening material is needed for steel structures to achieve a similar increase in strength due to the inherent high strength of steel and also since the debonding failure does not occur in the substrate as in concrete structures. Despite these challenges, since many structures built in the post-World War II era are already past their design life, the inventory of deteriorated steel structures and bridges in need of rehabilitation are extremely significant [Hollaway, 1994; Schwartz, 1997; Barbero, 1999; Peters, 1998].

A significant amount of research has been undertaken to determine bond characteristics of CFRP strengthened double strap joints under tension [Fawzia et al, 2004a, 2004b, 2005a, 2005b, 2006a, 2006b, 2007, 2008 and 2010]. But those researches were limited to steel plate double strap joint.

Bambach et al. (2008) studied the axial compressive behaviour of CFRP strengthened cold-formed square hollow sections using experiments. They showed that the application of CFRP delayed local buckling, and that the elastic buckling strength of slender sections is increased by up to 4 times. However they only considered two layouts of CFRP. More tests are required to derive an optimal combination of fibre orientation, number of layers and sequence in applying CFRP layers.

Shaat and Fam (2006) found that transverse CFRP layers are effective in confining the outward local buckling of short columns and that the load capacity increased by 18% for short columns and 13%–23% for long columns.

A thorough understanding of bond characteristics is essential for CFRP strengthen of different types of steel structures to establish a suitable design guideline. A series of laboratory testing were conducted to find out the ultimate load carrying capacity of CFRP strengthened circular hollow sections (CHS) for different orientation of CFRP layers under tensile load. This paper showed the stresses of different layers of CFRP strengthened hollow steel structure under tensile load by using non-linear finite element models. The main findings of the paper show a new way to strengthen steel hollow sections with cost effective CFRP wrapping.

2. MATERIALS PROPERTY

A series of laboratory tests were conducted to investigate the bonding characteristics between CFRP and circular hollow sections of steel. There were three materials used in the bonding process, they were CFRP, adhesive and mild steel circular hollow sections. Normal modulus of CFRP CF130 is a high tensile, unidirectional carbon fibre sheet of nominal strength. According to the manufacturer’s specifications, these carbon fibres have a Young’s Modulus of 240 GPa and a tensile strength of 3800 MPa [BASF, 2006].

The adhesive MBRACE Saturant is a two part epoxy resin. According to the manufacture’s specifications this adhesive has a Young’s Modulus greater than 3000 MPa [BASF,2006]. The resin should be mixed 3:1 ratio of Part A to Part B, and will be applied using a brush in a Wet Lay Up method [BASF, 2006]. Mild steel circular hollow sections 200 mm long, 2.9 mm thick and diameters 48.3mm and 60.3 mm were used in this test program.

The materials, used in the bonding process, should undergo quality assessment by material property testing to find the degree of strengthening. It is common practice to obtain the measured material properties such as tensile strength, modulus of elasticity and ultimate strain rather than relying on the values supplied by the manufacturer. In this case, the material properties of MBRACE Fibre and MBRACE Saturant which was measured by Fawzia[2008] using the coupon test was used. The properties of the three materials used in FE analysis has shown in Table 1.

Table 1: Material properties

Properties	**CFRP**	**Adhesive**	**Steel**
Tensile modulus(Gpa)	230	2.028	195
Tensile strength(MPa)	2675	24.8	484
Yield stress(Mpa)			359
Tensile strain	1.2%	1.46%	1.7%
Poisson’s ration	0.28	0.32	0.25

3. EXPERIMENTAL PROGRAM

Total six specimens having two different diameters of circular hollow sections (CHS) were prepared with normal modulus CFRP. A schematic view of the specimen is shown in Figure 1.

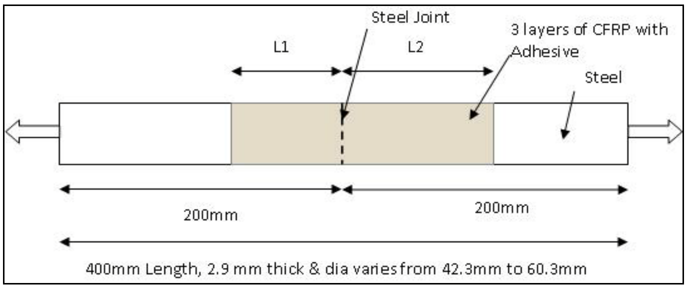

Figure 1: A schematic view of specimen (not to scale).

The surfaces were cleaned with acetone to remove grease, oil and rust. Two CHS were aligned in position before applying adhesives and CFRP. Three layers of CFRP sheets were wrapping around the steel CHS in three different wrapping schemes. First wrapping scheme was all layers longitudinal in direction denoted as 3L, second wrapping scheme was two layers longitudinal and one layer transverse in direction denoted as (2LT) and the rest one was one layer longitudinal and two layers transverse in direction denoted as (L2T) as showed in Figure 2.

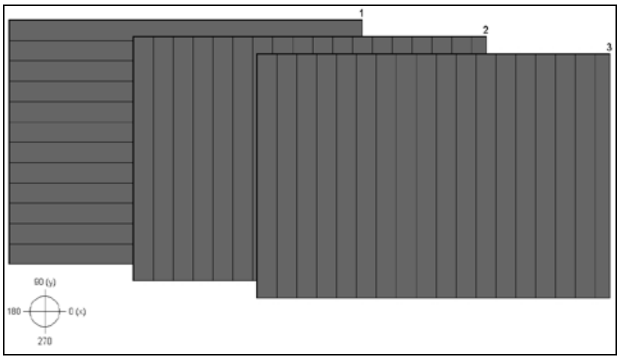

Figure 2: Layout of CFRP sheet for L2T specimen

The specimens were cured for two weeks at room temperature. Each specimen was loaded in tension in a 500 kN capacity Instron testing machine with a loading rate of 2 mm/min. The details of the tests procedure can be found in Fawzia et al. [2004(b)]. The observed failure mode for the normal modulus CFRP was bond failure. Table 2 gives test results of ultimate load carrying capacity for different wrapping scheme at effective bond length.

Table 1: Results of specimen testing

Specimens Label	Ultimate Load (kN)	Failure Mode
C48.33L	60.151	Bond failure
C48.32LT	47.990	Bond failure
C48.3L2T	30.870	Bond failure
C60.33L	70.770	Bond failure
C60.32LT	56.967	Bond failure
C60.3L2T	53.973	Bond failure

4. FINITE ELEMENT ANALYSIS

In this study a finite element model for circular hollow steel sections wrapped with CFRP composites was developed. The model was simulated using Strand7 finite element software. The simulation was done by running nonlinear FE analysis solver to account for the nonlinear properties of the materials.

To create the FE model a cylindrical coordinate system was selected. All materials were modelled as brick elements. First cross sectional areas of all materials were created as plate element by adding nodes and then extruded with different thickness along the longitudinal direction.

Generally the applied strain had been considered for calculating the damage rate. For composite materials, the stress of fibre and matrix were different for their volume and elastic modulus but they were functioned for the same strain. So strain was chosen to explain failure mode. In FE model all different layers of CFRP is created as an individual layer to find the stresses of each layers in different directions. The FE model has validated by experimental results and then carried out the comparative study.

4.1 The boundary conditions and loadings

In the present model, the Z-axis of the coordinate system coincided with the axis of the cylinder. The R and T axis represented the radial and hoop directions of the cylinder respectively. Top end of the steel section was fixed rotation but free in translation, i.e., the three degrees of freedom on that surface were constrained and others were free. Figure 3 was representing the boundary conditions of FE model at longitudinal section.

A uniform displacement was applied at the bottom end nodes in the axial direction. The axial displacement load was increased gradually until the CFRP fails. In FE model the thickness of adhesive layers were considered constant throughout the bond length and it was taken as all layers of adhesive were same i.e. 0.345 mm [Smith, 1973]. Each layer of CFRP has thickness 0.176 mm as given by the manufacturer was used in FE model.

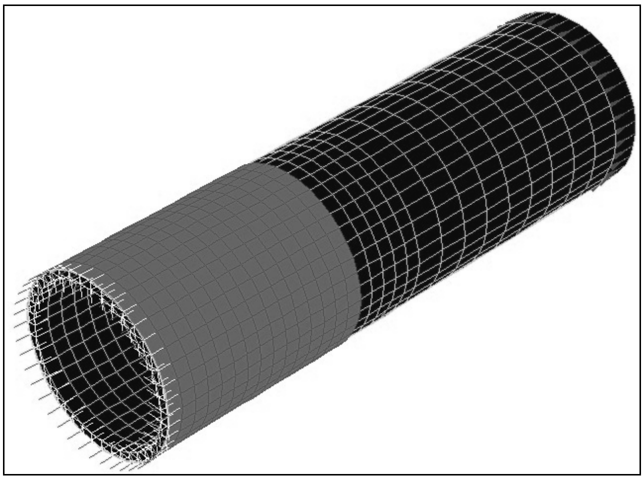

Figure 3: Details of the boundary conditions of the model

4.2 Material Property

The properties including tensile strength, strain and modulus can be found from Table 1. In the FE model, one layers of CFRP and one layers of adhesive were considered as full CFRP layer having an equivalent modulus of one layer was taken equal to 75.78 GPa [Fawzia et al.,2006(b)].

4.3 Failure mode and Ultimate Loads

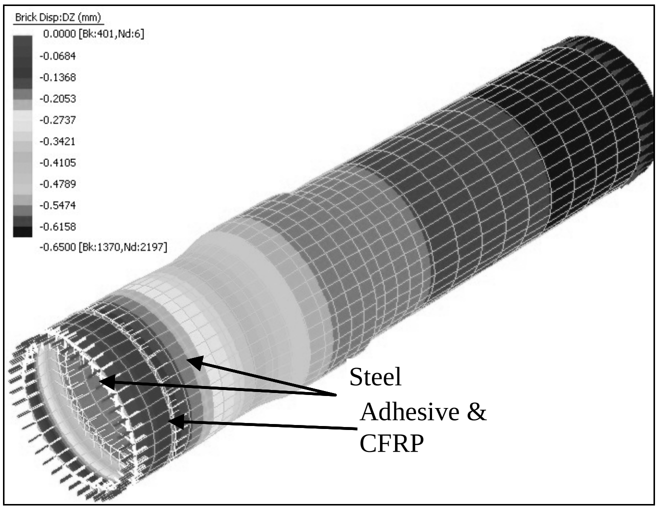

Figure 4: Bond Failure in FE Analysis

Strand7 uses information about the nominal size of the structure to automatically give a reasonable displacement scale, one that clearly shows the deformation. Figure 4 showed bond failure of the CFRP sheets in FE analysis. In case of experiment the failure mode was similar as with the FE. Figure 5 showed a typical experimental bond failure. This was similar to those observed previously from similar tests on normal modulus CFRP and steel tubes [Jiao and Zhao, 2004].

Figure 5: Bond failure of the specimen (experiment)

Figure 6 showed the comparison of the load caring capacity of CFRP strengthened CHS having diameter of 48.4mm and 60.3 mm at different CFRP orientation of experiment and FE analysis. It observed that 3L specimens attained maximum load carrying capacity and L2T specimens carried minimum load. They are differences are significant.

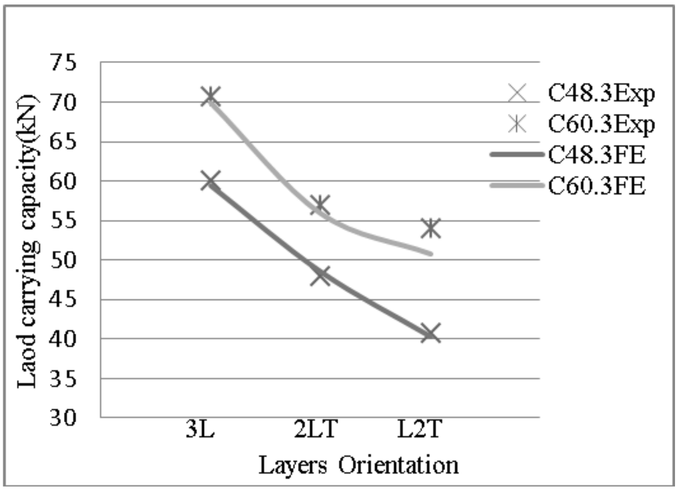

Figure 6: Load carrying capacity of CHS48.3

In Table 3 the comparison of load carrying capacity between experimental and FE results is shown with reasonable accuracy. The mean ratio is 0.9823 with coefficient of variables is 0.0238. In experiment the thickness of adhesive layer was not absolutely constant throughout its length but in FE model it is same for the entire length which made the difference between FE results and experimental results. Therefore there is a reasonably good agreement between the experimental and the proposed FE model results.

Table 2: Results of specimen testing

Specimens Label	Ultimate Load (kN) Experimental(Exp)	Ultimate Load (kN) FE Analysis(FE)	FE/Exp	Failure Mode
C48.33L	60.151	59.450	0.9883	Bond failure
C48.32LT	47.990	48.546	1.0115	Bond failure
C48.3L2T	40.870	40.332	0.9868	Bond failure
C60.33L	70.770	69.844	0.9869	Bond failure
C60.32LT	56.967	55.855	0.9804	Bond failure
C60.3L2T	53.973	50.725	0.9398	Bond failure
Average			0.9823	
COV			0.0238	

5. DISTRIBUTION OF LONGITUDINAL STRESS FOR EACH LAYER OF CFRP

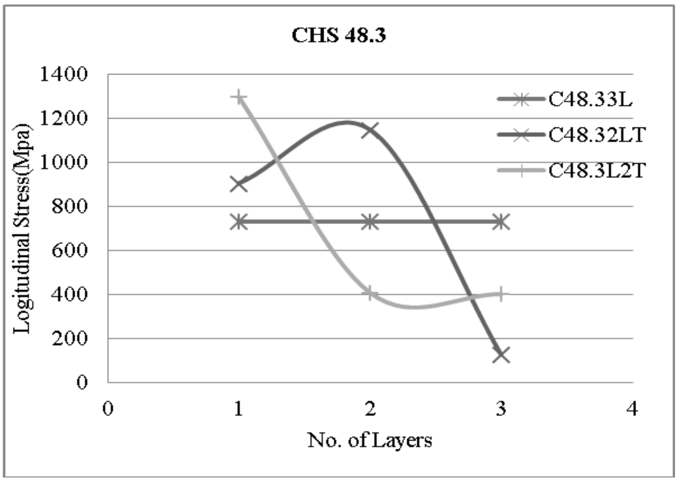

Figure 7: Longitudinal stress distribution for different orientation of CFRP layers for CHS48.3

Steel hollow sections especially circular sections are widely used in marine structures and liquid retaining structures for their high hoop stress along with longitudinal stress. With the increment of diameter of CHS load also increase but depends on their slenderness ratio. The effect of longitudinal stress distribution of different layers of CFRP sheet is investigated at failure load at the joint of CFRP strengthened CHS. Figure 7 show that C48.33L specimen has almost same longitudinal stress at all layers of CFRP. For C48.32LT specimen 2^{nd} layer of CFRP attains largest longitudinal stress for their interlayer shear stress between 2^{nd} and 3^{rd} layer because of their different orientation. For C48.3L2T specimens 1^{st} layer carries the largest longitudinal stress for their interlayer shear stress between 1^{st} and 2^{nd} layer because of their different orientation. It can be concluded that longitudinal stress sharply decreased due to change of CFRP layer orientation. The same phenomenon observes in Figure 8 which represents for CHS60.3.

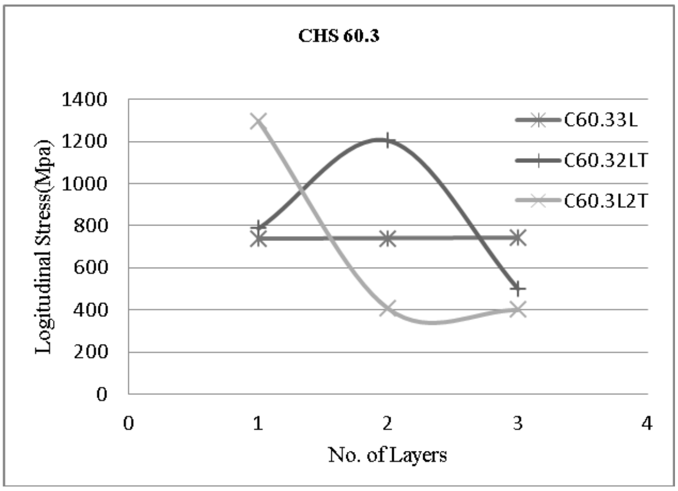

Figure 8: Longitudinal stress distribution for different orientation of CFRP layers for CHS60.3

6. DISTRIBUTION OF HOOP STRESSES FOR EACH LAYER OF CFRP

Circular hollow sections were subjected to tensile force along the longitudinal direction in this FE analysis. So their hoop stress distributions were same along the cross section but vary in longitudinal direction. The effect of hoop stress distribution of different layers of CFRP sheet was investigated at failure load at the joint of CFRP strengthened CHS. Figure 9 showed that C48.33L specimen has almost same hoop stress at all layers of CFRP and the value is unimportant. For C48.32LT specimen 1st and 2nd layer of CFRP attains very tiny hoop stress as they are oriented in longitudinal direction and unidirectional CFRP sheet has no stress in transverse direction. But 3rd layer oriented in transverse direction so this layer attains more hoop stress. For C48.3L2T specimens 3rd layer carries the largest hoop stress for their same orientation in 2nd and 3rd layer.

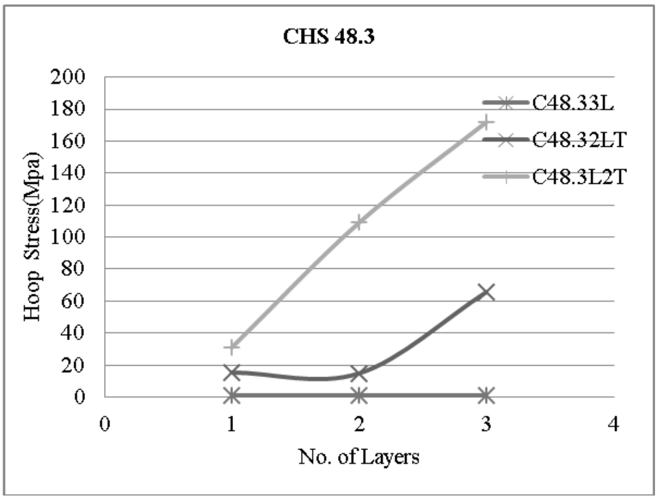

Figure 9: Hoop stress distribution for different orientation of CFRP layers for CHS48.3

The same phenomenon observes in Figure 10 which represents for CHS60.3. But hoop stress of C48.32LT specimens more than that of C60.32LT and hoop stress are same for C48.3L2T and C60.3L2T. It can be concluded that the L2T wrapping scheme is better for slender CHS.

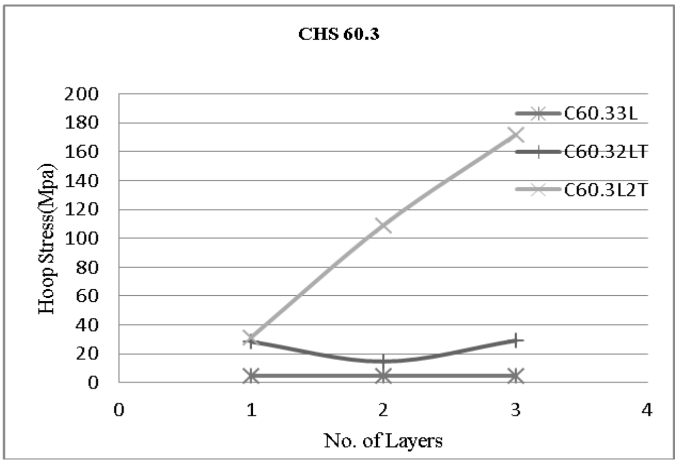

Figure 10: Hoop stress distribution for different orientation of CFRP layers for CHS60.3

7. CONCLUTION

In this paper the experimental test results from CFRP wrapped specimens under tension were presented. CFRP sheet was used in different orientation at effective bond length. The investigated variables were load carrying capacity and stress distribution at effective bond length of circular steel hollow sections having diameters of 48.3 mm and 60.3 mm. The findings from the comparative studies are summarised as follows:

- Load carrying capacity is maximum when there layers of CFRP oriented in longitudinal direction.
- Maximum longitudinal stress developed in 2nd layer of CFRP for 2LT specimens. Due to change of orientation of CFRP, the longitudinal stress distribution sharply decreased.
- Maximum hoop stress developed in 3rd layer of CFRP for L2T specimens. Due to change of orientation of CFRP, the hoop stress distribution sharply increased.
- L2T specimens are good for circular hollow sections having large slender ratio. Due to they are large hoop stress, the effect of slenderness could be minimized
- 2LT specimens are better for circular hollow sections due to load carrying capacity and both stresses are high compare with other combinations.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

OPTIMIZATION OF COLUMN

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ABSTRACT

The ratio of longitudinal steel area Ast to gross concrete cross section Ag is in the range from 0.01 to 0.08, according to ACI Code 10.9.1 While we design a section of a column the common practice is to choose an arbitrary section and check that for bending and axial load with a reinforcement about 2-5 % but whether is it economical or not we don't know. However for a particular moment and load only for a certain percentage of reinforcement the section is optimized. But thesis not for a fixed percentage of reinforcement. As the cost of concrete and reinforcement may increase or decrease independently so economic section is not fixed for 2-5% reinforcement only. This is dependent on a factor called price ratio X . X=Price of 1 cft reinforcement(490lb)/ Price of 1 cft concrete .This is true for every column that after a certain price ratio its optimized only at 1% reinforcement on that loading and moment condition. In many cases it is seen that Using 2-3% reinforcement costs about 1.5 times of 1% reinforcement .

Keywords: *Rectangular column, price ratio,1-1.2% reinforcement.*

1. INTRODUCTION

The ratio of longitudinal steel areaAst to gross concrete cross section Ag is in the range from 0.01 to 0.08, according to ACI Code 10.9.1. The lower limit is necessary to ensure resistance to bending moments not accounted for in the analysis and to reduce the effects of creep and shrinkage of the concrete under sustained compression. Ratios higher than 0.08 not only are uneconomical, but also would cause difficulty owing to congestion of the reinforcement, particularly where the steel must be spliced. Generally, the larger diameter bars are used to reduce placement costs and to avoid unnecessary congestion. According to ACI Code 10.9.2, a minimum of four longitudinal bars is required when the bars are enclosed by spaced rectangular . The behavior of short, axially loaded compression members, for lower loads for which both materials remain in their elastic range of response, the steel carries a relatively small portion of the total load. The steel stress fs is equal to n times the concrete stress:

$$fs = nfc \qquad (A)$$

where n = Es /Es is the modular ratio. In this range the axial load P is given by

$$P = f c \; [Ag + (n - 1)Ast] \qquad (B)$$

where the term in square brackets is the area of the transformed section. Equations (A) and (B) can be used to find concrete and steel stresses respectively, for given loads, provided both materials remain elastic. The nominal ultimate strength of an axially loaded column can be found, when the column is in by- axial bending we use LOAD CONTOUR METHOD or RECIPROCAL LOAD METHOD to design a section of a column. While we design a section of a column using this method we hardly think of the cost of that section. the common practice to design a column is to choose an arbitrary section and check that for bending and axial load with a reinforcement about 2-.5 % but whether is it economical or not we don't know . Design of a column for a particular biaxial moment and axial load may vary from engineer to engineer due to neglecting optimizing criteria . However for a particular moment and load there is only one section which is economical, it means only for a certain percentage of reinforcement the section is optimized. But this is not for a fixed percentage of reinforcement. As the section has its component i.e concrete and reinforcement and the cost of these material are different which may increase or decrease independently . For example let now at 2012 the cost of one cft reinforcement (490 lb) is 50 times of one cft concrete (Ready Mix) . For this criteria a particular section of column is optimized at 1% reinforcement. Let at 1980 the the cost of one cft reinforcement was 20 times of one cft concrete . For that criteria, Is the particular section of column was optimized at 1% reinforcement.? of course not. using 3.5-4 % or more reinforcement was economical on that time. In the similar way it may be more than 50 times like 100 or more at 2020. As the value changes independently so economic section is not fixed for a certain percentage of reinforcement only. Now for a building if we optimize all of column, there will be a great variation in section. in that case form working may be a remarkable problem. so we will optimize in such a way that form working and placement of beam reinforcement may not be interrupted.

2. METHODOLOGY

a) Different type of building was chosen where the arrangement of column were not symmetrical.
b) For a specific column different type of section was chosen for which the required reinforcement lies between 1-5 % of gross area.
c) Then economic analysis of different section for 1-5 % of reinforcement was done and most economical section ie, most economic percentage of reinforcement was found.
d) This is done for various column of a specific building where the axial load and bi-axial moment are different.
e) The whole process is repeated for different type of building model.
f) To design the column here we use ETABS 9.7 and PCA COL and to select a optimized section primarily we use Programming Language c++

3. RESULTS AND DISCUSSION

Data tables and graphs of analysis are shown below.
In graph A -1 we can see how the reinforcement percentage changes with the changes of cost. When the price ratio (price of 1cft reinforcement/price of 1 cft concrete) is one then we can use 8% reinforcement or even more if we can avoid the difficulty owing to congestion of the reinforcement , particularly where the steel must be spliced .To avoid difficulty 8% reinforcement is chosen. But when the price ratio increases then % of reinforcement also changes and after a certain price ratio the economical percentage of reinforcement become1 % For example we can see the graph and table below,here for axial load 500 kip and for a price ratio 1-100 the percentage of reinforcement changes. When X (price ratio) is 1-16 its 6.5% reinforcement , at X=17Its 4.5 % ,at X=18-24 its 1.75% and after X=25 its only 1%. This is true for every column that after a certain price ratio its optimized only at 1% reinforcement on that loading and moment condition. we can't avoid 1% of reinforcement to make it more economical as the lower limit is necessary to ensure resistance to bending moment not accounted for the analysis and to reduce effects of creep and shrinkage of concrete under sustained compression without this concrete have only 1/10 tensile strength of its compressive strength and zero tensile strength after crack.

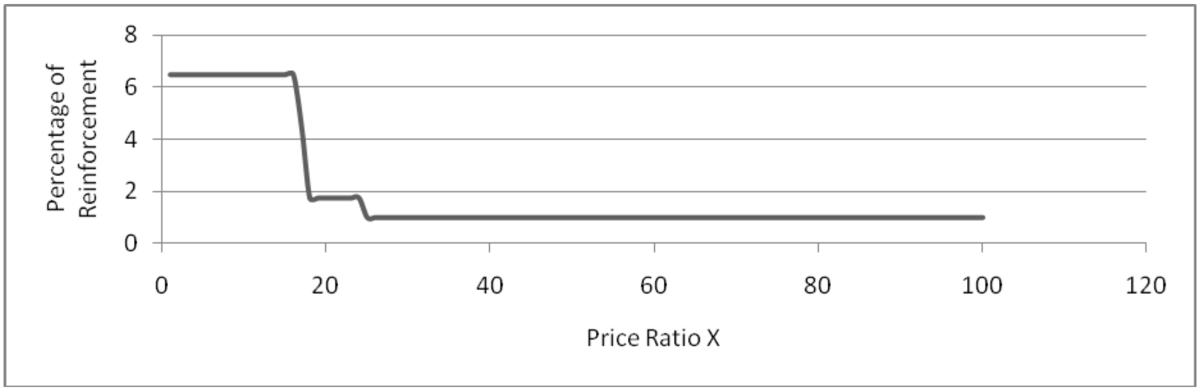

fig1:price ratio VS percentage of reinforcement graph

Ttable:1 Optimized percentage of reinforcement for various price ratio X

X=((price of rod(ton)/price of concrete(cft))x223x10^{-3}

	0	1	2	3	4	5	6	7	8	9
X	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
10	6.5	6.5	6.5	6.5	6.5	6.5	6.5	4.5	1.75	1.75
20	1.75	1.75	1.75	1.75	1.75	1	1	1	1	1
30	1	1	1	1	1	1	1	1	1	1
40	1	1	1	1	1	1	1	1	1	1
50	1	1	1	1	1	1	1	1	1	1
60	1	1	1	1	1	1	1	1	1	1
70	1	1	1	1	1	1	1	1	1	1
80	1	1	1	1	1	1	1	1	1	1
90	1	1	1	1	1	1	1	1	1	1
100	1									

Table :1

Again we analyzethe total cost of column for a 6 story building with 16 column. Here we use different percentage of reinforcement to design the column .

In the fig:2 below we can see at price ratio 50 how much variation in cost may arise.

It is seen that using 8% reinforcement costs almost 2 times of the cost of using 1% reinforcement.Using 2-3% reinforcement costs about 1.5 times of 1% reinforcement .

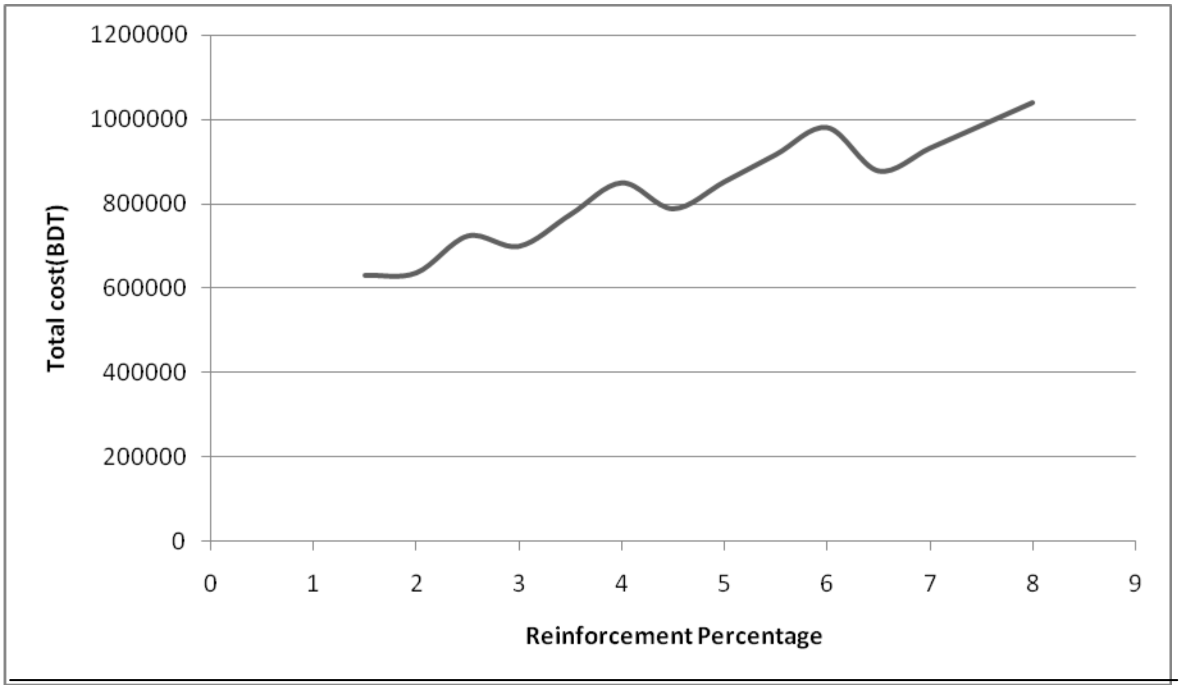

Fig 2:**Reinforcement percentage VS cost graph**

Table :2

Percentage of reinforcement	Size of column (inxin)	Cost(BDT)
1	15x15	530455
1.5	15x15	630682
2	14x14	636703
2.5	14x14	724013
3	13x13	699558
3.5	13x13	774840
4	13x13	850122
4.5	12x12	788510
5	12x12	852655
5.5	12x12	916800
6	12x12	980946
6.5	11x11	878167
7	11x11	932067
7.5	11x11	985967
8	11X11	1039867

Here we see that for this building 1% reinforcement provide the optimized section .Only 14.8 % increase in concrete area (from section 14x14 to 15x15) reduces the reinforcement from 2.5% to 1%. Which reduces the cost 193558 BDT or 26% reduction of total column cost.Here we only use square column .The cost increases in the same way while we use rectangular column. As it is seen that with same axial load capacity and same gross area (concrete area) a square column is more economical than rectangular one, so we choose square column here. In actual field we have to provide rectangular column due to unsymmetrical loading condition and to resist biaxial bending. But we must try to make the column section square as much as possible while cost is a vital factor .

4. CONCLUTIONS

It is seen that with same axial load capacity and same gross area (concrete area) a square column is more economical than rectangular one with a reinforcement of about 1-1.2% for a price ratio greater than 20. So to optimize the section we should select a square column with 1-1.2% reinforcement, when the price ratio is greater than 20. when the price ratio decreases selection of percentage of reinforcement may increases upto 7 or 8 percent gradually.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PROSPECT OF BANGLADESHI FLY ASH IN CEMENT PRODUCTION

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ABSTRACT

Sustainability issue in construction sector came forward in last two decades due to concerns regarding using virgin materials as well as emission of greenhouse gases from production of raw materials. Cement is the prime constituents of concrete and contributes approximately 7% of global man made CO_2 production. The use of fly ash in concrete construction has got increasing importance not only for enhance properties of concrete but also its environmental credentials. Fly ash is a by-product produced from pulverized coal combustion in power generation and formed from the non-combustible minerals found in coal. In the year 2006, two units of 125 MW coal based power plant has started generation in Barapukuria, Bangladesh. The power sector master plan of Bangladesh projects a high growth in coal based power generation although there is lack in strategy on the use of generated fly ash. This study aimed to explore the possibility of using Boropukuria fly ash in concrete construction. Two different grades of concrete (M28 and M38), each with eight different cement replacement level 0, 10, 20, 30, 40, 50, 60 and 70% were used for the experimental program. Among all the concretes studied, the optimum amount of cement replacement is reported 30%, which provides around 12% higher compressive and 53% higher tensile strength as compared to OPC concrete.

Keywords: *Cement, Compressive strength, Environmental pollution, Fly ash, Tensile strength*

1. INTRODUCTION

Sustainable development can be defined as development which meets the needs of people living today without compromising the ability of future generations to meet their own needs. It requires a long-term vision of industrial progress, preserving the foundations upon which human quality of life depends: respect for basic human needs and local as well as global ecosystems. Concrete is the second most consumed material on Earth after water and is an essential product in the building sector. Portland cement is one of the ingredients in concrete. Cement is a fine grey powder and constitutes 7 to 15% by weight of concrete's total mass. The net cement production in the world is increased from about 1.4 billion tonnes in 1995 to almost 3 billion tonnes in the year 2010, expected to be around 5 billion tonnes in the year of 2040. Cement manufacture is an energy intensive process. Consuming energy from fossil fuels such as oil and coal creates carbon dioxide (CO_2), the most important Greenhouse Gas (GHG) causing climate change. On an average 0.72–0.98 tonne of CO_2 is produced for every tonne of cement production (IEA, 2006). Almost all industries know that in order to continue to meet the demands of a growing world population, they must become smarter in the way they use, reuse and recycle raw material, energy and waste in the economy. Using waste from other industries as raw material is a huge opportunity for the cement industry to reduce its environmental impact, because it allows companies to access materials for use in the kiln and the mill without extracting them directly from the ground. There are a number of mineral by-products produced by the mining and power generation industries that contain useful materials that can be extracted for use in cement production, or in making concrete. Being emission a key issue to attain sustainability in construction industry, supplementary cementitious materials (SCM) are gaining interest. Numerous researches has shown potential of using SCMs for instance pulverized fuel ash (fly ash) from coal combustion, GBBS from iron industry, Silica Fume and Metakaolin (Duran, 2011). These SCMs provide duel benefits in concrete construction. Those not only reduce the emission of CO_2 in material production but also improve several properties of fresh and hardened concrete, for example, workability, water demand, permeability and finally durability. It is generally agreed that with the proper selection of admixtures, mixture proportioning and curing, supplementary cementitious materials can noticeably improve the durability of concrete (Zichao, 2003). Recently these has been a growing trend for the use of SCMs in the production of composite cement because of ecological, economical and diversified product quality reason.

Fly ash is an inorganic, non-combustible by-product of coal - burning power plants. As coal is burnt at high temperatures, carbon is burnt off and most of the mineral impurities are carried away by the flue gas in the form of ash. The molten ash is cooled rapidly and solidifies as spherical, glassy particles (Malhotra, 2002). Fly ash particles range in diameter from <1 microns up to 150 microns. Fly ash is removed from the flue gas by means of a series of mechanical separators followed by electrostatic precipitators or bag filters. As per ASTM C618 specification Class F ashes are mainly produced from bituminous or anthracite coals and that Class C ashes are mainly produced from sub-bituminous or lignite coals, but the main criterion for classification are its chemical requirements: SiO_2 + Al_2O_3 + Fe_2O_3 > 70% for Class F and SiO_2 + Al_2O_3 + Fe_2O_3 > 50% for Class C. However, many sub-bituminous and lignite coal ashes meet the chemical requirements of Class F.

Fly ash is a pozzolanic material i.e. a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but which will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (ASTM, 1975). In the presence of moisture, alumino-silicates within the fly ash react with calcium ions to form calcium silicate hydrates (Malhotra, 1994). Through pozzolanic activity, fly ash chemically combines with water and calcium hydroxide, forming additional cementitious compounds which result in denser, higher strength concrete. The calcium hydroxide chemically combined with fly ash is not subject to leaching, thereby helping to maintain high density. The conversion of soluble calcium hydroxide to cementitious compounds decreases bleed channels, capillary channels and void spaces and thereby reduces permeability. Depending on the location of each power plant, the unused fly ash is disposed at the ponds, lagoons or landfills. When unused fly ash and bottom ash disposed from coal combustion power plants, it makes major negative environment effects such as air pollution and groundwater quality problem due to leaching of metals from the ashes, specially the unused fly ash which has very small particle size (Janos, 2002). According to the ASTM 618, the fly ash is suitable for use in concrete when no more than 34 percent of the particle is retained on the No. 325 (45μm) sieve. Fineness of ground disposed fly ash plays very important role on compressive strength of concrete. However, the ground disposed fly ashes which have particle sizes retained on sieve No. 325 and less than 5% by weight can be used as good pozzolanic material (Cheerarot, 2004). It should be noted that since pozzolanic reaction can only proceed in the presence of water, fly ash concrete should be cured for long period. Thus, fly ash concrete used in under water structures such as dams will derive full benefits of attaining improved long term strength and water tightness. Sufficiently cured concrete containing good quality fly ash shows dense structure which offers high resistance to the infiltration of deleterious substances. The pozzolanic reactivity reduces the calcium hydroxide content, which results in reduction of passivity to the steel reinforcement but at the same time, the additional secondary cementitious material formed make the paste structure dense and thereby gives more resistance to the corrosion of reinforcement. Although fly ash is an industrial waste, its use in concrete significantly improve the long term strength and durability and also reduce the heat of hydration.

Today, there is a general trend to replace higher levels of Portland cement with fly ash in concrete and is due to considering two main aspects. The first aspect is economics as in most markets fly ash is less expensive than Portland cement. Therefore, as the replacement level of fly ash increases, the cost to produce concrete decreases. The second aspect and arguably the most important is the environment. Fly ash is an industrial by-product, much of which is deposited in landfills if not used in concrete. Also from an environmental perspective, the more fly ash being utilized in concrete, less the demand for Portland cement clinker i.e. the less requirement for Portland cement production and therefore the lower CO_2 emissions. The use of materials such as fly ash as a supplementary cementing material in concrete has become common in North America. If properly used, fly ash can significantly enhance the properties of concrete. Replacement of higher levels of Portland cement with fly ash in reducing the CO_2 emissions associated with the manufacturing of Portland cement is now a day's demand as for manufacture of every tonne of Portland cement, almost one tonne of CO_2 is released into the atmosphere and cement production accounts for approximately 7% of the total carbon dioxide emissions (Mehta, 1999). Therefore it is important for the cement and concrete industry to start utilizing more fly ash to meet these demands rather than increase the Portland cement production (Malhotra, 2002).

Replacing Portland cement with fly ash can reduce the exothermic reaction between cement and water (Bremner, 2004). Because of the slower pozzolanic reaction, partial replacement of Portland cement with fly ash results in a release of heat over a longer period of time. Therefore, the concrete temperature remains lower as the heat is dissipated during hydration (Joshi, 1997). It has been estimated that the contribution of fly ash to early age heat generation ranges from 15-30% of that of an equivalent mass of Portland cement (Berry, 1986). In a large concrete block made with same amount of cementitious material, cement concrete showed a temperature differential of 65°C between the interior and exterior surface whereas fly ash concrete showed only 35°C temperature differential at a placement temperature of 19°C under identical condition (Malhotra, 2002).

Both the strength at a given age and the rate of strength gain of fly ash concrete are affected by the characteristics of the fly ash (properties, chemical composition, particle size, reactivity), the cement with which it is used, the proportions of each used in the concrete, the temperature and other curing conditions (Hobbs, 1983). Although concrete mixtures containing fly ash tend to gain strength at a slower rate than concrete without fly ash, the long-term strength is usually higher (Bremner, 2004). After the rate of strength gain of hydraulic cement slows, the continued pozzolanic activity of fly ash provides strength gain at later ages if the concrete is kept moist; therefore, concrete containing fly ash with lower strength at early ages may have equivalent or higher strength at later ages than concrete without fly ash as long as the concrete is moist cured or exposed to sufficient quantities of moisture during service. High calcium fly ashes (Class C) will show a more rapid strength gain at early ages than concrete made with a lower calcium fly ash (Class F) because Class C ashes often exhibit a higher rate of reaction at early ages than Class F ashes (Smith, 1982). However, Class F ashes will contribute to greater long-term strength gain of concrete than Class C ashes in spite of its slower rate of strength development at early age. Because of its fineness and pozzolanic activity, fly ash in concrete improves the quality of cement paste and the microstructure of the transition zone between the binder matrix and the aggregate. As a result of the continual process of pore refinement due to the inclusion of fly ash hydration products in concrete, a gain in strength development with curing is achieved (Joshi, 1997).

The beneficial use of coal burning power plant fly ash in concrete has increased the interest of researcher for the evaluation of the performance of such concrete. The relevant studies indicate that the percentage of cement replaced with fly ash and their relative proportion for making concrete is very important. Concrete mixes made by replacing cement with fly ash are reported to show better results for compressive, tensile as well as flexural strength, freezing and thawing resistance, shrinkage, permeability and abrasion resistance than conventional concrete mixes (Tarun, 1996). Fly ash has dual effects in concrete i.e. as a micro-aggregate and as a pozzolana. Fly ash improves the interfacial bond between the paste and the aggregates in concrete (Poon, 2001). According to Malhotra (2000), the concrete incorporating moderate and high volumes of fly ash showed superior resistance against strength deterioration, rebar corrosion and the penetration of chloride ions compared to the control concrete specimen.

A sustainable industrial growth will influence the cement and concrete industry in many respects as the construction industry has environmental impact due to high consumption of energy and other resources. One important issue is the use of environmental-friendly concrete, which is termed as green concrete, to enable worldwide infrastructure growth without affecting the environment (Claus, 2005). Concrete is normally considered to be porous due to existence of capillary pores, gel pores and potentially porous cement-aggregate interface zones. Important traditional means to improve concrete durability are through reduction of water to cement ratio and/or increase of the moist curing period. Recently, many new materials and techniques have been developed to control corrosion by reduction of penetrable aggressive species. Partial replacement of Portland cement with supplementary cementitious materials has been used widely in aggressive environmental applications. It is generally recognized that the introduction of pozzolan in blended cements improves concrete protection against chloride-induced corrosion of steel reinforcement by reducing its permeability/diffusivity, particularly to chloride ion transportation and increasing the resistivity of the concrete (Thomas, 1999). The Mineral admixtures having high fineness react with the product liberated at early ages during hydration and form secondary C-S-H gel (also referred as tobermorite gel). This gel is less densed and has more volume than primary C-S-H gel. Therefore, it fills all the pores inside concrete and makes the concrete more impermeable thereby reducing the risk of chloride and sulfate induced deterioration.

1.1 Research Significance

Portland cement is the most important constituent of concrete. Unfortunately, cement manufacturing consumes large amount energy about $7.36*10^6$ kJ per tone of cement. The net cement production in the world is being increased from about 1.4 billion tones in 1995 to the projected value of almost 5 billion tones in the year 2040. This would lead to the emission of about 5 billion tones to CO_2 in the atmosphere. In order to reduce the harmful green house effect, use of cement may be replaced with other environmentally friendly and efficient cementitious material such as fly ash (Reiner, 2006). It also ensures the proper utilization of fly ash, by-product of coal combustion in power plants, in an effective way which otherwise been dumped making environmental hazard. Limited studies are reported to be carried out to investigate the performance of Barapukuria fly ash as partial replacement of cement in concrete production. In this study, an attempt has been made to observe the effect of fly ashes produced from coal based power generation in Barapukuria, Bangladesh with different levels of cement replacement on the strength characteristics of concrete.

2. EXPERIMENTAL PROGRAM

The experimental program was planed to quantify the compressive strength and tensile strength of concrete using fly ash as partial replacement of cement. Cement replacement at various percentage levels were used in this investigation to observe the effects of different fly ash levels in concrete in contributing strength at various ages of curing.

2.1 Materials Used

(a) Cement: ASTM Type I Portland Cement was used as binding material. Chemical compositions of OPC are given in Table-1.
(b) Fly ash: A low calcium ASTM Class F fly ash was used in this investigation. Chemical analysis of the fly ashes conducted using X-ray fluorescence (XRF) study is shown in **Table 1.**

Table 1 : Chemical Composition of Ordinary Portland Cement and Fly Ash

Constituents	Composition	%	%
Calcium Oxide	CaO	65.18	0.65
Silicon Di-Oxide	SiO_2	20.80	51.49
Aluminum Oxide	Al_2O_3	5.22	31.60
Ferric Oxide	Fe_2O_3	3.15	2.80
Magnesium Oxide	MgO	1.16	0.28
Sulfur Tri-Oxide	SO_3	2.19	0.19
Sodium Oxide	Na_2O	--	0.18
Loss on Ignition	--	1.70	4.2
Insoluble Residue	--	0.6	--

-- = not measured items.

(c) Aggregate: Locally available natural sand passing through 4.75 mm sieve and retained on 0.075 mm sieve was used as fine aggregate. The coarse aggregate was crushed stone with a maximum nominal size of 12.5 mm. The grading of the aggregates and its physical properties are given in **Table 2.**

Table 2: Grading and Physical Properties of Coarse and Fine Aggregate

Properties	Coarse Aggregate	Fine Aggregate
Grading of Aggregates		
Sieve Size (mm)	**Cumulative % Passing**	
25.0	100	--
12.5	100	--
9.5	45	--
4.75	0	100.0
2.36	--	94.0
1.18	--	78.5
0.6	--	55.5
0.3	--	13.0
0.15	--	2.5
Physical Properties of Aggregates		
Specific Gravity	2.67	2.59
Unit Weight	1635 kg/m^3	1540 kg/m^3
Fineness Modulus	6.45	2.57
Absorption Capacity	0.8 %	1.2 %

2.2 Mix Design and Sample Preparation

Two different grades of concrete namely M28 and M38 were used in the program. Seven different mix proportions of cement fly ash (90:10, 80:20, 70:30, 60:40, 50:50, 40:60, 30:70) were used as cmentitious material. Cement fly ash mix ratio of 100:0 i.e. plain concrete specimens were also cast as reference concrete for comparing the properties of fly ash concrete. Thus the fly ash concrete means the concrete made by using cement and fly ash as cementitious material with sand, stone chips and water. Relevant information of different concrete mixes is given in **Table-3.**

Table 3 : Mix proportions and properties of fresh concrete

Mixture constituent & properties	Grade of Concrete	
	M28	**M38**
Cement (kg/m^3)	435	500
Water (kg/m^3)	218	218
Sand (kg/m^3)	545	520
Stone Chips (kg/m^3)	1150	1120
w/(c+fa)	0.50	0.44
Slump (mm)	68	60
Air content %	1.3	1.1

Around 300 no's of cubical specimens of 100 mm size were prepared according to the mix proportion as described. The small size of specimen i.e. 100 mm cube was taken in order to accommodate large number of specimens in the limited sized curing tanks. The specimens were demoulded after 24 hours of casting and cured in plain water at 27±2°C. The concrete test specimens were designated keeping concrete grade and replacement as variable. Thus M38FA40 concrete means grade of concrete is M38 and cement fly ash mix ratio is 60:40.

2.3 Experimental Procedures

The compressive strength and tensile strength of various water cured concrete specimens were tested at the ages of 3, 7, 28, 56, 90 and 180 days in accordance with the BS EN 12390-3:2009 for compressive strength and BS EN 12390-6:2000 for tensile strength. At each case, the reported strength is taken as the average of three tests results.

3. RESULTS AND DISCUSSIONS

3.1 Compressive Strength

The compressive strength of OPC and fly ash concrete of two different grades M28 and M38 has been graphically presented in **Fig.1** and **Fig.2**. Also for the ease of comparison, the relative compressive strengths are plotted in **Fig.3** and **Fig.4**. At early ages of curing, OPC concretes achieve relatively higher compressive strength as compared to fly ash concrete. Test result shows that the 7 days compressive strength for OPC concrete is 9%, 16%, 26%, 34%, 43%, 59% and 75% higher than M38FA10, M38FA20, M38FA30, M38FA40, M38FA50, M38FA60 and M38FA70 concrete respectively. Up to curing period of 56 days, compressive strength is seen to decrease with the increase in fly ash content when compared with no fly ash concrete. 90 days compressive strength test result of the specimens up to 50% replacement level are very similar to OPC concrete, within the range of ±12% variation. Compressive strength is slightly higher by 6%, 9%, 12% 8% and 1% for M38FA10, M38FA20, M38FA30, M38FA40 and M38FA50 concrete respectively; whereas the 90 days strength for M38FA60 and M38FA70 concrete is reported to be lower by 31% and 43% respectively when compared with no fly ash concrete. 180 days compressive strength data for M38FA10, M38FA20, M38FA30, M38FA40 and M38FA50 concrete are respectively 8%, 11%, 16%, 13% and 2% higher than no fly ash concrete. M38FA60 and M38FA70 concrete strength are lower than M38FA0 concrete by 25% and 39%. Cement normally gains its maximum strength within 28 days. During that period, lime produced form cement hydration remains within the hydration product. Generally, this lime reacts with fly ash and imparts more strength. For this reason, concrete made with fly ash will have lower strength than cement concrete up to 28 days and subsequently higher strength at the later ages of curing. Fly ash retards the hydration of C_3S in the early stages but accelerates it at later stages. Conversely in cement concrete, this lime would remain intact and with time it would be susceptible to the effects of weathering, loss of strength and durability. Yamato and Sugita (1983) found that the later age strength of fly ash concrete was higher than that of the control.

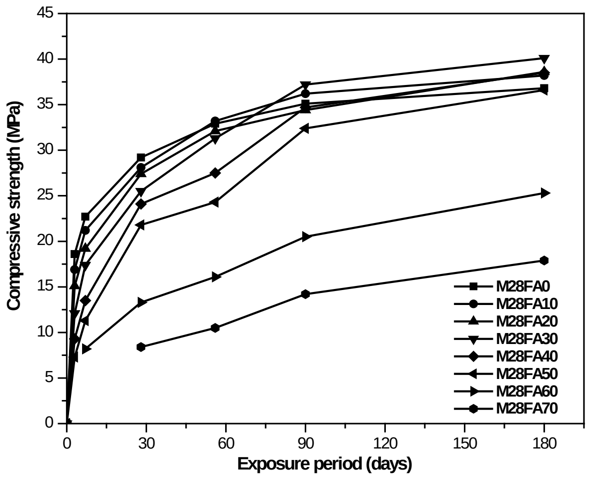

Figure 1: Compressive strength - Exposure time relation for M28 fly ash concretes

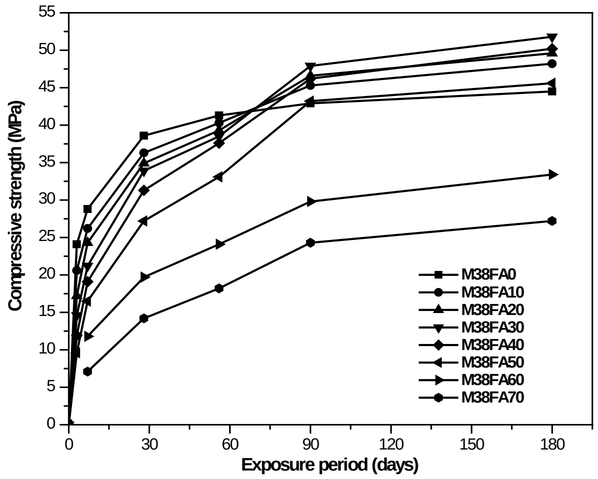

Figure 2: Compressive strength - Exposure time relation for M38 fly ash concretes

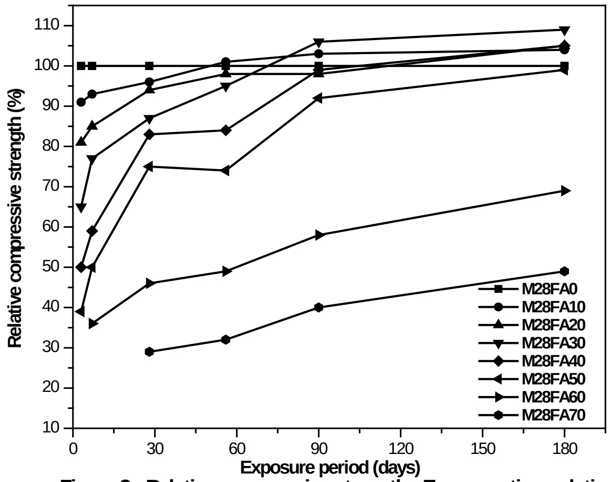

Figure 3: Relative compressive strength - Exposure time relation for M28 fly ash concretes

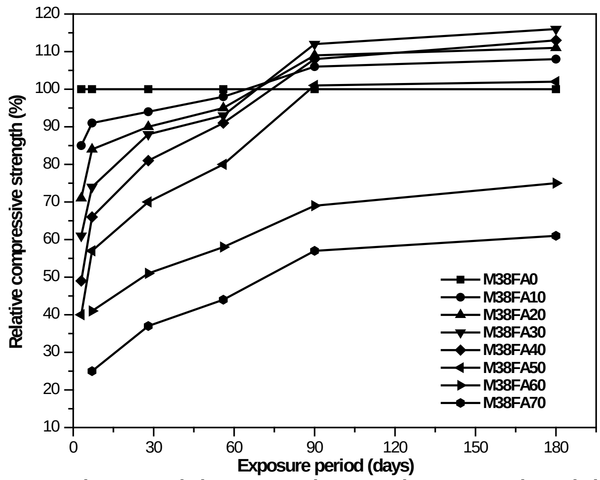

Figure 4: Relative compressive strength - Exposure time relation for M38 fly ash concretes

Rate of strength gaining for different types of concrete is observed to vary with the grade of concrete. Gain in strength is lower for the higher grade of concrete. Among all the concrete studied, 180 days compressive strength is increased by around 26%, 31%, 32%, 37%, 33% and 25% for concrete M28FA0, M28FA10, M28FA20, M28FA30, M28FA40 and M28FA50 respectively as compared to 28 days strength of M28 grade OPC concrete; whereas the same value is increased by around 15%, 25%, 28%, 34%, 30% and 18% for concrete M38FA0, M38FA10, M38FA20, M38FA30, M38FA40 and M38FA50 respectively compared to 28 days strength of no fly ash M38 grade concrete. At the end of 180 days curing period, the overall strength gaining for M38 grade concrete is around 3% lower as compared to M28 grade concrete. Thus it is seen that strength gaining is relatively faster for lower grade concrete as compared to higher grade concrete.

3.2 Tensile strength

The tensile strength of concrete mixes made with and with out fly ash was determined at the ages of 3, 7, 28, 56, 90 and 180 days. **Fig.5** and **Fig.6** shows the variation of tensile strength with age for different fly ash concretes**.** Also for the ease of comparison, the relative tensile strength is plotted in **Fig.7** and **Fig.8**. The tensile strength of the specimens is seen to increase with age. At early ages of curing (3 days and 7 days) the tensile strength decreases with increase in fly ash content in concrete. However the rate of decrease diminishes with increasing age of curing. As compared to control specimen tensile strength values are 94%, 90%, 99% and 91% for M28FA10, M28FA20, M28FA30 and M28FA50 concrete respectively at the curing age of 28 days.

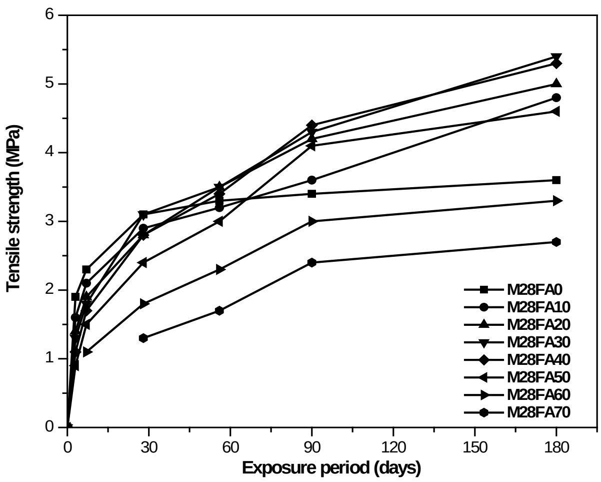

Figure 5: Tensile strength - Exposure time relation for M28 fly ash concretes

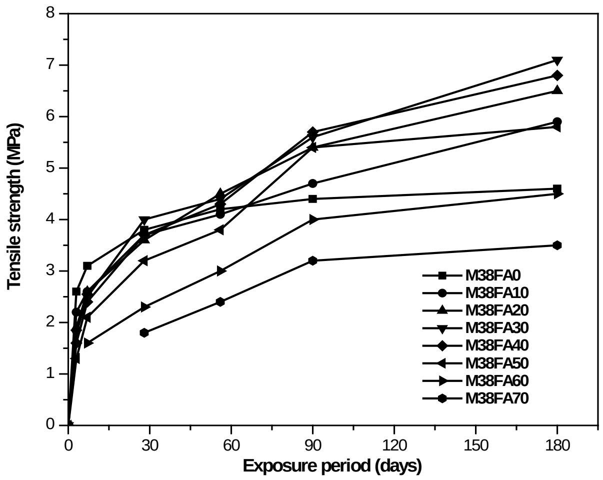

Figure 6: Tensile strength - Exposure time relation for M38 fly ash concretes

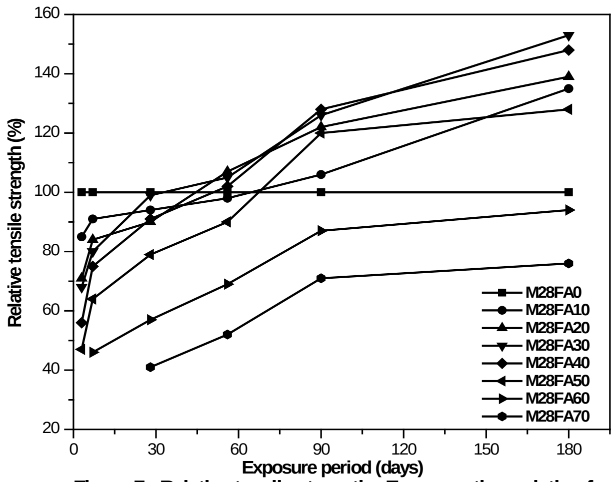

Figure 7: Relative tensile strength - Exposure time relation for M28 fly ash concretes

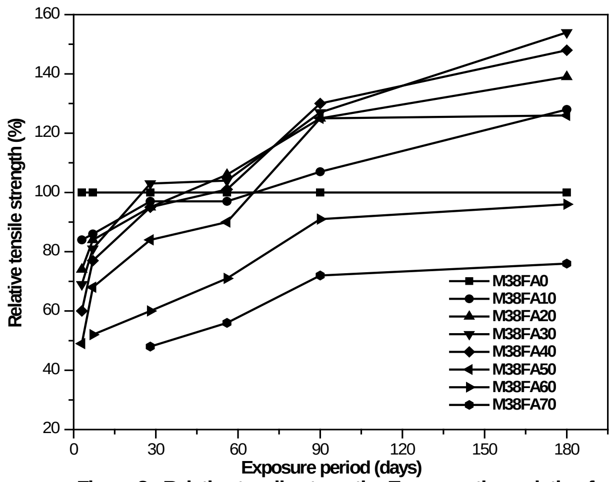

Figure 8: Relative tensile strength - Exposure time relation for M38 fly ash concretes

After 90 days, maximum tensile strength of 4.4 MPa was achieved for M28FA40 concrete, with an increase of 28% higher strength than M28FA0 concrete. At similar age of curing, even the concrete M28FA10, M28FA20, M28FA30 and M28FA50 showed higher tensile strength of 6%, 22%, 28% and 20% respectively than OPC concrete. At an age of 180 days curing, a maximum tensile strength of 5.4 MPa was achieved for M28FA30 concrete which is 53% higher than the reference concrete. Even 10%, 20%, 40% and 50% fly ash replaced concrete showed higher strength. However, M28FA60 and M28FA70 concrete provided a decrease in strength of around 6% and 24%. It is due to the fact that fly ash being a pozzolanic material, the reactive silica of pozzolan and calcium hydroxide producing form the hydration of cement react together and produce calcium silicate hydrate which imparts strength for concrete. As it takes time to produce $Ca(OH)_2$ by hydration of cement, strength gaining rate slows down at initial ages of curing but increases at the later ages. Korac and Ukraincik (1983) found that the early-age strengths upto 50% fly ash concretes were lower than that for the controls and after long curing period, the strengths were found comparable.

Again, the rate of strength gaining for different types of concrete is observed to vary with the grade of concrete. Gain in tensile strength is higher for the higher grade of concrete. Among all the concrete studied, 180 days tensile strength is observed to be increased by about 15%, 56%, 61%, 76%, 71%, 48% and 8% for concrete M28FA0, M28FA10, M28FA20, M28FA30, M28FA40, M28FA50 and M28FA60 respectively as compared to 28 days strength M28 grade OPC concrete; whereas the same value is increased by around 21%, 54%, 68%, 85%, 78%, 52% and 16% for concrete M38FA0, M38FA10, M38FA20, M38FA30, M38FA40, M38FA50 and M38FA60 respectively compared to 28 days strength of M38 grade no fly ash concrete. At the end of 180 days curing period, the overall strength gaining for M38 grade concrete is around 7% higher as compared to M28 grade concrete. Thus it is seen that tensile strength gaining is relatively faster for higher grade concrete as compared to lower grade concrete.

4. CONCLUSIONS

Based on the results of the investigation conducted on different fly ash concrete made with various level of cement replacement as mentioned and cured for various curing period up to 180 days, the following conclusions can be drawn:

(1) The rate of gain in strength of fly ash concrete specimens is observed to be lower than the corresponding OPC concrete.

(2) Fly ash concrete mix having various cement replacement level up to 40% exhibited satisfactory results for both compressive and tensile strength.

(3) The optimum fly ash content is observed to be 30% of cement. Fly ash concrete with 30% cement replacement shows around 12% higher compressive strength than OPC concrete after 180 days curing. The corresponding increase in tensile strength is reported to be around 53%.

(4) Use of high volume fly ash in any construction work as a replacement of cement, provides lower impact on environment (less CO_2 emission) and judicious use of resources (energy conservation, use of by-product).

(5) Use of fly ash reduces the amount of cement content as well as heat of hydration in a concrete mix. Thus, the construction work with fly ash concrete is environmentally safe and also economical.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE BASED FIRE SAFETY MANAGEMENT IN COMMERCIAL MIXED USE BUILDING OF BANGLADESH

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ABSTRACT

The modern approach to fire safety design move forward to performance based fire safety engineering which started about 15 years ago in developed industrialized countries. The regulations of Bangladesh national building code (BNBC 1993) like Fire Resistance Rating(FRR) as passive protection has come to fall down in recent fire incidents occurred in different commercial mixed use buildings like Bashundhara city, BSEC Bhavan. A detailed survey was conducted to a typical commercial mixed use building in Dhaka city incorporating the process of data collection involved the collection of fire statistics data from BFSCD (Bangladesh Fire Service and Civil Defense). Based on the results of data analysis and previous study a fire risk estimate is determined using an Event Tree Analysis (ETA). The important findings of the study includes the high risk of fire in commercial mixed use building with inappropriate fire safety measure both in the fire department and building inspection. The results obtained from the study focuses as the barrier of optimum fire safety are inappropriate installation of fire inspection, failure of human response to suppress fire, inappropriate response of fire brigade, lack of training of occupants in response to fire. The results from ETA analysis concluded the most cost effective scenario with a ranked form.

Keywords: *Event Tree Analysis (ETA), Smoke Detection System, Fire Fighters Action Time (FAT), Manual Fire Fighting, Fire Resistance Rating (FRR).*

1. INTRODUCTION

Urban settlements of the world are more vulnerable to fire hazard due to the involvement of huge population in economic activities, industrial and other activities. Urban fire incidence is alarmingly increasing all over Bangladesh particularly in larger urban centers. Although the rate of incidence is high, it may be controlled by taking proper management of fire safety through addressing the present risks. In Bangladesh the expansion of commercial mixed use building is notified due to its rapid urbanization during last decades. Especially the utility of the building ranges from the market place, commercial space, offices to residential apartments. But the most vulnerable thing is the improper management of fire safety of the commercial mixed use buildings. Fire incidence is one of such phenomena which is at present causing huge economic loss as well as tragedy of human death in frequent manner. The death numbers due to the accidents were 102, 269, 214, 92 in the years 2008 to 2011. Apart from death tragedy total economic losses were estimated 230.93, 305.90, 326.461, 554.15 crore taka at the same years (BFSCDA, 2011).

2. OBJECTIVES AND SCOPE OF THE STUDY

This research work tries to find out proper managerial approach for fire safety through introducing the risk in different segment of fire safety measure that have already taken or mandatory as legislative requirements of code practices. The study focuses the fire safety management from both point of structural safety itself and institutional capacity of Bangladesh Fire Service & Civil Defense (BFSCD). The perception of fire service authorities also been shown here. Furthermore the study also conducts about code practices in Bangladesh like BNBC Code. This work basically covers the field of commercial mixed use building especially in Dhaka city, because the urban growth is more in Dhaka city than others. The sample buildings studied here is seven storied commercial mixed use (utility like market place, offices, super shop etc.).

3. METHODOLOGY

In the preliminary approach of methodology the secondary data are collected from the relevant publications, newspapers and Bangladesh Fire Service and Civil Defense (BFCD) official records up to 2011. A field survey is also conducted for assessing the system installments for fire safety of the building (sample) properties like building geometry, combustible materials etc. Data analysis procedure comprises with a time series analysis of relevant data from secondary data collection. Risk assessment procedure combined with the Event Tree Analysis (ETA) followed here as clearly described in Hagg (1999). Risk assessment procedure also conducted a probabilistic simulation called Monte Carlo simulation for the sensitivity of the estimated risks by a software packages @Risk 5.7.

4. RESULTS AND DATA ANALYSIS

4.1 Fire Incidences Scenario Regarding Commercial Mixed Use Building

The incidences scenario is most alarming from the last two or three decades. The data base of Bangladesh Fire Service and Civil Defense (BFCD) keeps the through and compact records of all fire incidences during last decades. The data base oriented with several types as like:

1. Number of accidents with time and occupancy basis.
2. Accident causes with both the monthly and yearly basis.
3. Causality and Property damages in causes and occupancy basis.

The fire incidences from 2008 to 2011 in Bangladesh were 9310, 12182, 11857, 13041 whereas in Dhaka city the incidences were 3103, 3746, 3879, 3998. In last ten years the rate is increased notably (figure 1). In case of commercial mixed use building the number of accidents were 2249, 2597, 2075, 2254 throughout the whole country. This covers a rate of 24%, 21.3%,17.5%, 17.3% of total accidents. The major causes of fire incidents are electric wiring and cooking appliances as recorded (BFCDA, 2011). Electric wiring fire responsible for about 40%,37%, 42%, 36% of total fire incidences from 2008 to 2011 meanwhile cooking appliances responses from about 23% to 24% during those years.

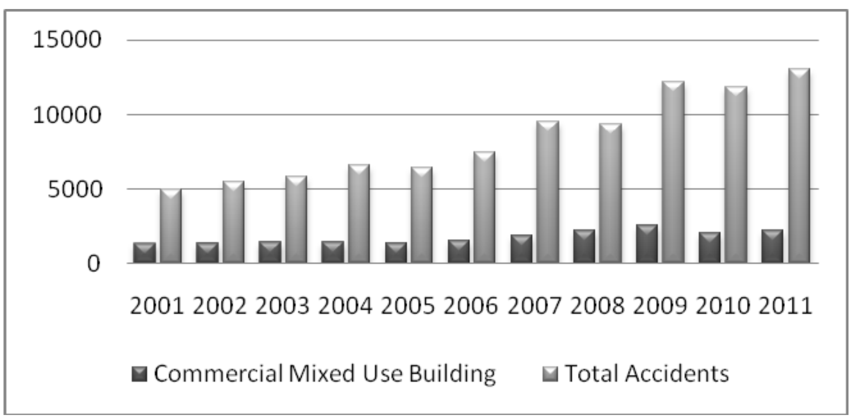

Figure 1: Reported Fire Incidents in Bangladesh (2001-2011)

This is most alarming in the rate of economic losses which were 76%, 92%, 74%, 76% of the total property losses, that were 230.9, 305.9, 326.4, 554.15 crore taka from 2008 to 2011(figure 2). The losses represent a scatter percentage rate for sometimes equal rate or sometimes with excessive growth of rate.

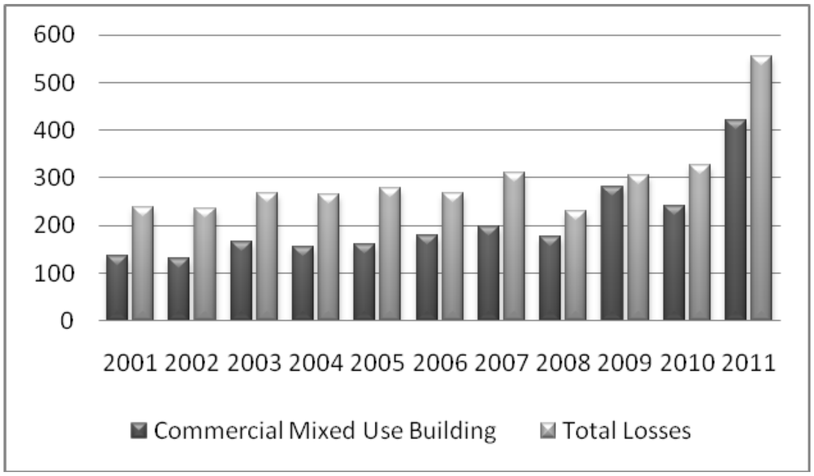

Figure 2: Economic Losses due to Fire Incidences (2001-2011)

In case of causalities the number death is of 23, 47, 38, and 24 in last four years which makes a significant percentage total death. The figure of death rate does not show the complete scenario that is more devastating from the real scenario.

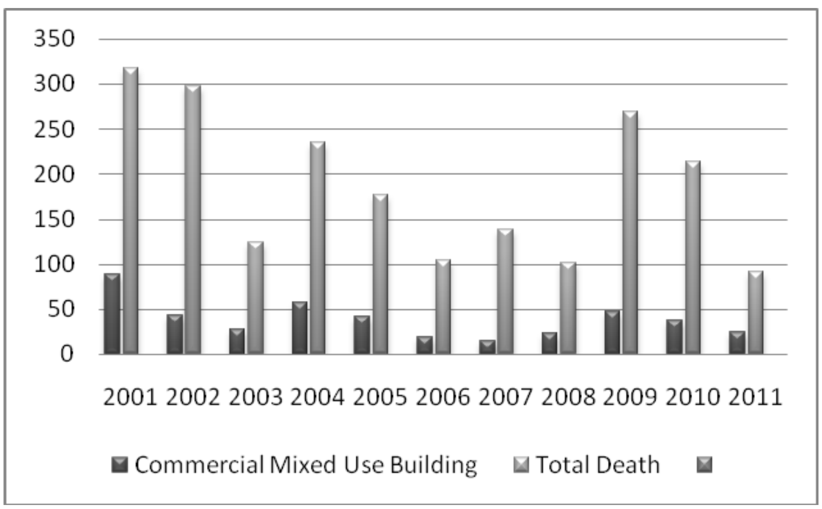

Figure 3: Number of Death in Fire Incidents (2001-2011)

In 2011, almost 3130 fire accidents originated from the 'kitchen burner' which can be identified as the second important reason for fire incidents (figure 3). Every year few 'intentional fires'(Arson) caused in Dhaka city. In the year of 2010 and 2011 intentional fire causes about 245 and 652 accidents but in most of the cases the overall loss due to this reason is very high. Intentional fire is one of the important issues for Dhaka city which is generally the outcome of interpersonal conflict, political destabilization and social unrest, especially in slum areas (Monowarul and Neelopal, 2008).

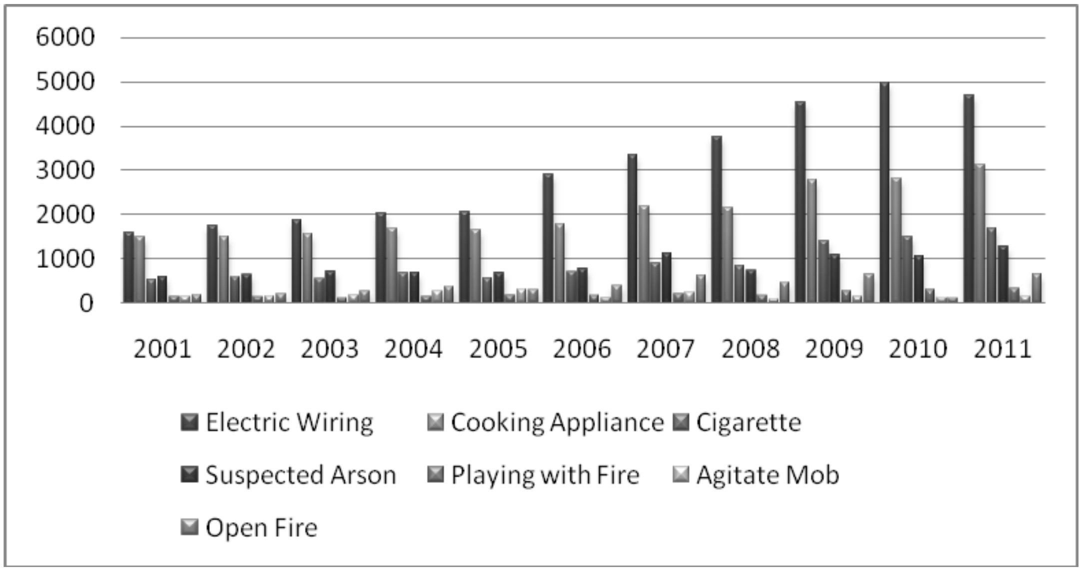

Figure 4: Causes of Fire Incidents (2001-2011)

4.2 Capability of Fire Department

The capability and efficiency of BFCD is quite vulnerable that was observed from the scenario of field survey and previous study. From an study of 2008, there were only 13 fire stations with an average of 35 employees along with two vehicles(one water tendon another is pump) for each station in Dhaka city (Monowarul and Neelopal, 2008). The present scenario is not most reliable at all. In case of the efficiency of fire department the recovery of resources were 104, 116, 279 crore taka whereas the losses were 306, 326, 554 crore taka from 2009 to 2011(figure 5). This values signify the certain portion of efficiency at a percentage rate of 25%, 26%, 33.5% during the years. In the year of 2007 and 2008 the recovery rate of resources by fire department were about 67% where in the year from 2003 to 2006 the efficiency of fire department were also reasonable with a rate of approximately 50%.

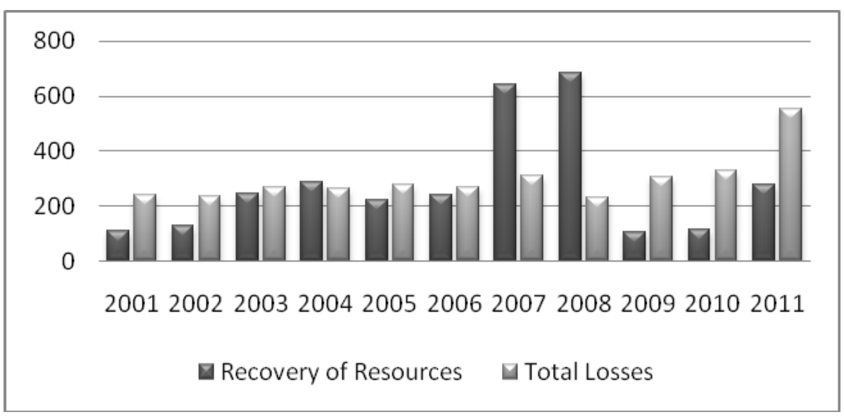

Figure 5: Efficiency of Fire Services.

The instrumental cost for fire department 33, 39, 47, 38 crore taka for the year of 2007-08 to 2010-11(figure 6). In the comparison of the costs to the recovery rate the ratio varies from 0.05 to 0.2.The number of manpower and equipments are also very insufficient compared to the present population of Bangladesh especially in the metropolitan cities like Dhaka, Chittagong, Sylhet.

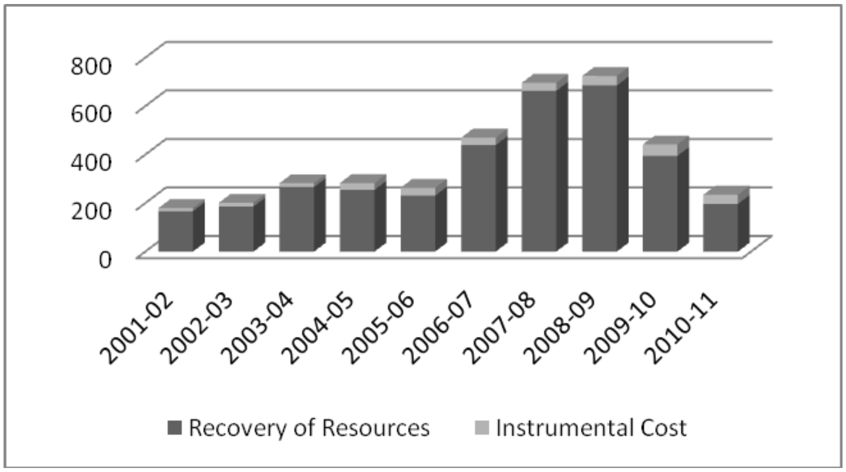

Figure 6: Instrumental Cost in Different Fiscal year

In 2001, there were only an average of 35 fire fighters and two vehicles for each station of Dhaka city. That means, each fire station has to serve almost 1 million people of the city where the ratio of the citizen and Manpower is 26,905: 1 and the ratio of the citizen and vehicle is 470,833: 1(Monowarul and Neelopal, 2008). In the year of 2011 the facilities of fire department in Dhaka city are summarizes below :

Table 1: Comparison of Fire Department's Facilities

Types of Station	Number of Station	Manpower	Vehicles	No of Population be Served	Ratio of the Citizen		
					Station	Manpower	Vehicle
1st	12	35	07	11.2 million	0.93: 1	0.32:1	1.6:1
2nd	11	27	05	4.6 million	0.542:1	0.17:1	0.92:1
3rd	08	26	03	1.75 million	0.22:1	0.067:1	0.58:1

There is only one first class station to serve about one million people where there is only one vehicle to serve about 1.6 million people in Dhaka city (table 1). As a whole there are only 31 stations to serve a population of about 1.75 million (table 1). This is representative figure of the country's most developed city. So the other scenario of country might be predicted to be worse than Dhaka city. The under construction project of fire station is only two, one of which is in Khilgaon and another one is in Dhamrai of Dhaka city. There are four project under land acquisition. So the situation has no chance to increase within a short time.

5. FIRE RISK ASSESSMENT

Fire risk can be defined as the product of the occur rence likelihood and the expected consequence for each fire scenario (Sekizawa, 2005). For fire r isk for life safety, the expected consequence is the expected number of casualties for each fire scenario. Therefore, the objective of fire risk for life safety is to determine the occurrence likelihood and the expected number of casualties for each fire scenario. Designing proper fire scenarios is essential for performance-based fire protection design. For the reason that there are many factors influencing fire spread and smoke movement, such as the fuel characteristics, the building geometry and fire protection systems, various fire scenarios may occur. For a specific building, the operational reliability of fire protection systems plays an important role in the occurrence of different fire scenarios (Chu, 2007).

5.1 Model Building Overview

Prior to the fire risk assessment, it is significant to define the characteristics of the model building. Two approaches were carried out to define a base building for the analysis. These included:

1. Site visits to the existing high-rise buildings in Dhaka, November 2011.
2. Review of high-rise building plans (table 2).

Table 2: Sample Building Parameter

Parameter	Values	
Area	8500 ft^2	
Height of Each Floor	11.5 ft	
Total Height	80.5 ft	
No. of Exists	3	
Occupant Number (Person per floor)	**Market Spaces**	150
	Offices	67
	Residential Apartments	48

5.2 Event Tree Analysis

With respect to consider the fluence of the operational reliability of fire protection systems on di fferent fire scenarios, event tree method is employed. Since the majority of casualties ifire is caused by inhalation of smoke, four basic events influencing smoke movement are considered: Smoke detection system (SDS) , Sprinkler system (SS), Manual Fire Service Suppression, which is further separated into two parts: Firefighters' action time (FAT) and Manual Fire Fighting(MFF) and Fire Resistance Rating(FRR)

5.2.1 *Structuring the Event Tree Branch Logic*

The event tree logic is read off from the source (A) on the left hand side, through the pathways (B - F). There are two outcome segments, Yes/No, for each pathway factors, where 'Yes' implies success and 'No' implies failure. The probability of the various consequences is then calculated by multiplying together the various branch probabilities of each factor. The pathway factors of the event tree are developed as follow:

- Smoke detection system (SDS) - Success/ Failure
- Sprinkler system (SS) - Success/ Failure or not installed
- Manual Fire Service suppression, which is further separated into two parts:
  -Firefighters' action time (FAT): the time between when the Fire Service is alerted and when the firefighters start to fight the fire is less than 30 minutes - Success/ Failure
  -Manual Fire Fighting (MFF) - Success/ Failure
- Fire Resistance Rating (FRR) - Success/ Failure.

The event tree is shown as Figure 7

Fire Ignition(A)	SDS(B)	SS(C)	FAT(D)	MFF(E)	FRR(F)	Scenario

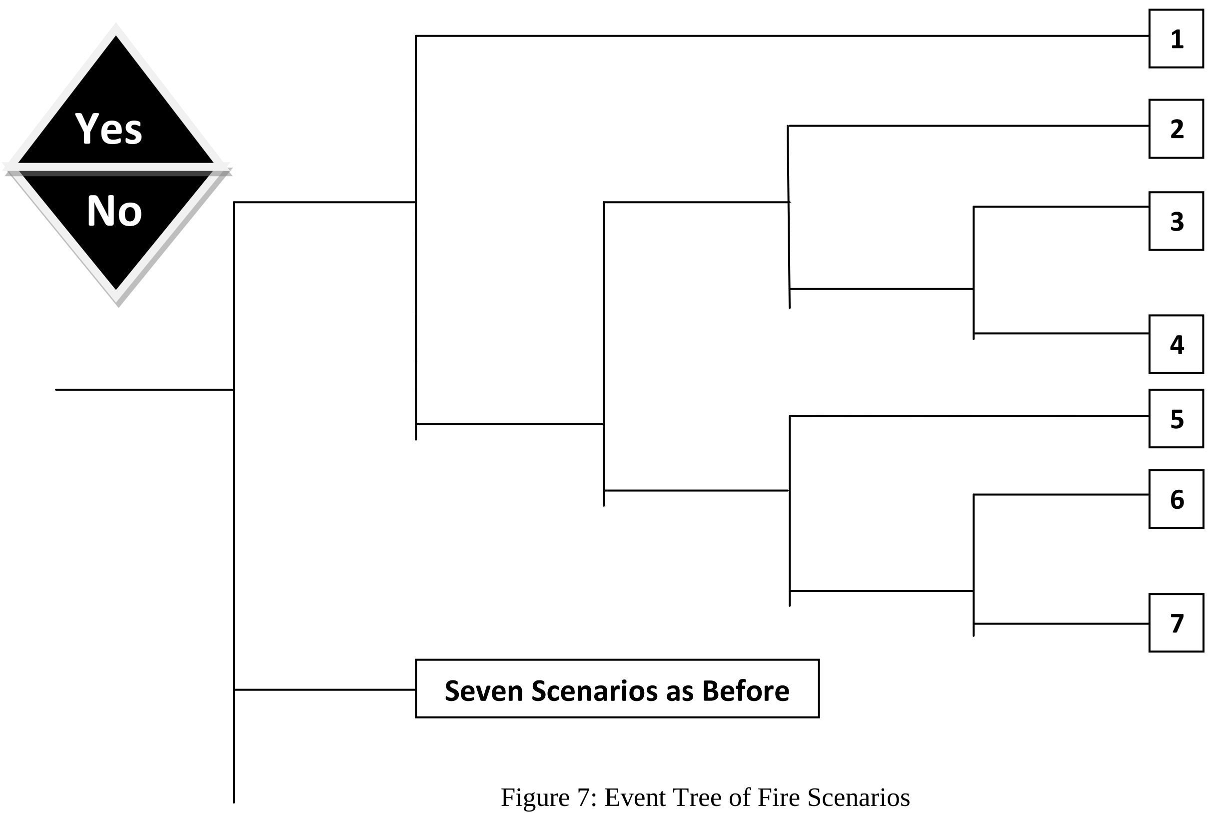

Figure 7: Event Tree of Fire Scenarios

Where

SDS = Smoke detection system (B); SS = Sprinkler system (C); FAT = Firefighters' action time (D); MFF = Manual fire fighting (E); FRR = Fire Resistance Rating (F)

5.2.3 *System Reliability Data*

Fire Fighter Action Time (FAT) may vary from ten minutes to two hour maximum according to the experiences of BFCD officials. In most of the cases it is unable to start fire fighting within thirty minutes (as stated above) due to traffic congestion, narrow road wide, system failure etc. There were no official records about the time to start the fire fighting. According to the view of fire service officials the success rate to make FAT within 30 minutes may reach maximum of only 40% of accidents per year.

Table 3: System Reliability Data (BFCD officials; Kwok-Lung Ng, 2006; Lu S. et.al., 2011)

Fire Protection Systems	Minimum	Average	Maximum
Automatic Smoke Detection	0.70	0.72	0.74
Manual Smoke Detection	0.60	0.70	0.80

Sprinkler System	0.88	0.93	0.98
FAT-Success	0	0.21	0.40
FAT-Failure	0.60	0.79	1.00
MFF	0.40(FAT<30 min)	0.51(FAT<30 min)	0.60(FAT<30 min)
	0.15(FAT>30 min)	0.225(Fat>30 min)	0.30(FAT>30 min)

In case of Manual Fire Fighting (MFF) the success rate depends on the availability of modern equipments, availability of professional fire fighters and heat, smoke and fire load density of fire etc. The success rate vary from only 15-30% without any life damage according to the view of high officials of BFCD (table 3). If the FAT within 30 minutes can be maintained properly success rate increases to 40-60% with lower availability of modern equipments (table 3).

5.2.4 *Fire Resistance Rating (FRR)*

The selection of the fire resistance rating (FRR) for building construction often depends on the fire load density, ventilation factor and conversion factor, as mentioned in the Fire Engineering Design Guide (Proter A, 2003). In order to estimate the probability of the Fire Resistance Rating (FRR) it is necessary to determine the probability of the equivalent time of fire exposure. If the equivalent time of fire exposure is over the provided FRR, the wall barrier is considered to have failed to maintain the construction integrity. The elaborate equations for calculating the equivalent time of fire exposure are described in the Fire engineering design guide (Proter A, 2003). The reliability data of different types of FRR construction are based on the different studies like Kwok-Lung Ng A(2006), Lu S. et.al.(2011) (table 4).

Table 4 : Reliability Data of Fire Resistance Rating (FRR)

FRR construction	Success/Failure	Wood or Wooden façade	Foam	Cloths	False ceiling
30 minute	Success	0%	0%	15%	0%
	Failure	100%	100%	85%	100%
1 hour	Success	0%	08%	26%	0%
	Failure	100%	92%	74%	100%
2 hour	Success	22%	52%	76%	26%
	Failure	78%	48%	24%	74%
3 hour	Success	65%	88%	92%	72%
	Failure	35%	12%	08%	28%

5.3 Risk Estimation

Risk estimation is a process for assigning the frequencies and consequences of the hazardous event into various levels of risk. Monte Carlo simulations for the fire risk estimation were conducted using @Risk5.7. In the simulations, the settings generally follow the default choice except for the sampling type and the number of iterations. Consequence of fire can generally be categorized into one of the followings:

1. Property damage
2. Life safety exposure

Due to lacking of the available data of statistical life values of Bangladeshi people, the estimation were conducted by taking property damages into consideration. The property losses are estimated for equivalent risk calculation of each scenario. Due to unavailability of proper data of previous years the property losses from the year of 2008 to 2011 are estimated for risk calculation. Total property losses in this four years are 132.934 crore taka per thousand fire accidents (table 6).

Table 6: Consequences Calculation for Each Scenario of Event tree

Event No	Property losses per 1000 fire(in crore taka)	Property losses per fire(in crore taka)	Event No	Property losses per 1000 fire (in crore taka)	Property losses per fire (in crore taka)
1	8.906	0.008906	8	11.9	0.0119
2	21.934	0.021934	9	26.6	0.0266
3	21.934	0.021934	10	26.6	0.0266
4	28.04	0.02804	11	35.4	0.0354
5	21.934	0.021934	12	26.6	0.0266
6	21.934	0.021934	13	26.6	0.0266
7	28.04	0.02804	14	35.4	0.0354

5.3.1 *Probability distribution of Risk*

A sample risk estimation are shown here for 30 minutes FRR construction and the wooden façade materials. The simulation result of @Risk 5.7 software package is also shown for the conveniences of interpretation.

Table 7: Sample Risk Estimation Procedure

Scenario 1 : 30 minutes FRR, Wooden Façade fire								
Outcome Event No	**SDS**	**SS**	**FAT**	**MFF**	**FRR**	**Probabilities of each outcome**	**Consequence (loss in Taka)**	**Risk/incident**
1	0.505	0.93	------	-----	1.00	0.469	0.008906	0.00417
2	0.505	0.07	0.203	0.503	1.00	0.00361	0.021934	$7.92 * 10^{-5}$
3	0.505	0.07	0.203	0.497	0.00	0.000	0.021934	0.000
4	0.505	0.07	0.203	0.497	1.00	0.00357	0.02804	$1.00 * 10^{-4}$
5	0.505	0.07	0.797	0.225	1.00	0.00634	0.021934	$1.39 * 10^{-4}$
6	0.505	0.07	0.797	0.775	0.00	0.000	0.021934	0.000
7	0.505	0.07	0.797	0.775	1.00	0.022	0.02804	$6.169 * 10^{-4}$
8	0.494	0.93	-----	-------	1.00	0.459	0.0119	$5.462 * 10^{-3}$
9	0.494	0.07	0.203	0.503	1.00	0.00353	0.0266	$9.3898 * 10^{-5}$
10	0.494	0.07	0.203	0.225	0.00	0.000	0.0266	0.000
11	0.494	0.07	0.203	0.497	1.00	0.05	0.0354	$1.77 * 10^{-3}$
12	0.494	0.07	0.797	0.225	1.00	0.0062	0.0266	$1.65 * 10^{-4}$
13	0.494	0.07	0.797	0.775	0.00	0.000	0.0266	0.000
14	0.494	0.07	0.797	0.775	1.00	0.0214	0.0354	$7.576 * 10^{-4}$
					Total	1.0000	**Total**	0.013293
Note that the values above only shows the mean values of the distributions								

5.3.2 *Simulation Result of @Risk 5.7*

A sample simulation result is shown here for the values of table 1. The result obtained here shows the 90% confidentiality for the result obtained from risk estimation.

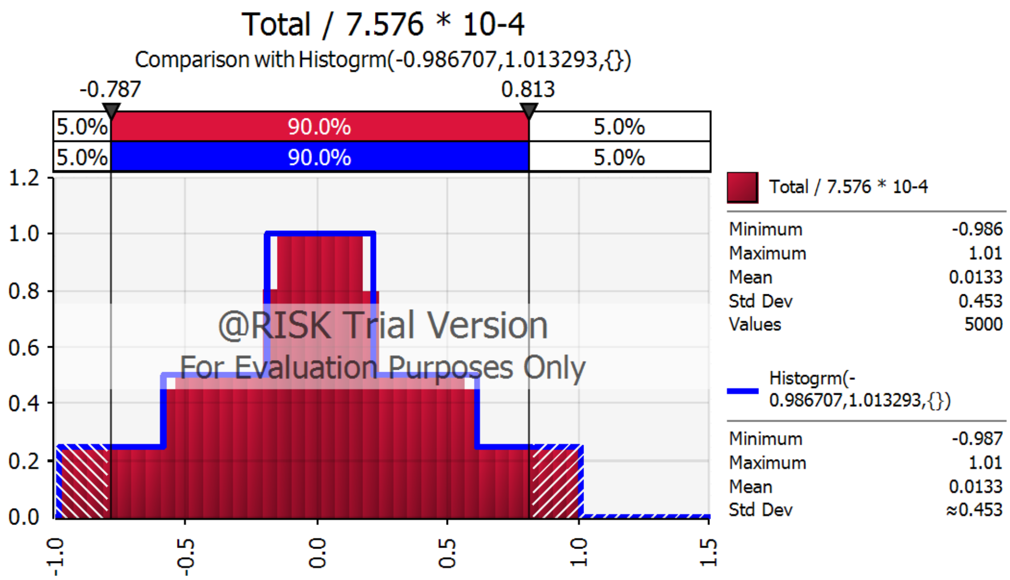

Figure 08 Simulation results @Risk 5.7 based on the values of table 7

5.4 Results of Risk Assessment

The result of the risk estimation concludes that the 30 minutes FRR construction for combustible material of gypsum board (false ceiling) make reasonably low risk of fire. Overall, it is found that scenarios with a sprinkler system, such as Scenario 6 to Scenario 10, would generally have higher benefit values than scenarios with no sprinkler protection, such as Scenario 1 to Scenario 5. As the results of the risk analysis, Scenario 6 is found to be the economically best option and followed by scenarios having a higher FRR construction. In the scenario with sprinkler system, it is found that the higher FRR construction would have the lower risk value but due to the high construction and system installation costs it is found to be uneconomical. In case of scenario with 3 hour FRR construction with sprinkler installation considered to be the best option with respect to property damages. Though the passive protection system like FRR might be considered as low risk option but due to the lower standards of subsequent measures of fire safety like system implementation, fire department responses; the effectiveness of FRR might be lower.

6. DISCUSSION

Based on the study the problems related to the fire safety management are categorized as follows: (a) Constraint of Fire department to minimize risks of fire, (b) Constraint due to building construction (Passive protection), (c) Constraints due to improper safety in building system.

<u>Constraints of Fire department</u>

This constraints include lower number of manpower and modern equipments, lower number of station provided for serving a largest portion of population, late response of fire department due to traffic congestion and narrow wide of streets, and lacking in the governmental budget for fire department.

<u>Constraints due to building construction</u>

1. Significant lacking in the construction practices to maintain the requirements of BNBC code.
2. Faulty architectural plan for fire safety also be witnessed in our field survey especially in case of the position, height and orientation of atrium in commercial mixed use building.
3. Larger amount of combustible materials (not intentionally) to decorate the office, market spaces which have a large amount of fire load with longer duration of heat release rate (HRR).

<u>Constraints due to improper safety in building system</u>

The observation of the field survey shows that the practices for fire safety installment are not complete to take the clearances of authorities. In several cases modern equipments are installed but the people are not quite aware of the uses of this equipments or they are not trained enough.

7. CONCLUSION

The study shows that the fire department of Bangladesh is not well equipped and the facilities is also not up to the standards. Obviously the procedure followed for fire safety management has been lacking with number of bottlenecks such as the failure of the effectiveness of passive protection of fire safety like FRR construction due to improper system installment with improper response of fire department, the unawareness for the uses of fire safety installment due to lack of proper trainings of occupancies of the building, the failure of atrium fire control in recent fire accidents in Bangladesh due to the faulty architectural plan especially in case of the height, orientation and shape of the atrium in the building, and the deficient code of practices in the improper passive protection system installation in different occupancy types of building.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

ANALYYICAL STUDY ON SLENDER PARTIALLY ENCASED COMPOSITE COLUMNS UNDER ECCENTRIC LOADING

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ABSTRACT

This paper presents the behaviour of slender partially encased composite columns under eccentric axial load causing symmetrical single curvature bending. The load versus deflection response of slender partially encased composite column is formulated using the Newmark's iterative procedure. A parametric study is conducted using this method to identify the potential variables that can significantly affect the behaviour of slender partially encased composite column. The geometric properties that can greatly affect the behaviour of PEC columns include the column cross-sectional dimensions, length of the column, longitudinal spacing of the transverse links, thickness of the steel flange and web plates, and initial load eccentricity. The effects of these parameters on the axial capacity and second order deflection of the slender column is studied. The axial capacity of a partially encased composite columns are found to decrease significantly as the overall slenderness ratio increases, particularly for columns with slender plates. The load eccentricity ratio also has a significant impact on the capacity and deflection of these columns. Besides, a reduction in bearing capacity has been found with increasing the flange slenderness ratio. On the other hand, spacing link-to-depth ratio has no effect on the axial capacity and lateral deflection of slender column. The results are presented in detail in the paper.

Keywords: *Steel, concrete, composite, slender columns, eccentric loads, axial capacity, deflection.*

1. INTRODUCTION

An outstanding feature of efficient, innovative structures is very often an ideal combination of various different building materials. The favourable combination of steel, with its high tensile strength and ductility, and concrete, with its high compressive strength and good resistance to corrosion, has long been recognized in structural concrete construction. With the method of composite construction, it is now possible to combine the positive features of steel construction and structural concrete, without having to accept the drawbacks. A steel concrete composite column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally used as a load-bearing member in a composite framed structure. Typical cross-sections of composite columns with fully and partially encased steel sections and concrete filled tubular sections are illustrated in Fig. 1. Steel-concrete composite columns are very effective in providing the required stiffness to limit the lateral drift of the building to the acceptable level as well as to resist the lateral seismic and wind loads. The introduction of steel rolled shapes and high strength concrete has made it possible to design columns of large slenderness.

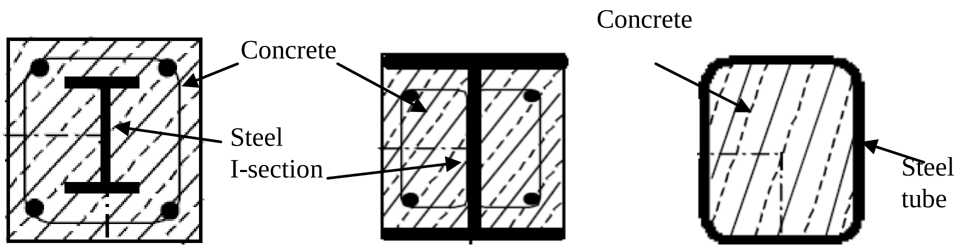

Figure 1: Typical cross-sections of (a) Fully encased composite column (FEC), (b) Partially encased composite column (PEC) and (c) Concrete filled tubular sections (CFT).

Partially encased composite (PEC) columns consisting of thin walled built up steel section with concrete infill cast between the flanges, is a relatively new concept in composite construction. Transverse links are provided between the flanges at regular intervals to enhance the resistance to local instability of the thin steel plates.

Typical cross-section and 3D view of the steel skeleton of a PEC column is shown in Fig. 2. This innovative composite system not only reduces the cost of construction using relatively low-cost concrete by minimizing the use of higher cost steel, but also helps to overcome the complexities related to erection and design of connections of more commonly used composite columns. Several research works including both numerical and experimental works (Fillion (1998); Tremblay et al. 1998; Bouchereau & Toupin (2003); Prikett & Driver (2006); Begum et al. 2007; Chicoine et al. 2002) have been carried out for establishing the behaviour and the design provisions for this new type of composite column under various loading conditions. Most of these research works were confined in exploring the short (length-to-depth ratio of 5) column behaviour of PEC columns. However, a few long column tests (length-to-depth ratio of 20) were carried out by (Chicoine et al. 2002) under static loading. This test database is not sufficient to establish a design guideline for slender PEC columns. Therefore, extensive research work is required to fully understand the behavior of slender PEC columns under eccentric loading.

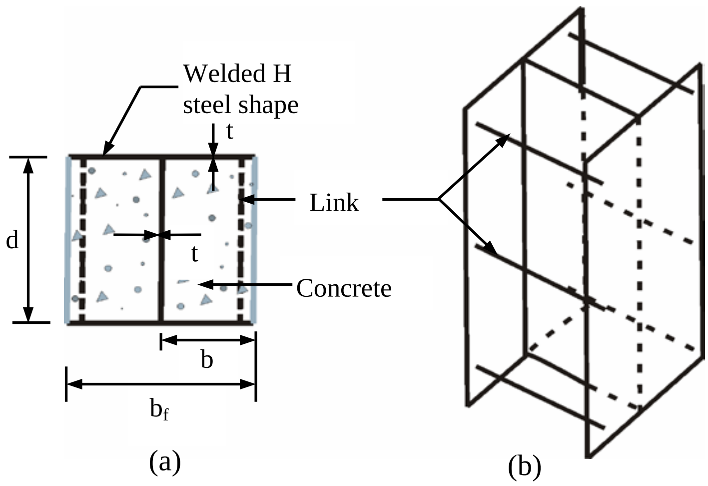

Figure 2: Partially encased composite columns, (a) cross section and (b) 3D view of the steel configuration.

2. OBJECTIVES AND SCOPE

The primary objective of this study is to investigate the structural behaviour of slender partially encased composite columns subjected to eccentric axial loading. To this end, Newmark's numerical iterative procedure is used to develop the load deflection curves of partially encased composite columns subjected to symmetrical single curvature bending about major axis of the column cross-section. This procedure is then used to determine the influence the overall column slenderness ratio and load eccentricity ratio of the axial capacity and lateral deflection of the column. A parametric study is conducted on four references PEC columns with variable overall slenderness ratio and load eccentricity ratio. Five different slenderness ratios—10, 15, 20, 25 and 30—has been employed in the parametric study to cover the short, intermediate and a wide range of slender columns. The load eccentricity ratios used in this study are 0.1, 0.2, 0.3, 0.4 and 0.5. The flange slenderness ratio has been selected as 25, 30 and 35. Finally, link spacing-to-depth ratio has been taken as 0.5 and 0.7. The effects of these parameters were studied on 450 mm × 450 mm cross-section. The parametric columns were analysed under monotonic loading conditions with bending about the strong axis.

3. LOAD DEFLECTION RESPONSE OF SLENDER COLUMN

The behaviour of a slender column of length L, subjected to eccentric loading is greatly influenced by the second order bending moment at midheight resulting from the deflection due to applied eccentric loading. The resulting deflection at mid height of the column is termed as second order deflection. For columns having a small slenderness ratio this second order deflection is negligible. However for slender columns this value becomes significant and controls the maximum moment of the column. Consequently, the bending moment strength of a slender column subjected to eccentric axial load is much lower than that of its cross-section. The second order deflection therefore plays a prominent role in identifying the strength of slender PEC columns. Newmark's numerical iterative procedure is implemented to compute this second order deflection for slender PEC columns under symmetrical single curvature bending for a given axial load and applied eccentricity.
In this study a pin ended PEC column of length L, subjected to eccentric axial loading as shown in Fig. 3(a), is selected. The bending moment diagram of the column is shown in Fig. 3(b). The bending moment diagram has

two parts – a constant moment (Pe) from the applied eccentricity and a variable moment (PΔ) resulting from the deflection (Δ) of the column from its original position.

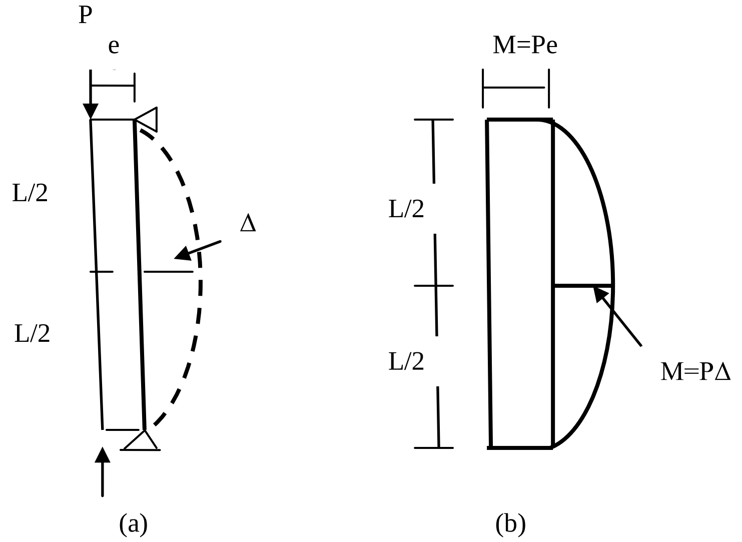

Figure 3: (a) Column subjected to eccentric loading (b) Bending moment diagram of the column

Newmark's method (Newmarks (1943)) is used to determine the equilibrium configuration for a given combination of axial load and end moments that were applied to column. The column is subdivided into segments or stations of equal length for which initial deflections have been assumed based on the applied end moments. The first-order moments and the second-order moments due to slenderness effects are computed and summed at each station. The curvature corresponding to the total moment at each station are retrieved from the cross section moment-curvature curve for the given axial load level in order to define the distribution of curvature along the column length. Then the conjugate beam method is used to compute the deflection at each of the stations. If the computed deflections and the initial deflections are within prescribed limits of 0.05%, an equilibrium solution is obtained. If not, the computed deflections are substituted for the assumed deflections and the process is repeated until the deflections converged. For each value of axial load, similar procedure is adopted. Thus for a specific eccentricity, the load deflection curve is plotted. Using the value from load deflection curve, moment at mid height of composite slender column is determined by multiplying the axial load to the summation of assumed eccentricity and second order deflection at mid height of column. In case of drawing load deflection curve at different slenderness ratios, eccentricity of column has been fixed and length of column has been varied and the above procedure has been followed.

From the load deflection curve, the ultimate capacity of column is determined by calculating the moment from deflection. In connection with it, for the specific cross-section, interaction diagram of the column is drawn using the strain compatibility relationship as applied for reinforced concrete columns. From the load moment interaction diagram, for a specific eccentricity, angle of straight line intersecting the interaction diagram, is determined. The corresponding point represents the axial load capacity of the column at a specific moment and eccentricity. The value of moment is noted. To determine the axial load capacity of a composite slender column, this value of moment is considered as the moment capacity of the column. Both values of axial load and mid height deflection are assumed and trial was done using Newmark's formula for several times to obtain the desired moment. The trial from which the required value of the cross-sectional moment is achieved has been noted. This axial load is the ultimate axial load capacity of the composite slender column at the specific eccentricity. Similar procedure is followed for determining the axial load capacity at other eccentricities. The load eccentricity of the column is then fixed and length of column has been varied and the procedure described above has been followed to determine capacity of column at different slenderness ratios.

4. DESIGN OF PARAMETRIC STUDY

The potential variables that can significantly affect the behaviour of slender partially encased composite columns are the overall column slenderness ratio, flange plate slenderness ratio, load eccentricity ratio and link spacing. This paper demonstrates the effect of the overall column slenderness ratio and load eccentricity ratio in combination with flange plate slenderness ratio and link spacing. The column cross-section was fixed at

450mmx450mm. Four reference columns were designed with variable plate thickness and link spacing. The properties of these columns are shown in Table 1. In each of these columns the overall slenderness ratio and load eccentricity ratio was varied. The global stability of the column is controlled by the overall slenderness ratio, which is defined as the ratio of the length of the column, L, to the depth of the column cross-section, d. Five different slenderness ratios— 10, 15, 20, 25 and 30—were employed in the parametric study to cover the range of short, intermediate and a wide range of slender columns. The load eccentricity ratios which can be obtained by dividing the initial eccentricity, e, of the applied axial load by the depth of the column cross-section, d, used in this study are 0.1, 0.2 0.3, 0.4 and 0.5. The flange slenderness ratio has been selected as 25, 30 and 35. Finally, link spacing-to-depth ratio has been taken as 0.5 and 0.7. The effects of the selected parameters on the load deflection response, axial capacity and deflection of PEC columns under single curvature bending about major axis are presented in the subsequent sections.

Table1: Geometric properties of the reference columns

Column	Width (b_f) (mm)	Depth (d) (mm)	Thickness(t) (mm)	Link Spacing (s) (mm)
P_a	450	450	9	225
P_b	450	450	9	315
P_c	450	450	7.5	225
P_d	450	450	7.5	315

4.1 Effect of load eccentricity (e/d) ratio

The behaviour of a PEC column under bending induced by an eccentrically applied axial load is found to be greatly affected by the initial load eccentricity ratio. Table 2 to 5 shows the effect of column load eccentricity (e/d) ratio on the axial capacity and mid-height deflection of the column. For column P_b with L/d ratio of 15, increasing in the e/d ratio from 0.1 to 0.2, 0.3, 0.4 and 0.5 reduces the ultimate load capacity by 13%, 24%, 33% and 41% respectively and increases the lateral deflection by 43%, 66%, 75% and 76% respectively with respect to the column with e/d ratio of 0.1. Again, for column P_b with L/d ratio of 25, increasing the e/d ratio from 0.1 to 0.2, 0.3, 0.4 and 0.5 decreases the ultimate load capacity by 7%, 14%, 20% and 27% and increases the deflection by 42%, 64%, 72% and 73% respectively. Both of the analysis results show that reduction in ultimate load capacity and increase in lateral deflection accelerated with the increase in load eccentricity ratio. For column P_d with L/d ratio of 15 and 25, for the previous increment, the ultimate load capacity was reduced by 14%, 24%, 34%, 42% and 7%, 13%, 20%, 27% respectively. At the same time, deflection was increased by 37%, 60%, 66%, 67% and 42%, 65%, 70%, 71% respectively. Similar to column P_b, ultimate load capacity was reduced and deflection was increased significantly with increasing the load eccentricity ratio.

Table 2: Effect of Load Eccentricity (e/d) Ratio of Column P_b at Slenderness Ratio 15

Column	e/d	L/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu

P_b	0.1				3850	103	_	_
	0.2				3340	148	13	43
	0.3	15	25	0.7	2945	172	24	66
	0.4				2580	181	33	75
	0.5				2260	182	41	76

Table 3: Effect of Load Eccentricity (e/d) Ratio of Column P_b at Slenderness Ratio 25

Column	e/d	L/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu
P_b	0.1				1815	271	_	_
	0.2				1680	384	7	42
	0.3	25	25	0.7	1565	445	14	64
	0.4				1450	466	20	72
	0.5				1330	469	27	73

Table 4: Effect of Load Eccentricity (e/d) Ratio of Column P_d at Slenderness Ratio 15

Column	e/d	L/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variationin Δu
P_d	0.1				3360	110	_	_
	0.2				2880	151	14	37
	0.3	15	30	0.7	2555	177	24	60
	0.4				2215	183	34	66
	0.5				1940	184	42	67

Table 5: Effect of Load Eccentricity (e/d) Ratio of Column P_d at Slenderness Ratio 25

Column	e/d	L/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu

P_d	0.1				1555	277	_	_
	0.2				1445	392	7	42
	0.3	25	30	0.7	1350	456	13	65
	0.4				1240	470	20	70
	0.5				1145	473	27	71

Figure 4 to 7 presents the relation between axial and mid-height deflections obtained by the proposed analytical method for the columns subjected to different initial load eccentricities and the eccentricity varies for e/d = 0.1 to 0.5 about the major axis. These columns have slenderness ratio of 15 and 25 and pinned at the ends were subjected to single curvature bending. It is clear that column capacity is strongly affected by the amount of eccentricity. As the eccentricity increases, the load-carrying capacity drops significantly with an increase in the mid height deflection.

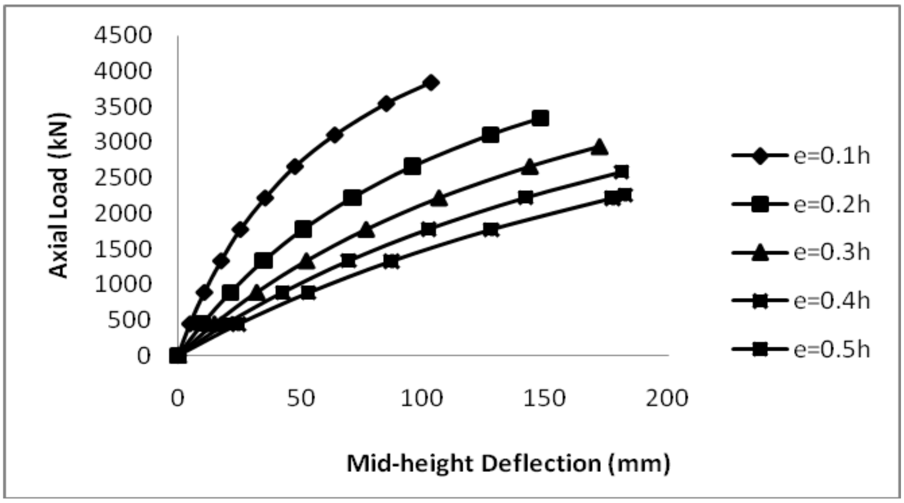

Figure 4: Load Deflection curve of P_b column with L/d ratio 15

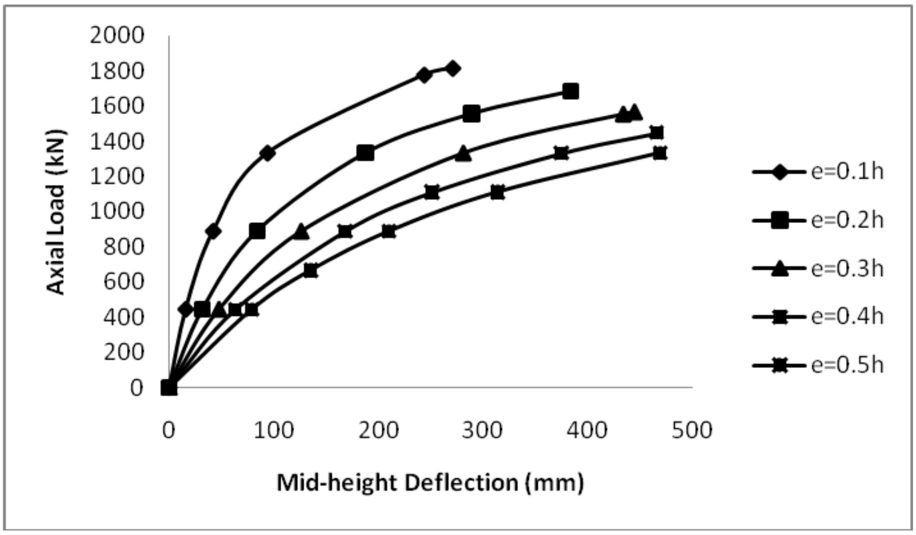

Figure 5: Load Deflection curve of P_b column with L/d ratio 25

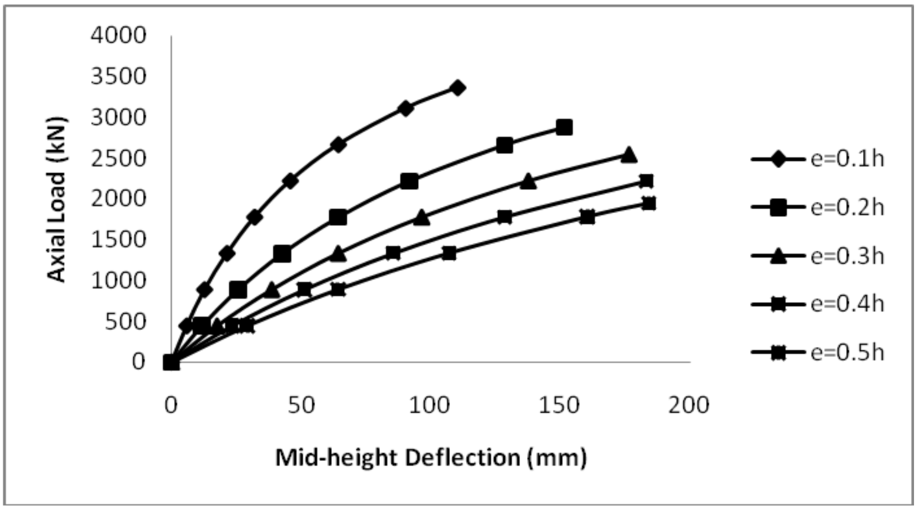

Figure 6: Load Deflection curve of P_d column with L/d ratio 15

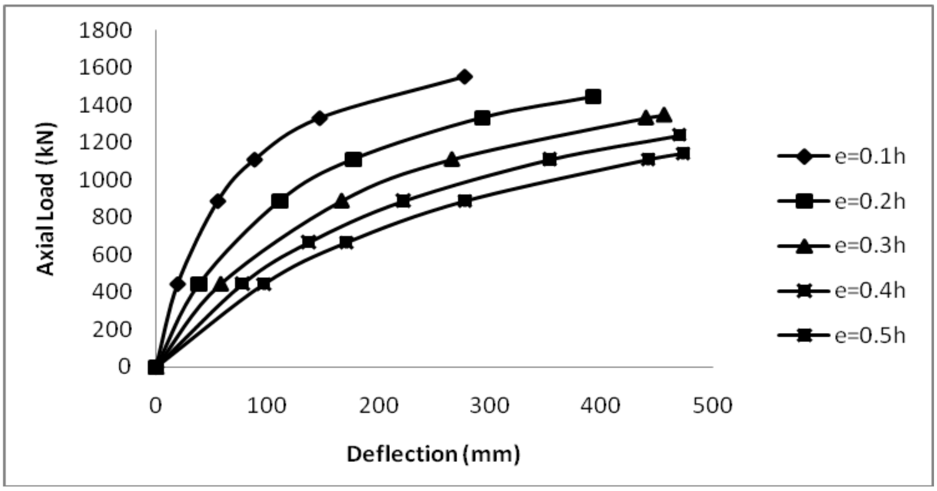

Figure 7: Load Deflection curve of P_d column with L/d ratio 25

4.2 Effect of overall column slenderness (e/d) ratio

The global stability of the column is controlled by the overall slenderness ratio, which is defined as the ratio of the length of the column, L, to the depth of column cross-section, d. In the parametric study five different slenderness ratios- 10,15,20,25 and 30- were employed. Tables 6 to 8 show the effect of overall column slenderness (L/d) ratio on the selected output parameters at the peak load point. For column P_a, if L/d ratio is increased by 5, 10, 15 and 20 the ultimate load capacity is reduced by 33%, 54%, 67% and 75% respectively and the lateral deflection is increased by 112%, 261%, 450% and 680% respectively. Again, for column P_b, an increase in the L/d ratio by 5, 10, 15 and 20 decreases the ultimate load capacity by 33%, 53%, 66% and 75% and increases the deflection by 112%, 261%, 449% and 677% respectively which are almost similar to column P_a. Both of the analysis results show that reduction in ultimate load capacity and increase in deflection accelerated with the increase in slenderness ratio. For column P_a and P_b, main distinguishing feature is the spacing of transverse links. The ultimate load capacity variation rate is observed to be similar for these columns with the applied variation in the overall slenderness ratio. Therefore, the effect of the spacing of the transverse links can be taken as insignificant for slender columns. For column P_d, an increase in the L/d ratio by 5, 10, 15 and 20 reduces the ultimate load capacity by 26%, 46%, 59% and 68% respectively. Again, lateral deflections increased by 113%, 262%, 446% and 669% respectively. The ultimate load capacity reduction rate is higher for column P_d as compared to columns P_a or P_b. This is due to the presence of slender flange plates of column P_d.

Table 6: Effect of Slenderness (L/d) Ratio of Column $P_{a.}$

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variationin Pu	% Variation in Δu
P_a	10				5130	74	_	_
	15				3425	156	33	112
	20	0.2	25	0.5	2370	267	54	261
	25				1705	406	67	450
	30				1275	577	75	680

The effects of overall column slenderness ratio on PEC columns at a fixed load eccentricity ratio are presented in Figures 8 to 10. The load versus deflection responses for the columns with five different slenderness ratios 10 to 30, subjected to an initial load eccentricity ratio of 0.2 and 0.4 is shown. From these figures it is clear that increase in the slenderness ratio increases the lateral deflection at mid height with a significant reduction in the load bearing capacity. It also shows that for L/d ratio 10, curve shows linear pattern. It implies that, at this length of column, initial deflection is more dominant than secondary deflection. At other slenderness ratios, nonlinear curves have been found and deflection at mid-height of column increases exponentially with the increase in the slenderness ratios.

Table 7: Effect of Slenderness (L/d) Ratio of Column $P_{b.}$

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variationin Δu
P_b	10				4950	70	_	_
	15				3340	148	33	112
	20	0.2	25	0.7	2330	253	53	261
	25				1680	384	66	449
	30				1260	543	75	677

Table 8: Effect of Slenderness (L/d) Ratio of Column $P_{d.}$

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu
P_d	10				3020	86	_	_
	15				2220	183	26	113
	20	0.4	30	0.7	1650	311	46	262
	25				1240	470	59	446
	30				960	662	68	669

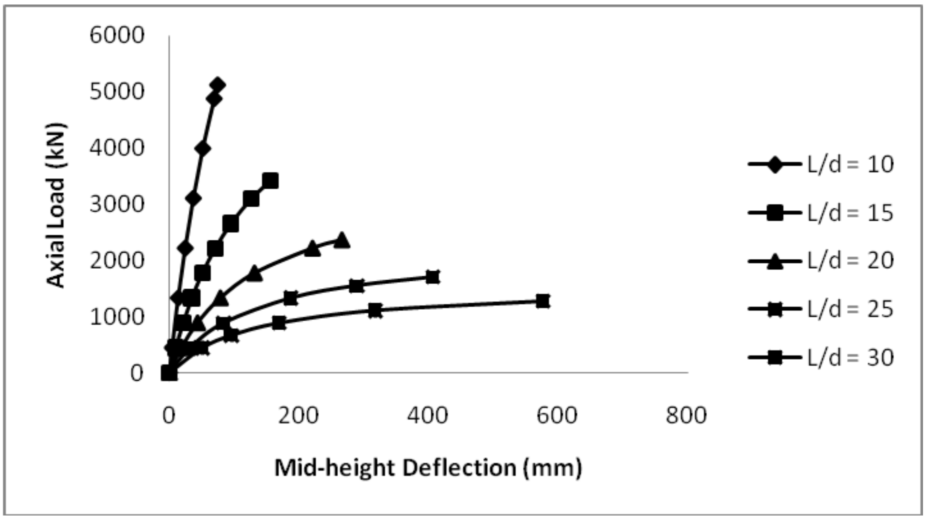

Figure 8: Load Deflection curve of P_a column with e/d ratio 0.2

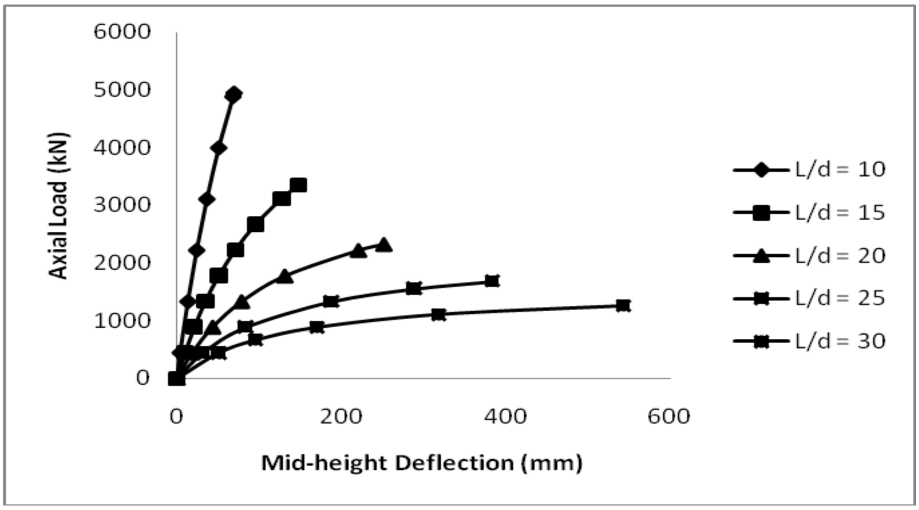

Figure 9: Load Deflection curve of P_b column with e/d ratio 0.2

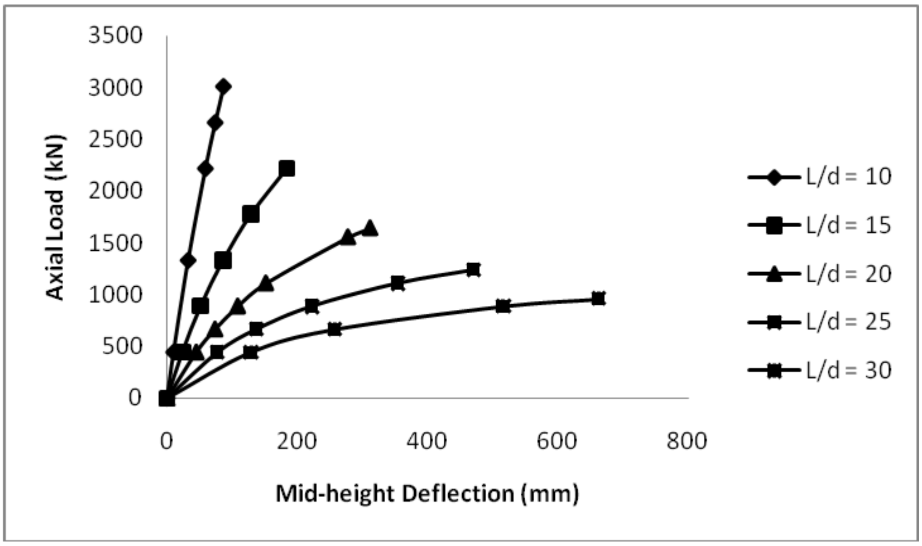

Figure 10: Load Deflection curve of P_d column with e/d ratio 0.4

4.3 Effect of flange plate slenderness (b/t) ratio

The flange plate slenderness ratio is defined as the ratio of the half-width of the flange, b, to its thickness, t. This parameter was varied between 25 and 35, with an intermediate value of 30. The ultimate capacity of a PEC column is significantly affected by this parameter, since it controls the occurrence of local instability in the flange plate of the column. Table 9 and 10 shows the effect of flange plate slenderness (b/t) ratio on the behaviour of slender partially encased composite column. Increasing b/t from 25 to 30 and 25 to 35 causes a reduction of 11% and 19% respectively in the axial load capacity of PEC column, P_a for the link spacing of 0.5. The average reduction in the axial capacity is 10%. At the same time, lateral deflection is increased by 16% and 32% respectively. Again, for column P_b, an increase in the b/t ratio by 5 and 10 decreases the ultimate load capacity by 12% and 22% and increases the deflection by 11% and 20% respectively which are almost similar to column $P_{a.}$ Effect of varying slenderness of column and load eccentricity ratio has been found negligible when flange plate slenderness ratio has been varied. Both of the analysis results show that reduction in ultimate load capacity and increase in deflection accelerated with the increase in flange plate slenderness ratio.

The effects of flange plate slenderness ratio on PEC columns at a fixed load eccentricity ratio and slenderness ratio are presented in Figures 11 and 12. The load versus deflection curves for the columns with three different flange plate slenderness ratios 25 to 35, subjected to an initial load eccentricity ratio of 0.2 and 0.3 and slenderness ratio 20 and 25 is shown. From these figures it is clear that increase in the flange plate slenderness ratio increases the lateral deflection at mid height and load carrying capacity is reduced considerably.

Table 9: Effect of Flange Plate Slenderness (b/t) Ratio of Column P_a

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu
P_b			25		2370	267	_	_
	20	0.2	30		2120	310	11	16
			35	0.5	1920	352	19	32

Table 10: Effect of Flange Plate Slenderness (b/t) Ratio of Column P_b

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu
P_b			25		1570	445	_	_
	25	0.3	30		1400	516	11	16
			35	0.7	1260	587	20	32

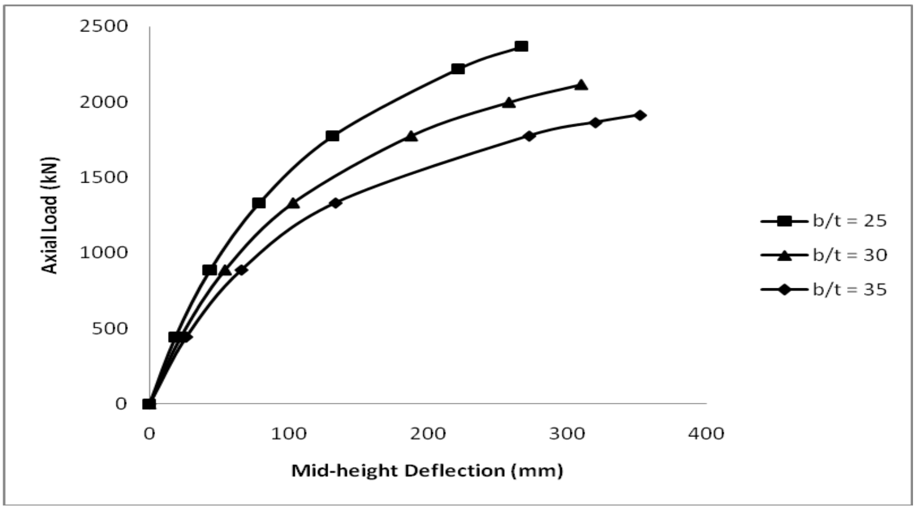

Figure 11: Load Deflection curve of P_a column with e/d 0.2 and L/d 20

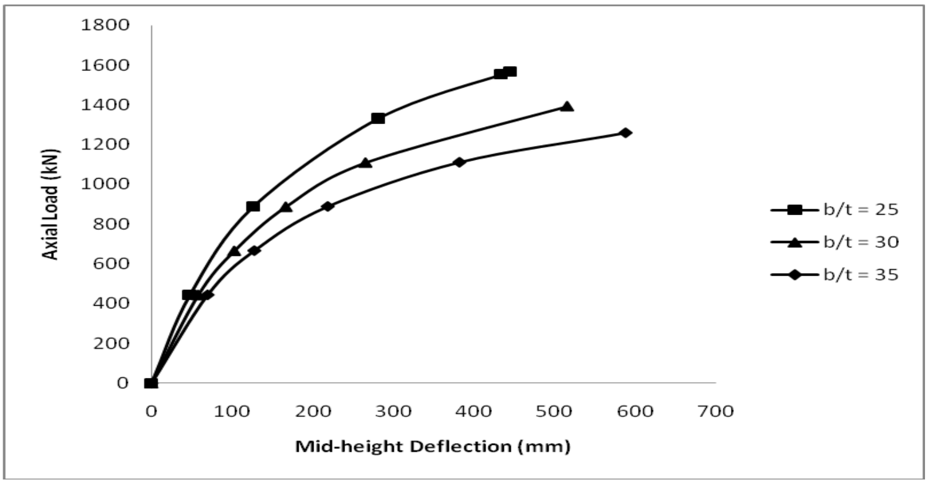

Figure 12: Load Deflection curve of P_b column with e/d 0.3 and L/d 25

4.4 Effect of link spacing to depth (s/d) ratio

Local flange buckling in a PEC column takes place in the unsupported length of the flange plate, i.e., in the flange panel between two successive links. Therefore, link spacing is clearly an important parameter affecting the behaviour of these columns. The effect of link spacing is studied by varying the ratio of link spacing, s, to the column cross-section, d. Two values of s/d ratio 0.5 and 0.7 have been studied in the parametric study. As shown in Table 10, changing the s/d ratio from 0.5 to 0.7 has no effect on the peak load. The axial capacities of slender columns are not affected at all by the s/d ratios and corresponding lateral deflection is also negligible.

Table 10: Effect of link spacing-to-depth (s/d) Ratio of Column P_b

Column	L/d	e/d	b/t	s/d	Pu (kN)	Δu (mm)	% Variation in Pu	% Variation in Δu
P_d	15	0.2	30	0.5	2880	152	_	_
				0.7	2880	152	Nil	Nil

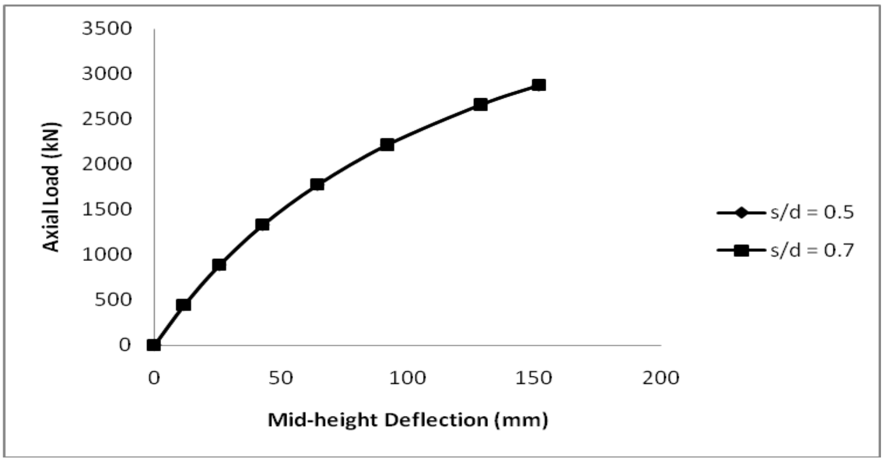

Figure 13: Load Deflection curve of P_d column with e/d 0.3 and L/d 15

The effects of link spacing-to-depth ratio on PEC columns at a fixed load eccentricity ratio and slenderness ratio are presented in Figures 13. The load versus deflection curves for the columns with two different link spacing-to-depth ratios 0.5 and 0.7, subjected to an initial load eccentricity ratio of 0.2 and slenderness ratio 15 is shown. It can be clearly seen that the two graphs overlap, i.e. variation in ultimate capacity of axial load and lateral deflection is totally negligible with changing the spacing-to-depth ratio.

5. CONCLUSIONS

The behaviour of slender partially encased composite column has been studied using Newmark's iterative procedure. A parametric study is conducted using this method to identify the potential variables that can significantly affect the behaviour of slender partially encased composite column. The variable parameters include load eccentricity ratio (e/d), slenderness ratio (L/d), flange plate thickness ratio (b/t) and link spacing-to-depth ratio (s/d). The effects of these parameters have been studied by formulating the load deflection response of the parametric column. The effects of these parameters on the axial capacity and second order deflection of the slender column has been demonstrated. The axial capacity of a partially encased composite column has been reduced prominently as the overall slenderness ratio increases, particularly for columns with slender plates. This reduction is more prominent in columns with larger link spacing. Besides, exponential increase in lateral deflection has been observed with the increase in the slenderness ratio. Effect of second order deflection on total deflection has been found to be insignificant when L/d ratio is 10. For the eccentrically loaded columns, load carrying capacity has been found to drop significantly with an increase of eccentricity. The effect of the ratio of initial load eccentricity to the overall depth of the column cross-section has been observed to increase the lateral displacement of columns significantly and has been found more pronounced for columns with higher L/d ratio. The axial capacity of the PEC column has been found to be reduced by an average value of 10% when the flange b/t ratio is changed from 25 to 35. Effect of slenderness ratio and load eccentricity ratio is negligible when b/t ratio is varied. Besides, lateral deflection of slender column has been increased when b/t ratio is increased within this range. The axial capacity and deflection of the PEC column has been observed to be unaffected by the range of link spacing selected in this parametric study.

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INVESTIGATION OF THE EFFECT OF NYLON FIBER IN CONCRETE REHABILITATION

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ABSTRACT

Reinforced concrete structures are susceptible to a variety of deterioration mechanisms due to creep and shrinkage, alkali-silica reaction, carbonation and corrosion of the reinforcemnent. The rehabilitation techniques currently available for concrete range from 'do nothing' through to 'rebuild', with a number of measures in between. It is unlikely that either of the above extremes is ever appropriate for a heritage building unless the damage is either very minor, or severe enough to jeopardize public safety. This paper aims to to rehabilitate the deteriorated structures by using nylon fiber to provide the improvement of mechanical properties of rehabilitated structure. In this research works, cylinders were constructed by using nylon fiber and also without using fiber and determine as well as compare the mechanical properties of fiber reinforced and plain concrete and mortar by conducting different tests, where the test parameters are based on compressive strength. A series of laboratory testing have been also conducted to rehabilitate some deteriorated columns by using nylon fiber and determine and compare the mechanical properties of those columns before and after rehabilitation. The effects of adding nylon fiber for the deteriotion of concrete structure have been discussed in details.

Keywords: *Concrete, Rehabilitation, Fiber, Nylon fiber*

1. INTRODUCTION

The deterioration and critical need for renewal of buildings and civil infrastructure has recently been the focus of considerable discussion among researchers in North America, Europe ,Japan and also in Bangladesh (Busel et al.,1997). The causes of structural deficiencies include poor design, use of inferior materials during construction, poor construction techniques, wear and tear, insufficient monitoring / maintenance, misuse and severe natural structural actions such as earthquake.

Buildings and infrastructure deteriorate or become functionally obsolete for a number of reasons. Potentially significant financial and environmental savings can be realised through the use of fibre composite materials to repair and rehabilitate structures rather than demolishing and reconstructing them (Matthew, 2003). Concrete rehabilitation is today a major industry. It is also a specialist activity. However, the issues surrounding the conservation of our historic concrete structures are only just beginning to be addressed. Unlike some traditional materials, concrete and other modern materials do not yet have well developed, universally implemented conservation methodologies for their investigation, repair or long-term maintenance. In addition, when new repair methods are being developed, conservation needs are rarely considered (Broomfield, 1997).

Now a day's use of natural and artificial fibres is very common in rehabilitation of reinforced concrete structures (Perkins, 2003). The concrete with using fiber is called Fiber Reinforced Concrete (FRC). FRC is Portland cement concrete reinforced with more or less randomly distributed fibres. In FRC, thousands of small fibres are dispersed and distributed randomly in the concrete during mixing, and thus improve concrete properties in all concrete directions. Fibres help to improve the post peak ductility performance, pre-crack tensile strength and compressive strength. Several different types of fibres, both artificial and natural, have been incorporated into concrete. Use of natural fibres in concrete precedes the advent of conventional reinforced concrete in historical context (Neville, 2005 and Shetty, 2003). However, the technical aspects of FRC systems remained essentially undeveloped. Since the advent of fiber reinforcing of concrete in the 1940's, a great deal of testing has been conducted on the various fibres materials to determine the actual characteristics and advantages for each product (Jothi, 2008).

Several different types of fibres have been used to reinforce the cement-based matrices. The choice of fibres varies from synthetic organic materials such as nylon, synthetic inorganic such as steel or glass, natural organic such as cellulose or sisal to natural inorganic asbestos (Bentur et al., 1992). Currently the commercial products are reinforced with steel, glass, polyester and nylon fibres. The selection of the type of fibres is guided by the properties of the fibres such as diameter, specific volume, Young's modulus, tensile strength etc and the extent these fibres affect the properties of the cement matrix (Rizkalla, 1999).

Nylon fibers are 100% virgin polypropylene fibrillated or monofilament fibers of nylon. It is a generic name that identifies a family of polymers. Nylon fibers properties are imparted by the base polymer type, addition of different levels of additive, manufacturing conditions and fiber dimensions. Currently only two types of nylon fibers are marketed for concrete. Nylon is heat stable, hydrophilic, relatively inert and resistant to a wide variety of materials. Nylon is particularly effective in imparting impact resistance and flexural toughness and sustaining and increasing the load carrying capacity of concrete following first crack (Matthew, 2003).

2. BEHAVIOR OF NYLON FIBERS IN A CEMENT MATRIX

This research is oriented towards concrete reinforced with nylon fibres, so it is most important to understand how nylon fibres behave in the cement composite matrix. The study of this mechanism helps to model the behaviour of the composites in a real world environment.

The behaviour of FRC under loading can be understood from the Figure 1. The plain concrete structure cracks into two pitches when the structure is subjected to the peak compressive strength and cannot withstand further load or deformation. The fiber reinforced concrete structure does not crack at the same peak compressive load. The area under the curve shows the energy absorbed by the FRC when subjected to compressive load.

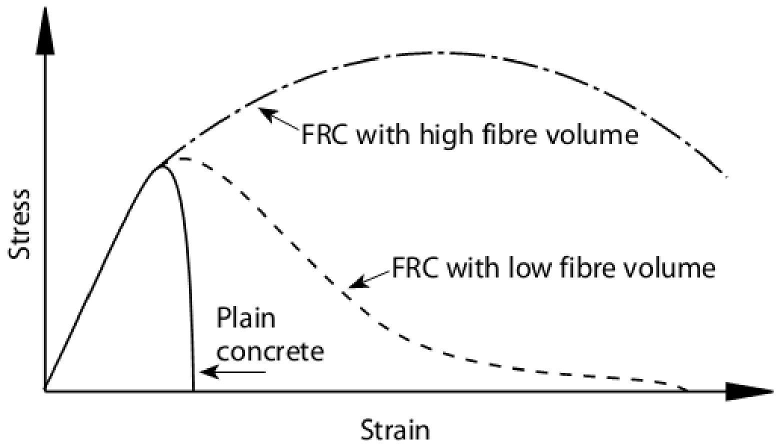

Figure 1: Typical stress-strain curves for fibre-reinforced concrete

The real advantage of adding fibres is when fibres bridge this cracks and undergo pullout processes, such that the deformation can continue only with the further input of energy from the loading source. Reinforcing fibres stretch more than concrete under loading. Therefore, the composite system of fiber reinforced concrete is assumed to work as if it were non-reinforced until it reaches its ''first crack strength''. It is from this point that fiber reinforced takes over and holds the concrete together. With reinforcing, the maximum load carrying capacity is controlled by fibres pulling out of the composite (Hannant, 1978). Reinforcing fibres do not have a deformed surface unlike larger steel reinforcing bars, which have a non-smooth surface, which helps mechanical bonding. This condition limits performance to a point far less than the yield strength of the fiber itself. This is important because some fibres pull out easier than others when used as reinforcing and will affect the toughness of the concrete product in which they are placed. Toughness is based on the total energy absorbed prior to compete failure. The main properties influencing toughness and maximum loading of fiber reinforced concrete are based on the type of fiber used, volume percent of the fiber, the aspect ratio and the orientation of the fiber in the matrix (Naaman, 2003)

The other factors that control the performance of the composite material are physical properties of the reinforced concrete and matrix, the strength of the bond between fibres and matrix. The chemical properties of the fiber in terms of their inertness or reactivity with the surrounding environment plays an important role in determining the bonding characteristics of the fiber and the composite as they may or may not from a chemical bond between the fiber and matrix. Behaviour of FRC when placed in various environments, it has been understood that the change in the properties of the nylon fibres over a period of time when subjected to similar environments also affects the bonding characteristics of the fibres with the matrix. The environmental effects on the nylon fibres used for reinforcement need to be studied to understand the changes of the performance of the FRC loaded conditions (Craig, 1984).

3. MATERIALS AND MIX DESIGN

3.1 Materials

3.1.1 Cement

The Cement (C) used in this research was Ordinary Portland Cement (ASTM Type-1) with specific gravity 3.15. Physical properties and chemical composition of cement is reported in Table 1 and Table 2 respectively.

Table1: Physical properties of cement

Description		ASTM Standard Requirement	Test Result
Fineness	Sieve No. 200 residue (%)	Fraction of Cement Retained < 10%	0.4
	Blaine air permeability (m2/kg)	Min 280	298.56
Setting Time	Consistency (%)	-	26.34%
	Initial Setting (min)	≥ 45 min.	117
	Final Setting (min)	≤ 375 min.	235
Compressive Strength	Age	MPa	MPa
	3 days	Min 12	17.7
	7 days	Min 19	27.43
	28 days	Min 28	35.39

Table 2: Chemical Composition of cement (provided by manufacturer)

Chemical Compound	ASTM C114 Requirement Proportion (% of Mass)		Test Result Proportion (% of Mass)
SiO_2	20.60	90-95%	39.94
Al_2O_3	5.07		4.90
Fe_2O_3	2.90		2.54
CaO	63.9		45.63
MgO	1.53		1.70
K_2O	0.73		-
Na_2O	0.15		-
SO_3	2.53		1.62
LOI	1.58		1.93
Total	**98.99**		**98.27**

3.1.2 Aggregate

The coarse aggregate (CA) used in this study was crushed stone and river sand was used as the fine aggregate (FA). Particle size distribution and physical properties of aggregates are given in Tables 3 & 4 respectively.

Table 3: Physical properties of aggregate used

Properties	River Sand	Crushed Stone
Size	4.75 mm passing	Maximum 10 mm
Bulk specific gravity	2.64	2.68
Water Absorption (%)	0.50	0.24
Bulk density (kg/m^3)	1459	1420
Void content (%)	29.68	43.29
Fineness Modulus (FM)	2.56	6.49
Los Angles Abrasion Test	-	22.3%

Table 4: Particle size distribution of aggregates

Sieve Size (mm)	River Sand (% passing)	Crushed Stone (% passing)
75	100	100
37.5	100	100
25	100	100
18.75	100	100
12.5	100	100
9.375	68.25	100
4.75	30.56	99.9
2.38	12.35	96.88

3.1.3 Nylon Fiber

Figure 2: Nylon Fibre

Table 2: Chemical and Physical Properties of Nylon Fiber

Chemical and Physical Properties	Results
Absorption	Nil
Fiber Length	1/4" - 3/4" Graded
Melt Point	3240 F
Thermal Conductivity	Low
Acid and Salt Resistance	High
Specific Gravity	0.91
Young Modulus	0.5 (3.5 Mpa)
Ignition Point	11000 F
Electrical Conductivity	Low
Alkali Resistance	Alkali Proof

Table 3: Standard amount of various components of Fiber Reinforced Concrete

Components	Standard Amount
Cement content	325 to 550 kg/m^3
W/C Ratio	0.4 to 0.6
Percentage of sand to total aggregate	50 to 100 percent
Maximum Aggregate Size	10 mm
Air-content	2 to 9 percent
Fiber content	0.9 kg/m^3

3.1.4 Water

Potable water was used in this research work.

3.2 Mix Design

Nylon fiber-reinforcing is a mechanical, not chemical process. The addition of fiber does not require any additional water in the mix design.

3.2.1 Mixing Procedures

Nylon fiber is added to the mixer before, during or after batching the other concrete materials. Mixing time and speed are specified in ASTM C-94. Mixing should be carried out for at least 5 minutes at batching plant or in transit mixer, at full speed while using the 0.9 Kg soluble bags.

3.2.2 Mixture Proportioning

The mixture proportioning was done according to the current ACI mix design method (Raju, 2002). The desired mean strength and slump values were 34 MPa and 30-50 mm correspondingly for the OPC control mixture. The total mixing time was 5 minutes; the samples were then casted and left for 24 hrs before demoulding. They were then placed in the curing tank at 23 ± 2^{o}C until the day of testing. Mixing proportion of concrete and mortar samples are given in the Table 7.

Table 4: Concrete and mortar mixture proportioning

Material	**Concrete**		**Mortar**	
Sample	**Plane Concrete (PC)**	**Fiber Reinforced Concrete (FRC)**	**Plane Morter (PM)**	**Fiber Reinforced Morter (FRM)**
Cement (kg)	427	427	427	427
FA (kg)	1004.5	1004.5	1671.5	1671.5
CA (kg)	667	667	0	0
Water (kg)	205	205	205	205
W/C ratio	0.48	0.48	0.48	0.48
Fiber (kg)	0	0.9	0	0.9

3.3 Techniques of Rehabilitation by using Nylon Fiber

At first some amount of concrete or mortar from the deteriorated portion of structure was plucked. Then cement grout was provided through that portion to impart bonding between old and new concrete. After that concrete and/ or mortar made by using nylon fiber was placed on the deteriorated portion. Next, plaster work was done as finishing work. After 24 hours curing was done in that rehabilitation work.

4. EXPERIMENTAL APPROACH

4.1 Location of Deteriorated Structures

For the progress of the research work, four deteriorated columns in Civil Engineering Building in KUET which are to be rehabilitated are located.

4.2 Non-destructive Compressive Strength Test of Deteriorated Columns (Schmidt Hammer test)

The Schmidt Hammer test is the most famous and the simplest method to estimate the compressive strength of concrete, but the deterioration of the concrete surface, that is usually observed in the aged structure, influences on the obtained results seriously. The test measures the rebound of a hardened steel hammer impacted on the concrete by a spring. This method has the same limitations as the surface hardness tests.
The test was carried out as follows:

1. A hole of 1 sq. ft. and 2 in thickness was drilled in the concrete and cleaned out;
2. Test no., date, time, unit, angle and no of blow were adjusted as per requirement.
3. 20 blows were performed by Schmidt hammer on to the concrete, and successive rebound number had been counted.
4. Finally the compressive strength of concrete had been found out automatically with respect to rebound number.

4.3 Mixing of Concrete Batches

The mixing of concrete batches was carried out, with a small drum mixer or small electrical pan mixer. The mixer used for this project is a 60-litre pan mixer. To encourage a uniform distribution of fibres throughout of the concrete, fibres were added to the concrete mix by slowly and evenly. This prevents the congregation of the fibres on the paddle, which leads to balling of fibres. All mixing was performed at the engineering materials laboratory of Civil Engineering Department. The same mix sequence was undertaken for all the mix batches throughout of the project to ensure uniformity. Once the concrete mixing was finished, the fresh concrete was ready for casting of concrete specimens into the concrete moulds to meet the scope of this project.

4.4 Casting of Cylinder

12 nos. 4 in x 8 in and 12 nos. 2 in x 4 in cylinders for concrete and mortars successively casted with using nylon fiber and also casted same amount of cylinders for plain concrete and mortar. Before any fresh concrete was poured into the concrete moulds, all concrete moulds were cleaned from the existing concrete stain and diesel oil was applied inside and around the moulds. The fresh concrete was placed in three equal layers into a cylinder mould. Each layer was consolidated by using a vibrator. After filling the concrete moulds the surface of the concrete was levelled.

4.5 Compressive Strength Test

Compressive strength of a concrete is a measure of its ability to resist static load, which tends to crush it. Most common test on hardened concrete is compressive strength test. It is because the test is easy to perform. Furthermore, many desirable characteristics of concrete are qualitatively related to its strength of concrete in structural design. The compressive strength gives a good and clear indication that how the strength is affected with the use of nylon fiber with concrete.

Concrete specimen for compressive strength test were 4 in diameter and 8 in height and mortar specimen for this test were 2 in diameter and 4 in height. The specimen was placed in the Universal Testing Machine (between the two platens). The axis of the specimen was carefully aligned with the centre of thrust of the spherically seated platen. The apparatus set up is shown in Figure 4. The maximum force was read from the testing machine meter and the reading was recorded. The compressive strength of the specimen was obtained by through the maximum load from the meter over the cross sectional area of the specimen. This test was performed in the “Strength of Materials Laboratory” of Department of Civil Engineering In KUET after the concrete and mortar specimens were cured for 7 , 28 and 90 days. The test procedure was carried out accordance with ASTM C 39.

Figure 3: Non-destructive test of concrete by Schmidt hammer

Figure 4: Typical setup for compressive strength test

4.6 Rehabilitation of Deteriorated Columns

At first some amount of concrete or mortar from the deteriorated portion of structure was plucked. Then cement grout was provided through that portion to impart bonding between old and new concrete. After that concrete and/ or mortar made by using nylon fibre was placed on the deteriorated portion. Next, plaster work was done as finishing work. After 24 hours curing was done in that rehabilitation work. Figure 5 and 6 show the rehabilitation technique of column by using nylon fibre.

Figure 5: Plucking of old deteriorated concrete from column

Figure 6: Rehabilitation of column by using nylon fiber

5. RESULTS AND DISCUSSIONS

5.1 Compressive Strength Test of Cylinders

The results of compressive strength test indicate that the addition of nylon fibres to concrete and mortar has significance effects on the ultimate capacity of the concrete and mortar in compression.

Table 8: 7, 28 and 90 Days compressive strength of fibber reinforced and plain concrete and mortar

Sample	7 Days (MPa)	28 Days (MPa)	90 Days (MPa)
PC	16.90	28.97	39.31
FRC	17.24	30.00	42.07
PM	24.48	35.86	46.21
FRM	25.52	37.93	49.66

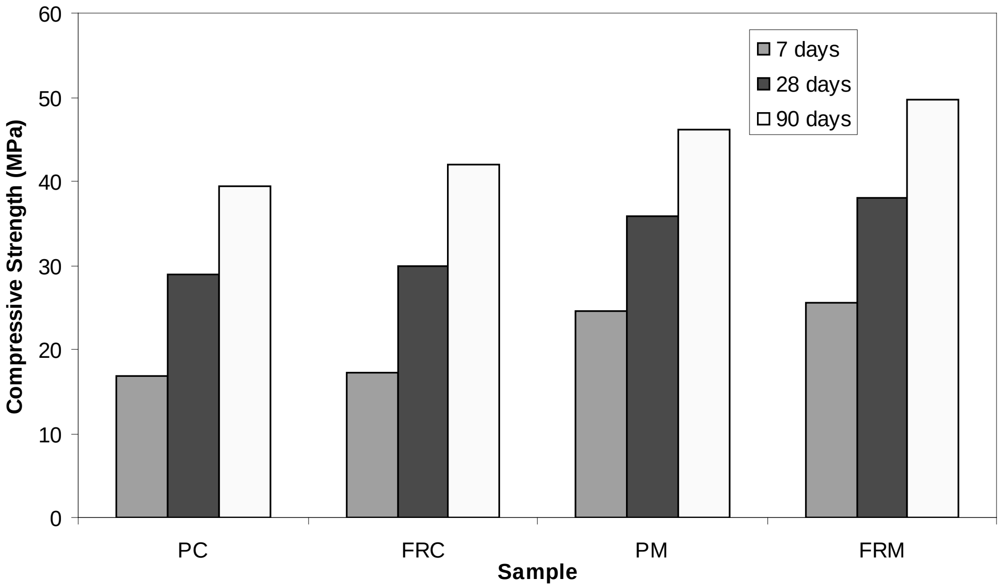

Figure 7: Compressive Strength of Various Samples

Figure 7 shows the variation of compressive strength of concrete and mortar with respect to age of curing. It is seen that the compressive strength decreases with the use of nylon fibre in both concrete and mortar compared with plain concrete and mortar. As the use of nylon fibre increases the inter-connected voids in concrete so the strength decreases. But the amount of decreasing rate is about 3 – 4 % and is not so high. So it can be used Fibre Reinforced Concrete and Mortar instead of Plain Concrete and Mortar.

5.2 Compressive Strength Test on Columns

The results of non-destructive compressive strength test of concrete on columns indicate the significance differences of strength of concrete before and after rehabilitation of columns using Fibre Reinforced Concrete (FRC).

Table 9: 28 Days compressive strength of concrete of columns before and after rehabilitation using fibre reinforced concrete and mortar.

Column No.	Rehabilitated By	Compressive Strength (MPa)		
		Before Rehabilitation	After Rehabilitation	Percentage Increase
# 1	FRC & FRM	7.93	13.79	73.91
# 2		8.97	14.48	61.54
# 3	PC & PM	15.17	18.28	20.45
# 4		12.41	15.17	22.22

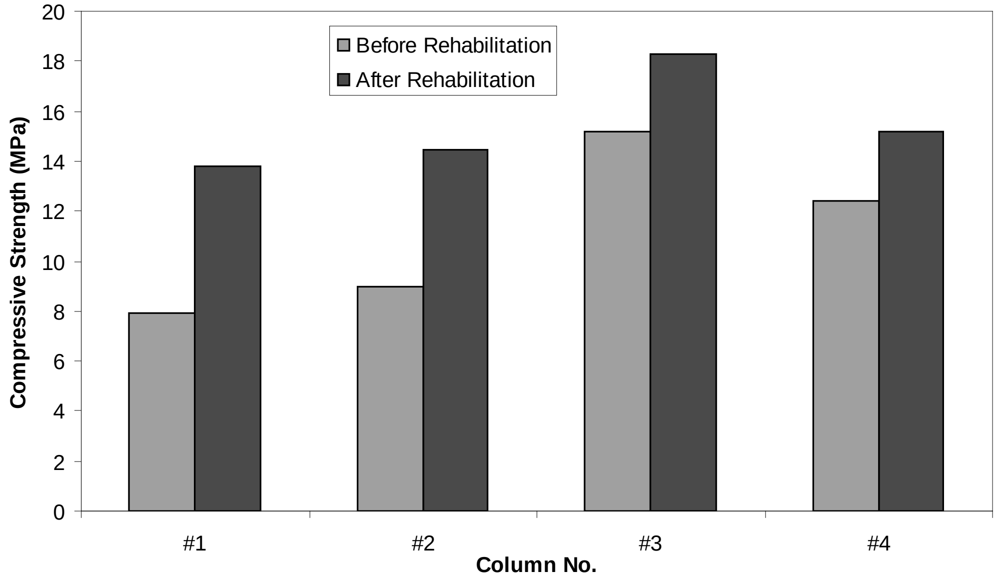

Figure 8: Bar diagram of compressive strength vs. column no. for the columns rehabilitated by various mixes

Figure 8 shows the differences of compressive strength of concrete before and after rehabilitation and indicate the increase of strength after rehabilitation compared with the strength before rehabilitation. The average percentage increase of strength is greater for the columns rehabilitated by using FRC than that of columns rehabilitated using plain concrete and mortar. So, it can be said that rehabilitation of columns using fibre provides better mechanical properties of concrete.

6. CONCLUSIONS

The compressive strength of concrete is one of the most important and useful properties of concrete. In most structural applications concrete is employed primarily to resist compressive stresses. Therefore, the concrete making properties of various ingredients of mix are usually measured in terms of the compressive strength. The results indicate that the concrete properties were significantly influenced by using fibre. From this result, it is evident that for improving the properties of concrete, the percentage increment of compressive strength using FRC and FRM is more than 60 and that for PC and PM is around 20. In the present investigation, though the available results clearly imply the influence of nylon fibre on the properties of fresh and hardened concrete s, it is deemed essential to extend the present studies using different types of aggregate and binders to conclude precisely that the relationships apply irrespective of the constituent types. The above said investigations will be performed as a future work

The major conclusions derived from this study are as follows:

1. The use of nylon fiber decreases the compressive strength of both concrete and mortar.
2. Rehabilitation by using nylon fiber provides better result in the sense of compressive strength with respect to plane concrete.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

SEISMIC ANALYSIS OF MUD HOUSE

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ABSTRACT

Earthquake is a sudden natural disaster that happens unlike other natural disasters without any proven pre-warning system. This phenomenon makes earthquake most devastative and unpredictable natural calamity. In recent years, some earthquakes having their magnitude from 4.0 to 6.9 Richter scale (light – strong) jolted Bangladesh and its neighboring countries, which substantiate the prediction that Bangladesh is in high risk of major earthquake in near future. It is necessary to take preventive measures for covering the risks associated with earthquake hazards. In Bangladesh most of the people lives in village and mud houses are very common in the rural areas. In quantity, fatalities in earthquake are mostly due to structural collapse of weak masonry building i.e. adobe, rammed earth construction etc. In the current paper, computer modeling and seismic analysis of two models of mud houses ong with reinforcement have been performed using the finite element software STAAD.Pro-2006 and compared with seismic test performed in the laboratory. From the comparison it was observed that finite element analysis can be an alternative of laboratory based seismic tests. Shaking table test is very costly and rarely available. Computer modeling to simulate seismic test involve less budget, labor and time and can be considered as influence from practical consideration. This paper also includes parametric study (height and thickness of wall, opening size, shape and location) and identification of critical location of failure using finite element analysis. The study is expected to give guidelines for design, construction and strengthening of mud houses to mitigate the earthquake hazards in the rural area.

Keywords: *mud house, computer modelling, solid element, seismic analysis, parametric study*

1. INTRODUCTION

Bangladesh is a developing country. It is located in a tectonically active region close to the junction of the north moving Indian plate and the Eurasian plate resulting in several seismic sources (fault zones) in and around the country. The Great Indian Earthquake (Magnitude 8.7) in 1897 had its epicenter only 230 kilometer away from the capital city of Bangladesh (Ansary, 2006). Every year, about 25-30 small to medium tremors occurs in Bangladesh. The possibility of a stronger earthquake is imminent and corresponding devastation is just destabilization of our hard earned development and huge life loss.

The history of earthquake in Bangladesh is sufficient enough to require their careful consideration in the design of structures and facilities. The objective of earthquake resistant design is to install a structure or facility that can withstand a certain level of ground shaking without excessive damage (Ansary, 2000). Casualties in earthquake arise mostly from structural collapse. The greatest proportion of fatalities results from the collapse of weak masonry buildings i.e. adobe, rubble stone or rammed earth construction (Aikaterini, 2004). About 80% people of Bangladesh live in rural areas and a significant portion of them resides in mud houses. Therefore it has become very much essential to analyze the effect of earthquake on these mud houses. Earthquake analysis with comprehensive coverage of parametric study incorporating openings (shape, size and location) in wall, wall thickness etc. is important.

2. THEORETICAL CONSIDERATION

2.1 Software and basic element

Shaking table test is very costly and rarely available in our country. Parametric study on the basis of experiment is again laborious, time consuming and expensive. Computer modeling is very suitable for this purpose, provided appropriate software is available. For the present study STAAD.Pro-2006 was used.

The structure was discretized by 8-noded solid elements. The solid elements used in STAAD have three translational degree-of-freedoms per node. Solid elements enable the solution of structural problems involving general three dimensional stresses. Like dam and embankment, solid element provides a tool for the finite element discretization and analysis for mud house.

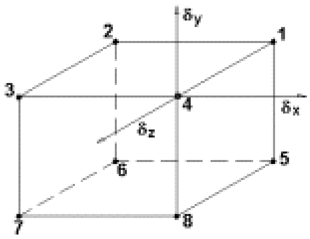

Figure 1: Eight noded solid element

Solid element stresses may be obtained at the center and at the joints of the solid element. The stress items are:

Normal Stresses: SXX, SYY and SZZ Normal
Shear Stresses: SXY, SYZ and SZX
Principal stresses: S1, S2 and S3.

2.2 Material property and seismic constants

Two types of materials were used for simulation of mud house model in STAAD i.e. a) Mud and b) Timber. The material property of mud and timber considered for this study are as follows:

Table 1: Various Properties of Mud and Timber

Property	Mud	Timber
Young’s modulus (E)	12,500 psi	13,32,800 psi
Poisson’s ratio (μ)	0.45	0.15
Density	100 pcf	25 pcf
Shear modulus (G)	4310.35 psi	579478.26 psi

The seismic constants used to define seismic load are, (i) Seismic zone factor ‘Z’; (ii)Importance factor ‘I’, (iii) Numerical coefficient ‘R’ representative of the inherent over strength and global ductility capacity of lateral-force-resisting systems; (iii) Soil Profile type S_A, S_B-S_E; (iv) Near source factor N_a, (v) Near source factor N_v ; (vi) C_t value to calculate time period (T).

2.3 Seismic Analysis

Seismic analysis is of two types, i.e. Static analysis and Dynamic analysis. In the current study, static analysis was preferred.

2.3.1 Static Force Procedures for Seismic Analysis

STAAD offers facilities for determining the lateral loads acting on structures due to seismic forces, using the rules available in several national codes and widely accepted publications. The codes and publications allow for equivalent static force methods to be used in place of more complex methods like response spectrum and time history analysis. Among the several such codes supported by STAAD are UBC (Uniform Building Code), IBC (International Building Code) etc. UBC code (1997) was used in case of static analysis of mud house due to its availability (STAAD-pro 2006-Technical reference).

STAAD is equipped with built-in algorithms to generate lateral seismic loads (as per the Uniform Building Code) on a structure. Once the lateral loads are generated in STAAD, the program can then analyze the structure for those loads using the applicable rules explained in the code documents **(**STAAD-pro 2006-Technical reference).

The STAAD seismic load generator follows the procedure of equivalent lateral load analysis explained in the UBC code. It is assumed that the lateral loads will be exerted in X and Z directions (horizontal) and Y will be the direction of the gravity loads. Total lateral seismic force or base shear is automatically calculated by STAAD using the appropriate equation from the code. After the base shear is calculated from the appropriate equation, it is distributed among the various levels of the structure as per UBC specifications. The distributed base shears are subsequently applied as lateral loads on the structure.

2.3.2 Comparison between UBC 97 and BNBC 93 Code

According to the Uniform Building Code (UBC 97), design base shear

$$V = \frac{C_V I}{RT} W$$

But the upper and lower limits of the design base shear are respectively

$$V = \frac{2.5C_g I}{R} W$$

and $V = 0.11 C_g I W$

Where, C_v and C_a are seismic coefficient dependent on soil type, I, R, W has their usual meaning mentioned earlier and elastic fundamental period of vibration, in seconds $T = C_\tau (h_n)^{3/4}$

Again, according to BNBC 93, the design base shear

$$V = \frac{ZIC}{R} W$$

Where,

$$C = \frac{1.25S}{T^{2/3}}$$

Z, I, R, W, T has their usual meaning and S represents the type of soil. Appropriate values of the following parameters were taken from the respective code as showed in Table 2 considering that, the mud house is located in Dhaka and height of mud house is 9.5ft.

Table 2: Difference of Seismic Parameter between UBC 97 and BNBC 93

Seismic Parameter	**UBC 97**	**BNBC 93**
Seismic zone factor *Z*	0.30	0.15
seismic coefficient C_V	0.84	-
seismic coefficient C_g	0.36	-
Importance factor I	1.0	1.0
Numerical coefficient R	2.9	4.0
C_τ	0.020	0.049
Height of mud house h_n	9.5 ft	9.5 ft
Type of soil *S*	-	1.5

Base shear of the mud house in terms of total seismic weight (W) of the structure as per UBC 97 and Bangladesh National Building Code (BNBC) 93 Code were *0.3103W* and *0.3086W* respectively. It was observed that the values of base shear obtained from UBC 97 and BNBC 93 code are almost same. Again, the distribution formula of base shear to the structure is same in UBC 97 and BNBC 93 codes. In the current study, Uniform Building Code (UBC) 97 code was used to define the seismic loading.

2.3.3 Seismic Load Auto Generation in STAAD.Pro-2006

STAAD is equipped with built-in algorithms to generate lateral seismic loads (as per the Uniform Building Code) on a structure. Once the lateral loads are generated in STAAD, the program can then analyze the structure for those loads using the applicable rules explained in the code documents.

The STAAD seismic load generator follows the procedure of equivalent lateral load analysis explained in the UBC code. It is assumed that the lateral loads will be exerted in X and Z directions (horizontal) and Y will be the direction of the gravity loads. Total lateral seismic force or base shear is automatically calculated by STAAD using the appropriate equation from the code. After the base shear is calculated from the appropriate equation, it is distributed among the various levels of the structure per UBC specifications. The distributed base shears are subsequently applied as lateral loads on the structure.

3. MODELING OF MUD HOUSE

For modeling of mud house at first the basic finite element unit “8-noded solid element” is generated. Translating the basic unit in global X, Z and Y direction the mud house model can be created. For this purpose, at first the solid element translated in global X direction according to the desired length of plinth; than all the solid elements in the existing window translated in global Z direction according to the desired width of plinth. Next all the solid elements in the existing window translated in global Y direction according to the height of plinth. According to the dimension and location of the walls, the walls are created first up to the height from where opening starts (For the ease of understanding the plinth is not shown in the figure 6) The locations and dimension of openings are selected and then the solids are translated in such a manner that, where there is an opening, a blank space is there in the wall. Finally a complete mud house model of required shape was created (Fig.8).
The step by step procedure of a typical mud house modeling is chronologically illustrated below:

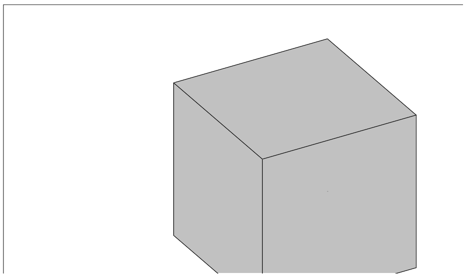

Figure 2: Single solid

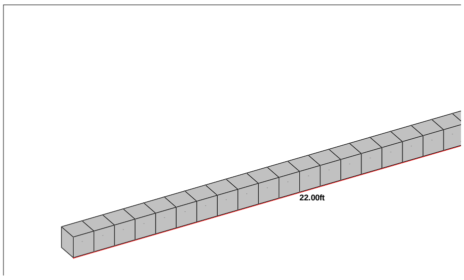

Figure 3: Solid element translation in X

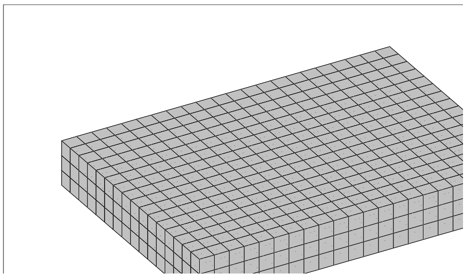

Figure 5: Plinth of mud house

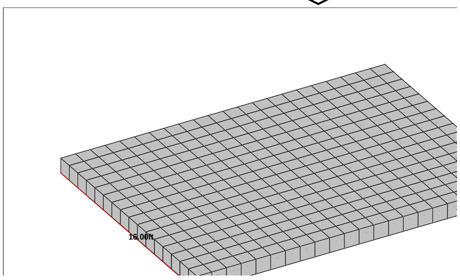

Figure 4: 1st layer of plinth

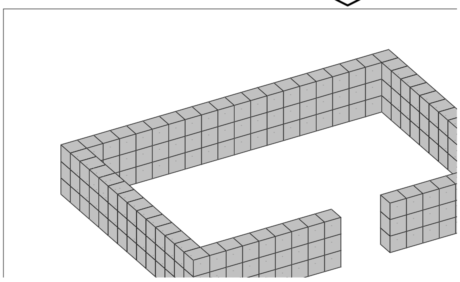

Figure 6: Mud wall (height below window) of mud house excluding plinth

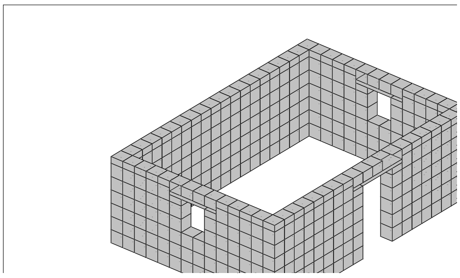

Figure 7: Mud house model without plinth

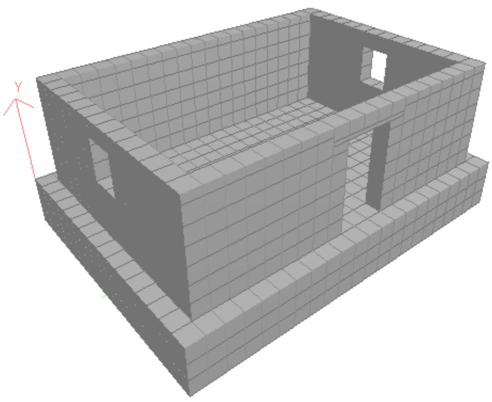

Figure 8: Complete mud house model

4. ANALYSIS AND VERIFICATION

In case of analysis by a model, the analysis result should be verified and material property should be representative. The efficiency of modelling needs to be verified. In most case the verification is done by experimental test.

4.1 Analysis and stress distribution

According to procedure described before a mud house model was created for analysis as shown in figure 8. The user friendly load menu of STAAD.Pro-2006 was used to generate load (density and UBC seismic load) on the structure. The resulting output includes stress distribution, base shear, deflection etc. Of which normal stresses are discussed in this paper. For ease of understanding of the result the limiting value as determined by laboratory tests (compressive stress,10 psi ; tensile stress,1.2 psi) was assigned to identified elements. The results having higher stress than allowed are correspondingly represented by color. For example green color shows the safe zones. Red color shows the compressive stress zone and the purple color shows the tensile stress zones. From the analysis result critical locations of failure identified by analysis were found near the base of mud house, at the bottom of door, above the lintel of door, corners of mud house, and the adjacent area of opening (as shown in figure 9,10 and 11)

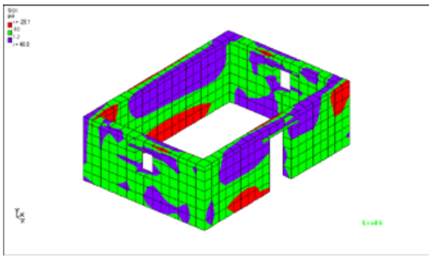

Figure 9: Stress (SXX) distribution contour in mud house

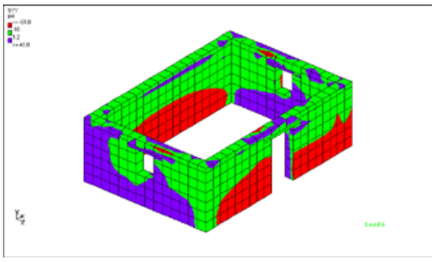

Figure 10: Stress (SYY) distribution contour in mud house

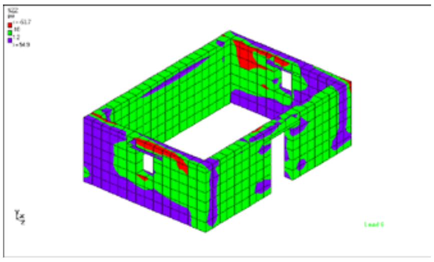

Figure 11: Stress (SZZ) distribution contour in mud house

4.2 Verification with experimental result

Two models of mud house (without reinforcement and with reinforcement) were analyzed for seismic loading and compared with experimental result with respect to the usual laggings of modeling procedure.

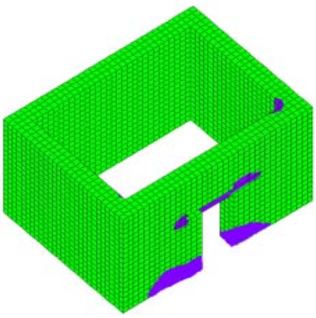

Figure 12: Stress contour in mud house model (without reinforcement) by STAAD analysis

Figure 13: Position of crack formation in unreinforced mud house in experiment.

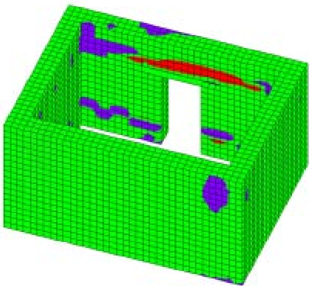

Figure 14: Stress contour in mud house model (with reinforcement) by STAAD analysis

Figure 15: Position of crack formation in reinforced experiment.

It was observed from the analysis result that mud house without reinforcement has greater stress in the bottom (left side wall and back wall), at the bottom of door and near the lintel of door which approximately simulates the crack due to first cycle of horizontal loading in the model tested. In case of reinforced mud house model, analysis result shows that greater stresses were adjacent to the location of reinforcement. In experimental mud house model having reinforcement; the cracks were not localized like unreinforced model, but the cracks were distributed in the whole structure.

The crack distribution predicted by analysis of models show good agreement with that of tested results.

5. PARAMETRIC STUDY

Parametric study on mud house was carried out in the following aspects with respect to seismic loading:

1. Variation of thickness of the mud wall in mud house with respect to a constant height.
2. Variation of the height of mud house with respect to a constant thickness of mud wall.
3. Variation of the size of window (opening) size.
4. Variation of the position of window (opening).

By parametric study following findings were obtained.

1. If the thickness of mud wall in mud house increased with respect to a constant height, the safe zone (green colored area) is also increased.
2. If the height of mud wall in mud house decreased with respect to a constant thickness, the safe zone (green colored area) is also increased.
3. If the size of window (opening) is smaller and square and the window is at mid position of wall, the safe zone (green colored area) is also increased.

6. CONCLUSIONS

It can be concluded from analysis result and parametric study that, the mud house will be more safe against earthquake if

1. Thickness of mud wall in mud house is increased.
2. Size of window (opening) is small and square one.
3. Window (opening) is at mid position of the mud wall.
4. Reinforcement is used in mud house.

Mud has a wide range of variation in its property and it differs from place to place, weather condition (i.e. in rainy season the mud walls will have definitely high moisture content than dry season) etc. It is possible to change the property of mud in STAAD analysis. For further study it is recommended that the analysis should be performed with a wide range of variation in property of mud, more parameters should be selected for parametric study i.e. changing the meshing size to know the effect of meshing size variation on result, and dynamic analysis can also be performed for parametric study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PREDICT SHEAR STRENGTH OF FERROCEMENT STRUCTURAL ELEMENTS (BEAMS AND SLABS) USING ARTIFICIAL NEURAL NETWORK METHOD

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ABSTRACT

The purpose of this study was to investigate the behavior of ferrocement beam sections (I , rectangular, channel and box) and ferrocement slab section, reinforced with different percentage of tensile reinforcements with different sizes. The research follows the theoretical approach by using Artificial Neural Network (ANN) method. No lab experiments will be done. The primary objective is to represent a model of shear strength and bending strength prediction by employing ANN of ferrocement beams with forces acting along the plane and out of plan. Developing design equations for ferrocement beams by analyzing ferrocement sections using finite element method and appropriate equations.Design charts will be introduced for ferrocement beam sections (rectangular, I, channel and box) to serve as an aid for a prospective designer to arrive at a section to suit their requirements. Using ANN method, information of laboratory test results from researches is needed to form a database of samples to make the models.Finding the appropriate model having material non-linearity in (ANSYS software) and doing a numerical investigation using Finite Element Method. A comparison between the results and laboratory tests of some of previous researches has been done as verification. Applying ANN method using (MATLAB toolbox) to simulate the appropriate model of ferrocemen beams and slabs subjected to different types of loadings presented in several researche. Beside this, some of the materials properties obtained from the performed experiment were frequently used in the modeling. Finally, using ANSYS , various section beam and slab models were developed and then compared. Due to not having uniform material properties as given in the model of ANSYS, in practical it shows non-linear behavior and more deflection than ANSYS model.

Keywords:

1. INTRODUCTION

Ferrocement is a composite material which is used in building or sculpture with cement, sand, water and wire or mesh material. Ferrocement has great strength and economy. In its role as a thin reinforced concrete product and as laminated cement based composite, ferrocement has found itself in numerous applications both in the construction of new structures and repair/rehabilitation of existing structures. Compared with conventional reinforced concrete, ferrocement is reinforced in two directions; therefore, it has homogenous isotropic properties in two directions. Benefiting from its usually high reinforcement ratio, ferrocement generally has a high tensile strength and high modules of rupture. In addition, because the specific surface of ferrocement reinforcement is higher than that of reinforced concrete, larger bond forces develops with matrix resulting in average crack spacing and crack width of smaller magnitude than that of conventional reinforced concrete [1], [2]. Other appealing features of ferrocement include ease of fabrication and low cost in maintenance and repair. Based on these advantages, ferrocement can be effectively utilized for water tanks, boats, housing wall panels, roofs, form work and retrofitting [3]–[5] . Also when a ferrocement sheet is mechanically overloaded, it will tend to fold instead of crack or rupture. Major disadvantage of ferrocement construction is the labor intensive nature of it, which make it expensive for industrial application in the western world [5].

The credit of using ferrocement in the present day goes to Joseph Louis Lambot who in 1848 constructed several rowing boats, plant pots, seats & other items from a material he called ferciment. Lambot's construction consisted of a mesh or grid reinforcement made of two layers of small diameter on bars at right angle & plastered with cement mortar with a thin cover to reinforcement Lambot's rowboats were 3.66 m long, 1.22 m wide & 25 mm to 38 mm thick. These were reinforced with grid & wire netting. One of the boat build by him, still in remarkably good condition, is on display in the museum at Brignoles, France [6].

There was very little application of true ferrocement construction between 1888 & 1942 when Pier Luigi Nervi began a series of experiments on ferrocement. He observed that reinforcing concrete with layers of wire mesh produced a material possessing the mechanical characteristics of an approximately homogenous material capable of resisting high impact. After the Second World War, Nervi demonstrated the utility of ferrocement as a boat building material.

2. METHODOLOGY

First, literature review will be done to collect related previous information about analytical and experimental work in connection to ferrocement beam and slab sections. Next a finite element model will be developed for analysis with ANSYS (rectangular, I, channel and box) columns. In the third step ANSYS analysis will be done. Then parametric study will be done. After that design equations for ferrocement beams will be developed. Analyzing ferrocement sections using finite element method and appropriate equations, design charts will be introduced for ferrocement beam sections (rectangular, I, channel and box) to serve as an aid for a prospective designer to arrive at a section to suit their requirements.Using ANN method, information of laboratory test results from researches will be data based to make the models . The appropriate model having material non-linearity in (ANSYS software) and numerical investigation using Finite Element Method will be done. A comparison between the results and laboratory tests of some of previous researches will be done as verification. Applying ANN method using (MATLAB toolbox) ,the appropriate model of ferrocement beams and slabs subjected to different types of loadings presented in several researches will be simulated.

2.1 Databases of Samples to Make the Models

From the analysis results the database for ANN method and mathematical simulation to make appropriate model is given below.

Table 1 Detailed Summary for box section beams

Beam	Maximum allowable total load	Loading	Length(ft)	Area	Maximum Shear(KN)	Maximum Moment (KNm)
$\boldsymbol{B^{B1}}$	115	One point	4’	60in^2	57.5	115
$\boldsymbol{B^{B2}}$	117	Two point	4.25’	65in^2	58.5	125.7
$\boldsymbol{B^{B3}}$	70	Horizontal	4.5’	70in^2	35	78.75
$\boldsymbol{B^{B4}}$	112	Combined	4.75’	75in^2	50.2	102
$\boldsymbol{B^{B5}}$	115	UDL	5’	80in^2	55.2	135
$\boldsymbol{B^{B6}}$	115	Combined	5.25’	85in^2	55.2	137
$\boldsymbol{B^{B7}}$	122	UDL(part)	5.5’	90in^2	57.2	142
$\boldsymbol{B^{B8}}$	70	UVL	5.75’	95in^2	55.4	137

Table 2 Details of rectangle section beams

Beam	Maximum allowable total load	Loading	Length(ft)	Area	Maximum Shear(KN)	Maximum Moment (KNm)
$\boldsymbol{B^{R1}}$	110	One point	4’	60in^2	40	80
$\boldsymbol{B^{R2}}$	150	Two point	4.25’	65in^2	46.34	115.52
$\boldsymbol{B^{R3}}$	135	Horizontal	4.5’	70in^2	60	135
$\boldsymbol{B^{R4}}$	100	Combined	4.75’	75in^2	45	120
$\boldsymbol{B^{R5}}$	110	UDL	5’	80in^2	50	115
$\boldsymbol{B^{R6}}$	140	Combined	5.25’	85in^2	54	127
$\boldsymbol{B^{R7}}$	100	UDL(part)	5.5’	90in^2	60	110
$\boldsymbol{B^{R8}}$	90	UVL	5.75’	95in^2	80	115

Table3 Details of channel section beams

Beam	Maximum allowable total load	Loading	Length(ft)	Area	Maximum Shear(KN)	Maximum Moment (KNm)
$\boldsymbol{B^{C1}}$	85	One point	4’	60in^2	42.5	85
$\boldsymbol{B^{C2}}$	90	Two point	4.25’	65in^2	45	90

B^{C3}	70	Horizontal	4.5’	70in^2	30	76
B^{C4}	130	Combined	4.75’	75in^2	53	110
B^{C5}	90	UDL	5’	80in^2	45	112.5
B^{C6}	100	Combined	5.25’	85in^2	50	125
B^{C7}	72	UDL(part)	5.5’	90in^2	40	95
B^{C8}	80	UVL	5.75’	95in^2	60	92

Table4: Details of I section beams

Beam	Maximum allowable total load	Loading	Length(ft)	Area	Maximum Shear (KN)	Maximum Moment (KNm)
B^{I1}	190	One point	4’	60in^2	90.34	190
B^{I2}	187	Two point	4.25’	65in^2	87	188
B^{I3}	110	Horizontal	4.5’	70in^2	42.7	100.75
B^{I3}	220	Combined	4.75’	75in^2	112	230
B^{I3}	160	UDL	5’	80in^2	80	216
B^{I3}	160	Combined	5.25’	85in^2	90	210
B^{I3}	150	UDL(part)	5.5’	90in^2	90	185
B^{I3}	140	UVL	5.75’	95in^2	100	177

Table 5Details of slabs

Slab	Maximum allowable total load	Length(ft)	Width(ft)	Thickness(inch)	Maximum Shear (KN)	Maximum Moment (KNm)
S^{P1}	90	12	12	6”	42.7	61.75
S^{P2}	72	12	10	6”	35.7	51.77
S^{P3}	64	12	8	6”	37.7	57.75

2.2 Modeling

The mathmetical model is developed to predict the shear force and bending moment capacity of ferrocement members using ANN method. The algorithm of this method is explained below.

For ANN method model developing we have used 4 inpouts (X section,loading type ,load and length of the beam.)and output is 2 (shear force and bending moment).We have used Mathlab toolbox for this modeling.5 types of modeling is being performed here(box section beam,rectangle section beam,channel section beam,I section beam and slab).The algorithm of the model is being given below.Allso see the Appendix C for the methodology of the model formation.

A neural network is a nonlinear system consisting of a large number of highly interconnected processing units, nodes or artificial neurons. Each input signal is multiplied by the associated weight value and summed at a neuron. The result is put through an activation function to generate a level of activity for the neuron. This activity is the output of the neuron. When the weight value at each link and the connection pattern are determined, the neural network is trained. This process is accomplished by learning from the training set and by applying for a certain learning rule. The trained network can be used to generalize for those inputs that are not included in the training set. Compared to conventional digital computing techniques, neural networks are advantageous because of their special features, such as the massively parallel processing, distributed storing of information, low sensitivity to error, their very robust operation after training, generalization and adaptability to new information . Neural networks (NNs) are a powerful computational tool able to `learn' from a set of examples with known inputs and outputs. An artificial neuron is composed of five main parts: inputs, weights, sum function, activation function and outputs. Inputs are information that enters the cell from other cells from the external world. Weights are values that express the effect of an input set or another process element in the previous layer on this process element. Sum function is a function that calculates the effect of inputs and weights totally on this process element. This function calculates the net input that comes to a cell. The information is propagated through the neural network layer by layer, always in the same direction. Besides the input and output layers, there can be other intermediate layers of neurons, which are usually called hidden layers.

3. RESULT AND COMPARISON

From ANN method the results from Ansys analysis and simulation results are given below.

Table6 Details of box section beams

Beam	Type	Loading	Length(ft)	Area	From Ansys Shear(KN)	From Ansys Moment (KNm)	From ANN Shear(KN)	From ANN Moment (KNm)
B^{B1}	Box section	One point	4’	$60in^2$	57.5	115	51.5	112
B^{B2}	Box section	Two point	4.25’	$65in^2$	58.5	125.7	54.2	120
B^{B3}	Box section	Horizontal	4.5’	$70in^2$	35	78.75	33	75
B^{B4}	Box section	Combined	4.75’	$75in^2$	50.2	102	50	98
B^{B5}	Box section	UDL	5’	$80in^2$	55.2	135	52	128
B^{B6}	Box section	Combined	5.25’	$85in^2$	55.2	137	53	129
B^{B7}	Box section	UDL(part)	5.5’	$90in^2$	57.2	142	54.4	142
B^{B8}	Box section	UVL	5.75’	$95in^2$	55.4	137	52.2	135

Table7 Details of box section beams

Beam	Type	Loading	Length(ft)	Area	From Ansys Shear(KN)	From Ansys Moment (KNm)	From ANN Shear(KN)	From ANN Moment (KNm)
B^{B1}	Rectangle	One point	4’	$60in^2$	40	80	35	75
B^{B2}	Rectangle	Two point	4.25’	$65in^2$	46.34	115.52	41	105
B^{B3}	Rectangle	Horizontal	4.5’	$70in^2$	60	135	53	125
B^{B4}	Rectangle	Combined	4.75’	$75in^2$	45	120	41	112
B^{B5}	Rectangle	UDL	5’	$80in^2$	50	115	47	110
B^{B6}	Rectangle	Combined	5.25’	$85in^2$	54	127	53	120
B^{B7}	Rectangle	UDL(part)	5.5’	$90in^2$	60	110	53	102
B^{B8}	Rectangle	UVL	5.75’	$95in^2$	80	115	76	106

Table8 Details of Channel section beams

Beam	Type	Loading	Length(ft)	Area	From Ansys Shear(KN)	From Ansys Moment (KNm)	From ANN Shear(KN)	From ANN Moment (KNm)
B^{C1}	Channel	One point	4’	$60in^2$	42.5	85	40	74
B^{C2}	Channel	Two point	4.25’	$65in^2$	45	90	44	83
B^{C3}	Channel	Horizontal	4.5’	$70in^2$	30	76	27	71
B^{C4}	Channel	Combined	4.75’	$75in^2$	53	110	50	103
B^{C5}	Channel	UDL	5’	$80in^2$	45	112.5	41	105
B^{C6}	Channel	Combined	5.25’	$85in^2$	50	125	49	109
B^{C7}	Channel	UDL(part)	5.5’	$90in^2$	40	95	38	91
B^{C8}	Channel	UVL	5.75’	$95in^2$	60	92	54	89

Table9Details of I section beams

Beam	Type	Loading	Length(ft)	Area	From Ansys Shear(KN)	From Ansys Moment (KNm)	From ANN Shear(KN)	From ANN Moment (KNm)
B*I1*	I section	One point	4’	60in^2	90.34	190	88	185
B*I2*	I section	Two point	4.25’	65in^2	87	188	85	173
B*I3*	I section	Horizontal	4.5’	70in^2	42.7	100.75	43	97
B*I3*	I section	Combined	4.75’	75in^2	112	230	114	221
B*I3*	I section	UDL	5’	80in^2	80	216	81	211
B*I3*	I section	Combined	5.25’	85in^2	90	210	88	203
B*I3*	I section	UDL(part)	5.5’	90in^2	90	185	87	181
B*I3*	I section	UVL	5.75’	95in^2	100	177	96	173

Table10 Details of I slabs

Slab	Type	Length(ft)	Width(ft)	Thicknes s(inch)	From Ansys Shear(KN)	From Ansys Moment (KNm)	From ANN Shear (KN)	From ANN Moment (KNm)
S*P1*	Two way	12	12	6”	42.7	61.75	41.2	57.3
S*P2*	Two way	12	10	6”	35.7	51.77	34	50
S*P3*	Two way	12	8	6”	37.7	57.75	35	55

3.1. Detailed Summary

The mathematical model is developed to predict the shear force and bending moment capacity of ferrocement members using ANN method. The algorithm of this method is being explained before. Readers are referred to References Appendix C for more details on the ANN method. Using membership functions (MFS) of type (gussmf) for all input variable and linear for output, the number of MFs assigned to each input variable is chosen by trial and error. After training and testing the number of MFs was fixed seven MFs for each input variable, when the ANN model reaches to the acceptable satisfactory level. Presents architecture of the adaptive ANN inference system to predict shear and moment capacity of ferrocement beam and slab. A comparison between the predictions from ANN and the experimental value is shown for both training and testing data set in the figures respectively. The predictions appear to be quite good with correlation R. 6. Comparison between experimental and theoretical results A comparison between the predictions of shear and moment capacity of ferrocement members from ANN, and the experimenter were closely in agreement with experimental results as shown. The correlation R of predicted shear and moment capacity.

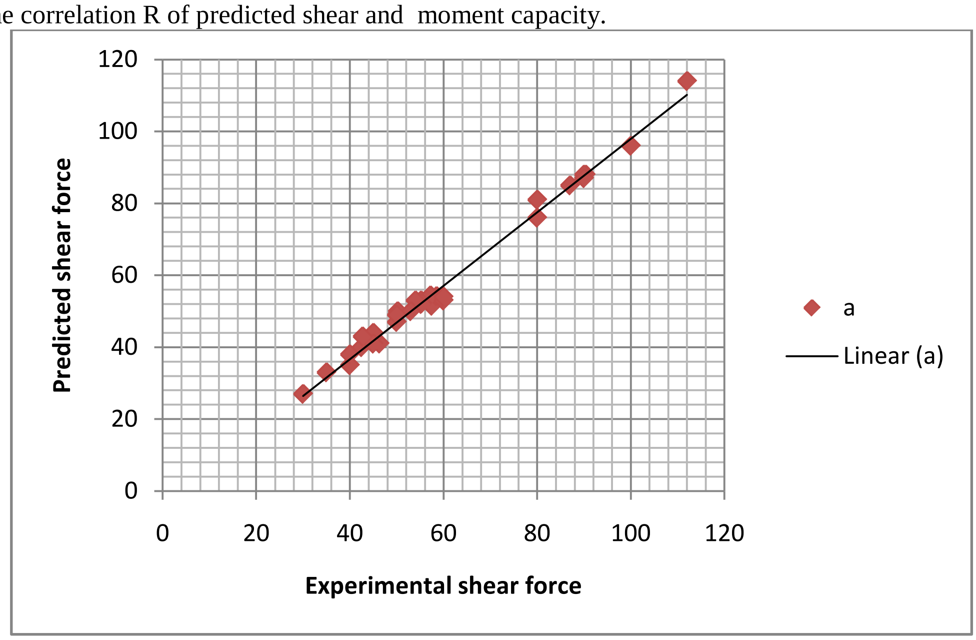

Fig.1. Comparison experimental and predicted values for training data set.

Here,shear force from ansys analysis is beng plotted in X axies and predicted shear force for same beams are plotted in Y axies.The results shows slight difference in this two type forms.So there might be some error in analysis or modeling by ANN method The correlation coefficient R is being evaluated from this graph and its value is 0.9932.from statistical analysis by Excell.

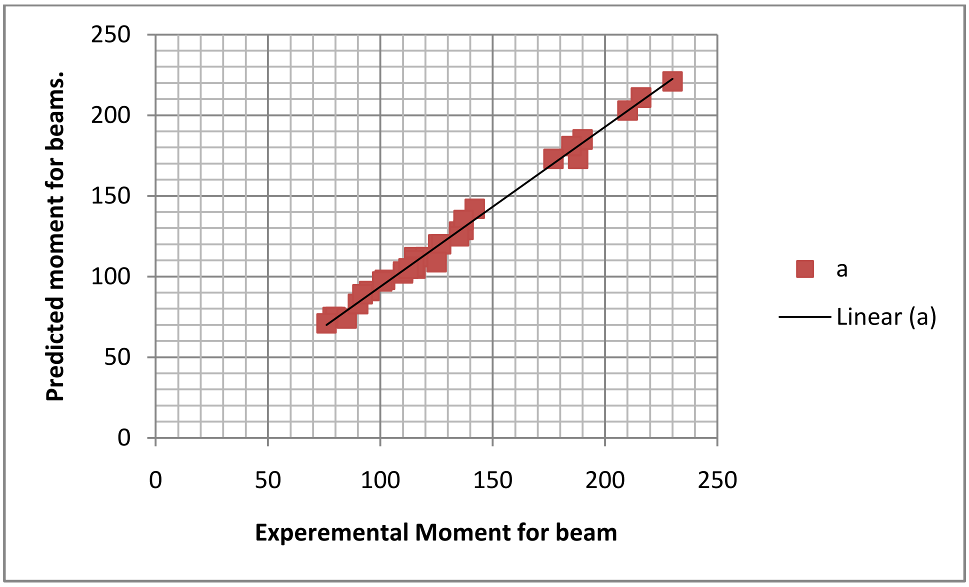

Fig2. Comparison experimental and predicted values for training data set.

Here,bending moment from ansys analysis is beng plotted in X axies and predicted bending moment for same beams are plotted in Y axies.The results shows slight difference in this two type forms.So there might be some error in analysis or modeling by ANN method The correlation coefficient R is being evaluated from this graph and its value is 0.9951.from statistical analysis by Excell.

3.2 Design Equitions Of Ferrocement

Design equation for ferrocement beam is being obtained from mathematical regression method from the database of the experimental work.The algorithem of the equation is given below.

So the final shear force equation is kn=x(1)+2*x(2)*lo+x(3)*l

So the final bending moment equation is knm=x(1)+2*x(2)*lo+x(3)*l*(y/2)

From this model equation we can fiend bending moment of any type ferrocement beam section for any type loading.

3.3 Simulation Of Appropriate Model

By applying ANN method using (MATLAB toolbox) simulation of the appropriate model of ferrocemen beams subjected to point loading at the middle of the beam presented below.

Table 11: Detailed Summary

Beam	Type	Loading	Length (ft)	Xsection Area(in^2)	Shear force	Bending moment	Type
B^{B1}	Box section	70 KN	5	60	47.9876	77.4708	Simple supported
B^{B2}	Rectangle section	70 KN	5	60	57.3349	96.8056	Simple supported
B^{B3}	Channel section	70 KN	5	60	46.1082	113.5261	Simple supported
B^{B4}	I section	70 KN	5	60	50.5870	103.0740	Simple supported

3.4 Comparison Between Results And Laboratory Tests Of Some Previous Reserches

Main objective of this part is to compare test results with laboratory test results from previous reserches.We have used I section beam results from previous reserches to compare the Ansys analysis results to the laboratory test results.Very little information is available in literature regarding ferrocement I-section beam. Thus, the main objective of this part is to study the behavior of ferrocement I-section beam, reinforced with different percentage of tensile reinforcements with different sizes. To carry out the investigation two prototypes of conventional ferrocement I-section beam were constructed and tested in the laboratory. Beside this, some of the materials properties obtained from the performed experiment are frequently used in the modeling. Uniformly distributing load was applied during testing in UTM (Universal Testing Machine). The dimensions of the ferrocement I-section beam were 14" x 10" x 2".Two sections were made and tested in laboratory.

Parametric Study taking beam (C^3) Section 12" x 8" x 1"

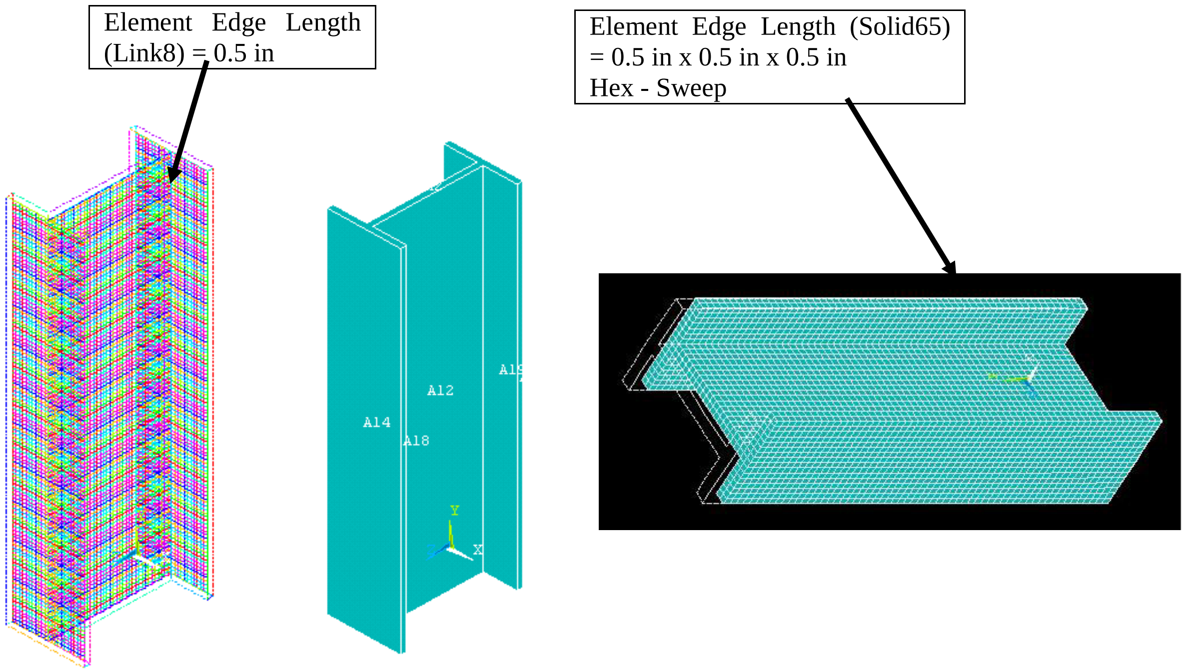

Figure 3: Plotting of beam

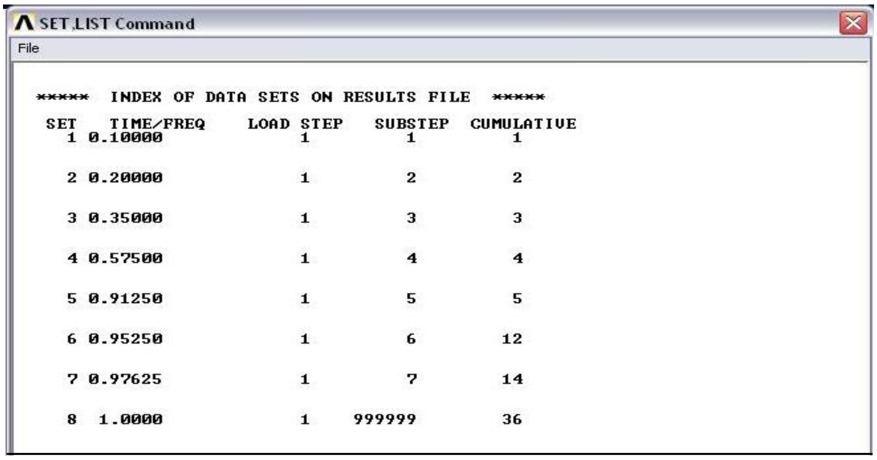

Figure 4: Detailed Summary

Table 11: Load and deflection values for C3

Total load on Beam (kN)	**Vertical Deflection (mm)**
26	0.17
52	0.34
75	0.52
101	0.68
127	0.85
153	1.02
179	1.2
205	1.36

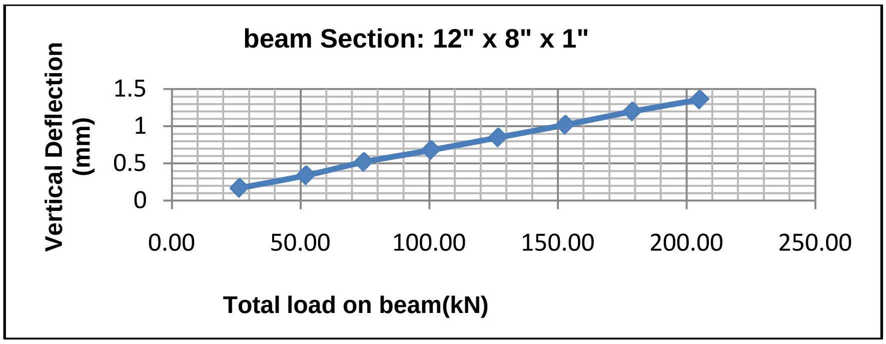

Figure 5: Load deflection curve

3.7 Result

From experimental programme it has been observed that the maximum deflection of the ferrocement beam is 0.0122 inch and average failure load was 10320lb. These properties were also compared to the result of analytical studies. Analytical result shows that the deflection of the beam is .0084inch and failure load was

13800lb. Failure load from experimental program was .49 times of average failure load from ANSYS analysis. The observation of an ANSYS model with the experimental result in load deflection curve shows it is comparable.Failure load from ANSYS analysis was 49% higher than average failure load from the test. The measured shortening of beam values versus applied loads for the ferrocement beam in experiment and ANSYS model analysis are plotted. So, from load deflection curve it is also proved that model made in ANSYS is most of all correct and acceptable to all.The design equitions for ferrocement beams have been developed. Model of shear strength and bending strength prediction by employing ANN method of ferrocement beams with forces acting along the plane and out of plan was done. By analyzing ferrocement sections using finite element method and appropriate equations, design charts have been introduced for ferrocement beam sections (rectangular, I, channel and box) to serve as an aid for a prospective designer to arrive at a section to suit their requirements.By applying ANN method using (MATLAB toolbox) simulation of the appropriate model of ferrocement beams and slabs subjected to different types of loadings presented in several researches have been done.From simulation ,we have found that I section beam is more economical and can carry more shear and bendindg moment than other type beam sections.So it is recommended that I section beam is more suitable for use in practical field.

3.8 Discussion

This project was finished with the help of some practical tests from previous researches and models made by ANSYS 11.0. The data and specifications were imputed in ANSYS model according to the given examples (see Appendix A and B). The analytical result shows linearity in the graphical representation. There is a variation in the result of experimental program and analytical study. It happened due to same property in all elements of finite elementary model and possible unequal property in test prototype. Edges and surfaces were not as smooth as required due to manual labor. Thickness of the whole structure was not exactly same and quality of materials and position of reinforcing meshes was not the same as required. Also there were some limitations when test in laboratory to make it exact as model in ANSYS. Observation between experimental results and analytical results shows acceptable variations. This project was also finished with the help of some models made by ANSYS 11 and simulations made by MATLAB toolbox. The data and specification were imputed in ANSYS model and MATLAB toolbox according to the given examples (see Appendix A , B and C). Then results were obtained and checked. The behavior of model in ANSYS was most of all similar to the practical prototype. In results some variations were observed and these happened due to the some faults occurred during modeling from previous researches such as beam and slab edges were not absolutely plain and thickness were not same throughout the length. Also there were some limitations when test in laboratory to make it exact as model in ANSYS. Capping of beam was not done during test and for test in UTM it was needed to short the beam about 0.6" in length and repair it which was cured only 3 days. If all limitations could be avoided then it hopes that test prototype shows more nearest results as ANSYS model analysis. Applying ANN method, there may be some correction needed. In ANN method the hidden layer property would be slight different. So there is a possible chance for wrong results. Though compression shows nearly accurate results from analysis and ANN simulation results. Simulation works were nearly appropriate, because those were done in software.

4. CONCLUSION

There are several reasons for widespread use of ferrocement. On the construction side, it can be fabricated into almost any shape, skills needed for construction can be easily acquired, heavy plant and machinery is not required and easy to repair. Meanwhile, on the material side, ferrocement possesses a degree of toughness, ductility, strength and crack resistance that is considerably greater than that found in other forms of concrete construction.Ferrocement is being using in the construction sides now a days more rapidly than previous time.It is also economically friendly to use.Cost analysis and its life expectances clearly shows that it is more economical and structurally safe to use besides normal reinforced concrete.There are many sectors in constructions where it can be easily used.

In this study the Ansys models and ANN models are developed to predict the shear force and moment capacity of ferrocement members. A database of (43) tests on ferrocement members is developed from the review of literature and some new tests are

used for training and testing of this model, and four variables are selected as input of ANN model. In the developed ANN model, the predictions are made using Mathlab toolbox as well as available methods, and they are also compared with actual measurements. From the comparison they are found to agree much better with the actual measured values as compared with other thesis data. In the developed ANN, the same input variables are used in equation model. Types and numbers of membership functions to each input variable are appropriately decided when testing data set provides the highest correlation R. Moment predictions are made using ANN as well as commonly used empirical methods, and they are also compared with actual measurements.

5. RECOMMENDATION FOR FUTURE STUDY

This work expands the path of modeling of ferrocement beams and also its analysis. This report may help designers providing the principles and guidelines to aid in the optimization in an easier manner. As the presented report is not sufficient to fulfill all the objectives further study and experimental programme with different layers of wire mesh and different span to depth ratio of beams and different voids sizes are needed to complete this study and to get a satisfactory result.It is required to investigate more and more taking ferrocement as beam to know its full behavior as structural member, because it has several advantages (see chapter 2) those are mostly required now a days. It is also suggested to check how to reduce cracks developed in the flange portion and joints of the flange and web. ANSYS analysis can be carried out for the same. To ensure that the finite element model is producing results that can be used for study, any model should be calibrated with good experimental data. This will then provide the proper modeling parameters needed for later use. So finally, it can be said that, it is not the end but initiation of the optimization task.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECT OF FLY ASH AND WATER CEMENT RATIO ON COMPRESSIVE STRENGTH AND BOND STRENGTH OF CONCRETE AT ELEVATED TEMPERATURE

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ABSTRACT

An experimental investigation was conducted in order to evaluate the effect of fly ash and water cement ratio on compressive strength and bond strength of concrete at elevated temperature. The blended cement used in this investigation consists of ordinary Portland cement (OPC) and fly ash. The OPC were partially replaced by 0%, 15%, 20%, 25%, 30%, 35% and 40% fly ash. The blended concrete pastes were prepared using the water cement ratio of 0.35, 0.40, 0.45, 0.50 and 0.55. The fresh concrete mixes were first cured at 100% relative humidity for 24 hours and then cured in water for 28 days. The hardened concrete was thermally treated at 100^{o}C, 200^{o}C, 300^{o}C, 450^{o}C and 600^{o}C for 2 hours. The compressive strength and bond strength of fly ash concrete were compared with those of the pure ordinary Portland concrete. The result showed that the addition of fly ash to OPC improves the performance of the produced blended concrete when exposed to elevated temperature up to 600^{o}C. Changes of water cement ratio for the concrete mixes have significant influence on compressive and bond strength.

Keywords*: Fly ash, elevated temperature, pozzolanic reaction , compressive strength, bond strength.*

1. INTRODUCTION

For good performance, ecological and economical reason, concrete containing mineral admixtures is used extensively throughout the world. In addition to Portland cement fly ash, ground granulated blast furnace slag, silica fume and rice husk ash are most commonly used cementitious materials that are used as concrete constituents. They save energy, preserve resources and have many technical benefits (Arioz, 2007). Owing to its low thermal conductivity and high specific heat, concrete is well known for its capacity to tolerate high temperatures and fires. However, it does not mean that fire as well as higher temperatures does not influence the concrete. Recent studies show that concrete characteristics such as color, compressive strength, elasticity, density and surface appearance are affected when concrete is exposed to fire or experiences high temperature (Xu et al., 2001; Tanyildizi and Coskun, 2008; Morsy et al., 2009). For that reason, improving concrete's fire resistance is a field of interest for many researchers recently. Cement replacement with pozzolanic materials is considered one of the very competent methods and several studies have also been conducted to this purpose (Yeginobali et al., 1997; Poon et al., 2001; Poon et al., 2003). Poon et al. (2003) concluded that the addition of silica fume to concrete densifies the pore structure of concrete, which can result in explosive spalling due to the build-up of pore pressure by steam. The addition of polypropylene fibers in a concrete mix is also found to be useful (Noumowe, 2005). However, the main attribution to thermal properties of concrete is provided by aggregates (Savva, et al., 2005).

In concrete, physically absorbed water evaporates due to the rise in temperature which induces thermal cracks due to thermal shrinkage. As a result concrete shows inferior performance at elevated temperatures as compared to concrete at normal temperatures. The physical changes and chemical decomposition of major concrete constituents at elevated temperatures is demonstrated in widely spread cracks, explosive spalling or both. As human safety in case of fire is one of the key considerations in the design of buildings, it is mainly necessary to have complete information about the behavior of all construction materials before using them in the structural elements.

It is considered that mineral admixtures can significantly modify the weak interfacial transition zone between cement paste and coarse aggregates in a concrete mix an experimental program was conducted on incorporating fly ash and silica fume in the concrete mix subjected to high temperatures. This paper is aimed at investigating the effect water cement ratio and fly ash as replacement on the performance of mechanical properties such as compressive and bond strength subjected to elevated temperature, containing fly ash as pozzolanic materials. These properties are very significant for a safe design of concrete as well as the repair and recovery of the post fire concrete structures (Morsy et al., 2000).

2. EXPERIMENTAL PROGRAM

2.1 Materials Used

For the subject experimental program necessary materials like cement, coarse and fine aggregates was procured form local market. Ordinary Portland cement (ASTM Type-I), Domar sand and crushed black stone are used for the investigations. The mineral admixture, fly ash was collected from Barapukuria 2X125MW Coal Fired Power Plant at Dinajpur. According to the ASTM Fly ash category, used Barapukuria fly ash was classified as F class for the low calcium contain. Laboratory supplied water was used for both production and curing of test specimens. Undertake necessary testing of concrete materials in the laboratory following ASTM codes accordingly. Chemical composition test of the fly ash was collocated from the Barapukuria 2X125MW Coal Fired Power Plant is presented in the Table 1.

Table 1: Chemical and physical properties of Barapukira fly ash

Chemical – Physical Properties	% by mass
Al_2O_3	54.4
SiO_2	35.6
Fe_2O_3	2.9
TiO_2	3.2
CaO	0.56
MgO	0.18
Specific surface area (cm^2/g)	2724
Specific gravity (g/cm^3)	2.24

2.2 Experimental Variables

In this study cement was replaced with different percentage of fly ash and also the water/cement ratio varied from 0.35 to 0.55. After curing different specimens were tested after heating them at different temperature ranging between 25 to 600^oC. The experimental variables are listed in Table 2.

Table 2: Experimental variables

Items	Variables
Materials	Portland cement, sand and water
Mix ratio of cement to sand	1:3
Water/cement ratio	0.35, 0.40, 0.45, 0.50, 0.55
Specimen size	50×50×50mm Cube, 300mm steel bar with 5.5mm dia
Heating temperature	25^oC, 50 ^oC, 100 ^oC, 200 ^oC, 300 ^oC, 450 ^oC, 600 ^oC
Fly-ash content	0%, 15%, 20%, 25%, 30%, 35%, 40%

2.3 Curing of Specimens

The specimens were removed from moulds after 24 hours. All the specimens were carefully cured by clean water available in the laboratory. The test specimens were moist cured for 28 days as shown in Figure 1 at room temperature as required in this study.

Figure 1: Curing of specimens

2.4 Heating of Specimens

After 28 days curing, the specimens were taken out from water and allow the specimen to dry up for several days. To observe the temperature susceptibility of cement mortar cubes the cube was heated in electric oven and furnace (Figure 2). The specimens were heated at five different temperatures 100ºC, 200ºC, 300ºC, 450ºC, and 600ºC respectively. Generally, the specimens were kept at the oven for two hours for each temperature. The specimens were removed from the oven immediately after the application of heat and allow it to cool down at normal temperature in the laboratory.

Figure 2: Heating arrangement (a) electric oven (b) electric furnace

2.5 Testing of Specimens

Compressive testing machine was used for testing compressive strength. The cubes without were tested for compressive strength. The value of ultimate failure load from the dial gauge was observed. Figure 3(a) shows the photographic view of compression testing setup. For testing of bond strength, “Torsee’s Universal testing machine” was used. The model no. was AMU-5. Its capacity is 50,000 N and leastcount 10N. The model cubes were hold properly by a special steel frame and load was applied gradually. The values of ultimate or failure load from the dial gauge reading were observed. Figure 3(b)shows the set up for bond strength and the schematic diagram of steel frame used in the bond test.

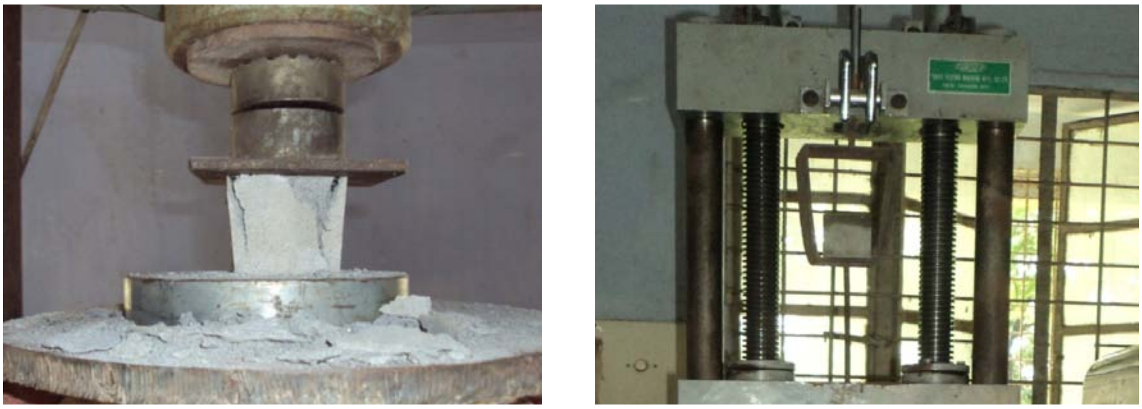


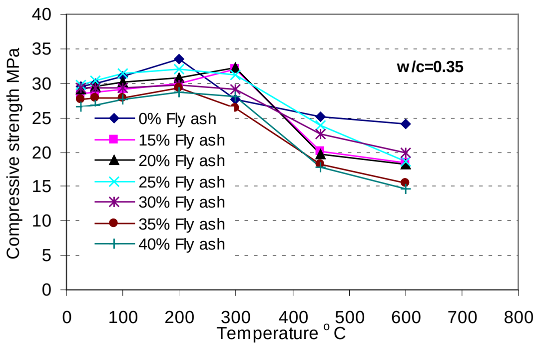
Figure 3: (a) Compressive strength experimental set up (b) Bond strength experimental set up

3. RESULTS AND DISCUSSION

Temperature is one of the main factors that influence the strength of mortar and concrete. High temperature induces a loss of strength (compression, tension and bond) and stiffness (Young's modulus). At high temperatures, chemical change of the gel weakened the matrix bonding, which brought about a loss of strength of fly ash mortar and concrete.

3.1 Compressive Strength

The results obtained from compressive strength tests on concrete mixes at temperatures 100ºC, 200ºC, 300ºC 450ºC, and 600ºC respectively are shown in Figures 4 - 8.



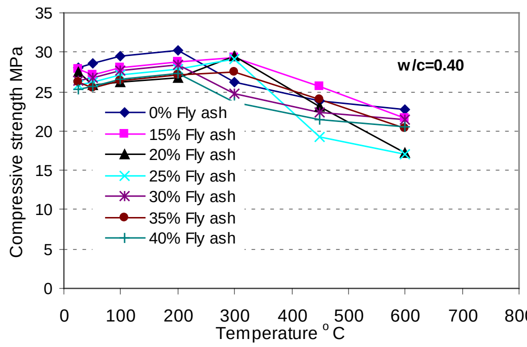
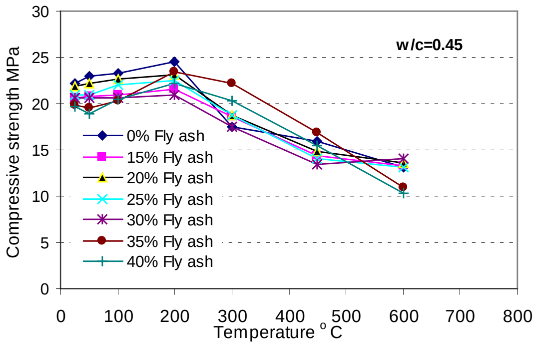


Figure 4: Compressive strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.35

Figure 5: Compressive strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.40



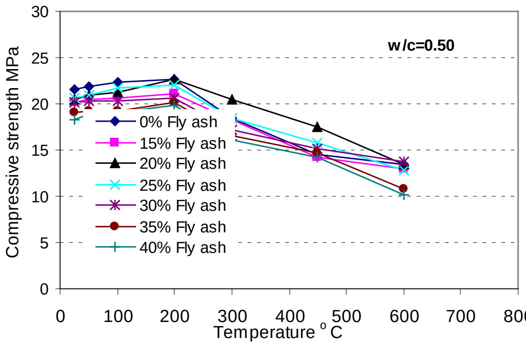


Figure 6: Compressive strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.45

Figure 7: Compressive strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.50

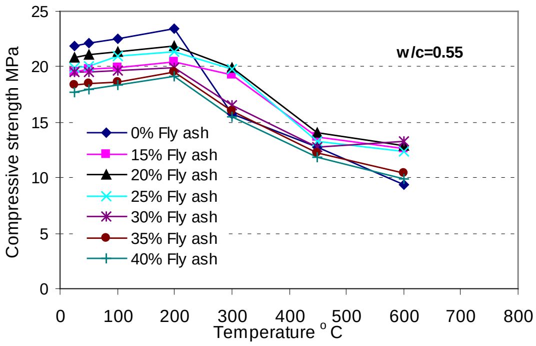


Figure 8: Compressive strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.55

The result shows a relative decrease in the compressive strength of each specimen thermally treated up to 600°C as compared to its original compressive strength before heating. From the outlook of residual compressive strength of concrete, the heating conditions can be divided into four regions as 25 – 200°C, 200-300°C, 300 – 450°C and 450 – 600 °C. Smooth patterns of strength gain were observed in first region, in second region distinct patterns of both loss and gain were observed and then subsequent sharp and flat loss in the third and forth region were found. In general, the compressive strength of concrete mixes is increased and decreased by various proportions as a result of exposure to high temperatures. For reference concrete mix at (considering 0% fly ash at 25°C as 100% strength), the residual compressive strengths are about 105%, 115%, 95%, 70% at temperatures of 100°C, 200°C, 300°C and 450° C respectively. Where as concrete mixes containing fly ash shows the inferior results i.e 90-95%, 90-95%, 95-110%, 60-65% for the temperatures of 100°C, 200°C, 300°C and 450° C respectively. The result at 300°C for the specimens containing fly ash between 15- 25% shows better performance than reference specimen. However the increase in compressive strength of blended concrete containing 15%, 20%, and 25% fly ash around 300°C, was mainly due to pozzolanic reaction; which led to the formation of additional amount of hydration products. At high temperatures, especially above 100°C, the thermal effect might cause water migration whereas dehydration of moisture supply from outside is insufficient. Internal stress and thus micro and miacrocracks are generated due to the heterogeneous volume dilatations of ingredients and the buildup of vapor in the pores. Therefore, at higher temperature, especially above 400°C, the observed decrease in compressive strength of blended concrete containing 15-40% fly ash, may be due to internal thermal stress generated around pores which generate microcracks.

3.2 Bond Strength

Figures 9 to 13 stated as under representing the bond strength of test specimens containing fly ash 0%, 15%, 20%, 25%, 30%, 35% and 40% after heated in a furnace up to 600°C .

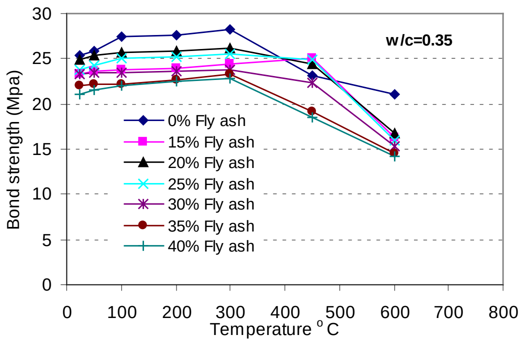


Figure 9: Bond strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.35

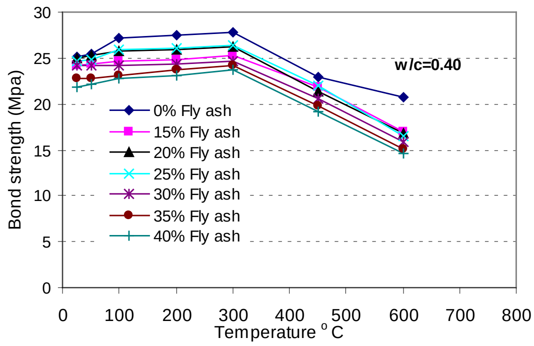
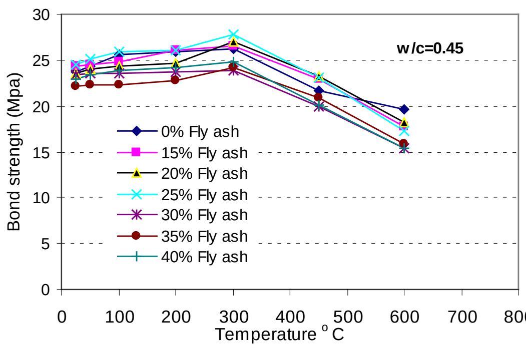


Figure 10: Bond strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.40



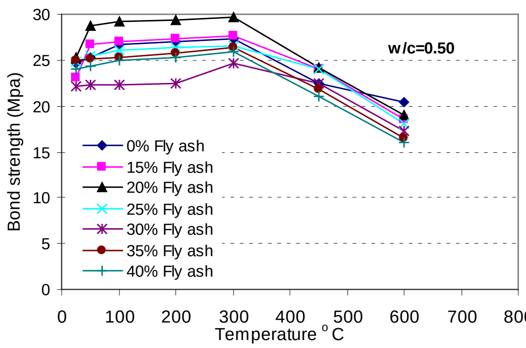


Figure 11: Bond strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.45

Figure 12: Bond strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.50

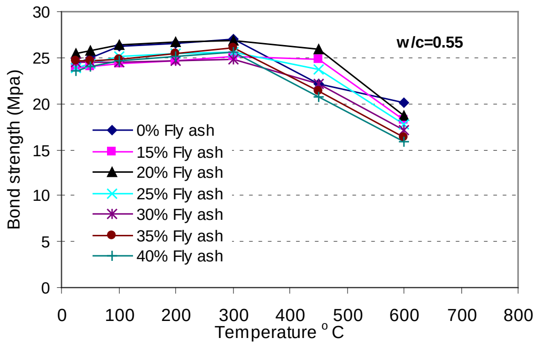


Figure 13: Bond strength of concrete exposed to elevated temperature after cooling at water cement ratio 0.55

From the experimental results it can be seen that fly ash containing specimens shows the inferior result with respect to reference specimen (considering 0% fly ash at 25°C as 100% strength) at normal temperature condition. The reference specimen shows sharp increase in bond strength upto 300°C. Reference concrete mix specimens are showing 105%, 110%, 115%, 90% bond strength at temperatures of 100°C, 200°C, 300°C and 450°C respectively. Where as concrete mixes containing fly ash shows the inferior results i.e between 60-90% strength for the temperatures of 100°C, 200°C, 300°C and 450° C respectively. From the perspective of residual compressive strength of concrete, the heating conditions can be divided into two regions as 25 – 300°C, 300-600°C. Smooth patterns of strength gain were observed in first region, in second region sharp patterns of strength losses were observed. 20% fly ash mix specimen showing best performance among the others. It can also be noticed that addition of fly ash also improves the performance of the concrete up to 300°C.

4. CONCLUSIONS

The following conclusions can be drawn from the present study: incorporation of fly ash has mainly two types effects on the compressive and bond strength when exposed to elevated temperature of different degrees upto 600°C. Under normal temperature condition, the effect of addition of fly ash both on the compressive and bond strength can be treated as detrimental as there is strength loss. However, at elevated temperature upto 300°C some positive effect of the cement replacement with fly ash also observed. The pozzolanic reaction of fly ash accelerated between 200 and 300°C; this may be due to the decrease of Ca $(OH)_2$ content. Based on the mechanical and physical properties of fly ash concrete, it was observed that 20% fly ash concrete was generally more constructive than 15, 20, 25, 35 and 40%, which is useful in structural elements exposed to elevated temperature up to 300°C. Traditional effect of water cement ratio on the strength has been observed through out the investigation.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

2011 SIKKIM EARTHQUAKE: A CASE STUDY AND THE NECESSITY OF MONITORING OF AGED STRUCTURES

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ABSTRACT

The September 18, 2011 Sikkim earthquake, also known as the 2011 Himalayan earthquake, was a magnitude 6.9 earthquake centered near the border of Nepal and the Indian state of Sikkim. The earthquake struck Bangladesh and its surrounding area (Nepal, north-eastern India, Bhutan, and southern Tibet), at 18:40 BdST is one of the major earthquakes in the history. The earthquake was felt most strongly in northern part of the country and no casualty was reported. However, several people were injured during rush out and several buildings were tilted and cracks also developed. A field investigation was conducted to investigate the consequences of the earthquake on the major and aged buildings of Rajshahi City. The investigation method was mainly based on the visual inspection and photographic examination. This paper summarizes the investigation findings. No major damage was found in the examined buildings except one which aged about 50 years and received severe damage. This damaged building was thoroughly investigated and the examination report reveals that poor construction materials used as well as the violation of building code triggered the damage during the earthquake. The findings of this investigation also clearly reveal that there is a need of assessing and monitoring of the structures specially the important and aged ones.

Key words: *Earthquake, assessment of structure, monitoring of structure, visual inspection, aged structure*

1. INTRODUCTION

In the recent past, Bangladesh did not experience any devastating disaster due to earthquake (Ansary, 2009) although the country has been jolted several times. It is well known that Bangladesh is one of the most disaster prone countries in the world. Bangladesh is located in a region of significant seismic activity but most of the people do not believe that seismic risk to be of great importance. Historically, earthquakes in the M6.0-7.0 range have been experienced in Chittagong, Dhaka and Sylhet divisions while events in the M5.0-6.0 range have been experienced in Khulna and Rajshahi divisions. It is also a matter of major concern that the 1885 earthquake of Manikganj, the 1897 earthquake of Great Assam, the 1918 earthquake of Srimangal, the 1930 earthquake of Dubai, and the 1950 earthquake of Assam are all quite matured to recur at any time and may create devastation (Ali, 1998). Moreover, records of the earthquakes show that Bangladesh and its surrounding areas experienced at least 1000 earthquakes having magnitude greater than or equal to 4 in Reichter scale in the last 100 years (Ansary, 2009).

Recently, some serious and horrible earthquakes occurred in Bangladesh have made the people worried. Experts in the relevant areas think that these earthquakes are alarming and larger earthquake is due which can harm Bangladesh horribly. In most of the cities, the building can be classified into two broad categories engineered and non-engineered buildings. Like many other countries the number of aging structures is also increasing. Moreover, the old structures do not cover the modern building codes and needs retrofitting or strengthening to reduce risk as they can not be demolished. It is also important that the old structures should be monitored periodically or at least after some big event like earthquake (Wu, 2003). For example, after the 4.1M earthquake on 5 July 2008 occurred on the Bangladesh-India border the Bangladesh Network for Urban Safety (BNUS) team conducted a survey and found about 30 buildings developed cracks and a 6-storey building "tilted" in the division (Figure 1). This quake was also felt in parts of West Bengal in India. The earthquake was centered 6.4 kms WNW of Rajshahi, Bangladesh. In fact, such investigation can help identify the weak structures as well as the experience can be good learning and help improve future design of the structures. However, similar activities are very new in Bangladesh. In this paper, an investigation report of the field survey conducted after the well known Sikkim 2011 earthquake is presented.

Figure 1: Photograph of some affected buildings in Rajshahi district after the earthquake on July 5, 2008

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2. 2011 SIKKIM EARTHQUAKE

The September 18, 2011 Sikkim earthquake, also known as the 2011 Himalayan earthquake, was a magnitude 6.9 earthquake centered near the border of Nepal and the Indian state of Sikkim. The earthquake struck Bangladesh and its surrounding area (Nepal, north-eastern India, Bhutan, and southern Tibet), at 18:40 BdST is one of the major earthquakes in the history. The preliminary hypo-central parameters of this earthquake, as estimated by the Seismic Monitoring Network of India Meteorological Department (IMD) are given below:

Table 1: Details of the Sikkim 2011 earthquake (Source: URL 1)

Date of occurrence	18/09/2011
Time	18:40 hrs (BdST)
Magnitude	6.9
Focal depth	10 Km
Epicentre: Latitude and longitude	27.7^{o} N and 88.2^{o} E
Region	Sikkim-Nepal Border region
Casualty	At least 111 [India (97); Nepal (6); China(7); Bhutan (1)]

The earthquake was located at a shallow depth beneath the surface and caused strong shaking in many areas adjacent to its epicenter reportedly lasting 30–40 seconds (Source: URL 1). No casualty was reported in Bangladesh. However, more than 111 people were killed in this earthquake in India, China, Nepal and Bhutan and hundreds of others sustained injuries. As the earthquake occurred in the monsoon season, heavy rain and landslides rendered rescue work more difficult. The earthquake caused significant damaged in India. The strong shaking caused significant building collapse. As a result, tens of thousands of residents evacuated their homes, and many areas suffered from communication and power outages. In India, property damage was primarily estimated to around US$ 20.28 billion. In Bangladesh, no significant damage was reported soon after except some cracks and tilted building. For instances, in the capital Dhaka cracks developed in at least two multi-story buildings and in Bogra two buildings were tilted (Figure 2). Also people were panicked and try rushing out of their home and offices and get injured (Source URL 2-4).

Figure 2: Tilting of two 7-stories and 5-stories building in Bogra (Source: URL5)

3. FIELD INVESTIGATION AFTER THE 2011 SIKKIM EARTHQUAKE

3.1 Description of the Study Area

What an earthquake can cause to human civilization can easily be understand from the experience of Japan in 2011. The 2001 Bhuj earthquake is another good example which causes medium to severe damage to over 980,000 masonry structures and collapse of 230,000 in Gujrat located 70 km away from the epicenter and 90% of masonry structures of old Bhuj collapsed and most of the reinforced concrete structures were badly damaged (Jain et al., 2002). The 2011 Sikkim earthquake created much panic to the people. So, after the 2011 Sikkim earthquake to estimate the damage caused to buildings and other structures a field survey was conducted in Rajshahi city (Figure 3).

Geographical coordinates of Rajshahi are 24° 22' 0" North, 88° 36' 0" East. Rajshahi city is located about 400 km from the epicenter. The city of Rajshahi is the divisional headquarters of Rajshahi division as well as the administrative district that bears its name and is one of the six metropolitan cities of Bangladesh. From the earthquake zoning map of Bangladesh (Figure 4), it is clarified that Rajshahi is under lower middle risk zone. Rajshahi is under the zone-2 and zone-3. Where the basic seismic co-efficient are 0.05 and 0.04 respectively. The city is one of the major cities in the country and recent growth of the city is very rapid. Majority of the old buildings are about 2-4 storied masonry buildings. However, the new trend of construction is to construct framed building. Again by considering population and the urbanization of Rajshahi city and corresponding area it is generally considered that the risk of living in this city is less compared to other city like Dhaka or Chittagong. From the computer intensity map of USGS (Figure 5) it can be noticed that the intensity of the 2011 Sikkim earthquake in Rajshahi was estimated about IV-V. With this shaking intensity a structure will suffer from no damage to little damage. From the conducted field survey no damage was found in the buildings except one which is located in the Rajshahi University of Engineering and Technology. Therefore, this study report is primarily aimed at focusing the damage survey study of this particular building, Shahid Lt. Selim Hall.

3.2 Shaid Lt. Selim Hall

Shahid Lt. Selim hall is a three storied building of about 50 years age. More than 350 students reside in this student dormitory. The building was constructed with load bearing brick masonry wall as well as reinforced columns. A one storey dinning hall as shown in Figure 6 is also situated in the dormitory building complex. There are 15 columns in the dining hall and each column is of 12 inch diameter. The unsupported length of the column is 13 ft. The columns are equally spaced and connected by beam at the roof level. Detail of the reinforcement was not found. The main building is a masonry building with a 8 ft wide veranda oriented in the east-west direction. The veranda columns are equally spaced on 11.25 ft span having cross section 12 inch by 18 inch. There are 192 veranda columns in four rows and in three levels. A continuous railing of 3 ft height is seen in the veranda.

Figure 3: Map of the Rajshahi city

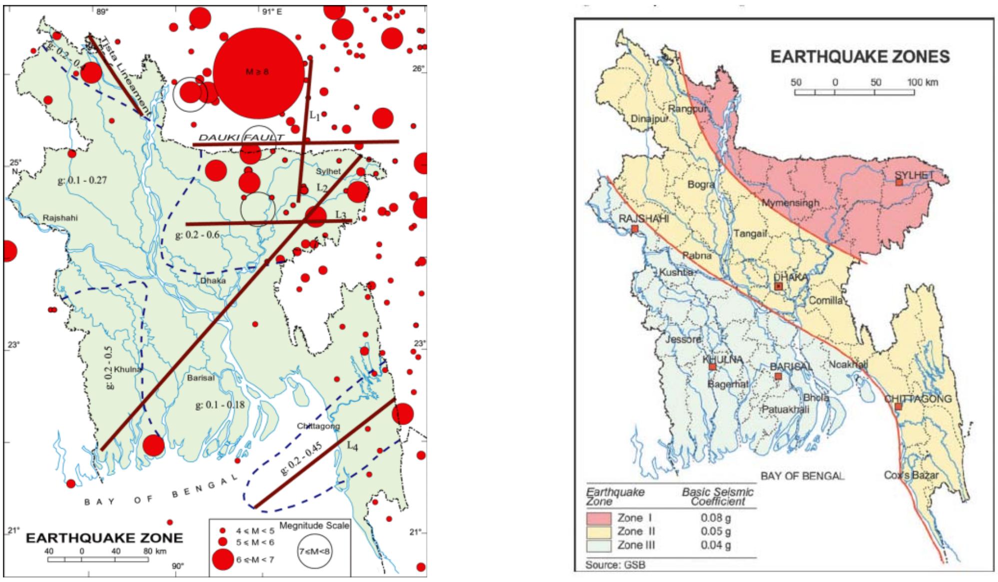

Figure 4: (a) Major earthquakes in Bangladesh (b) earthquake zoning map of Bangladesh (Source: BNBC, 1993)

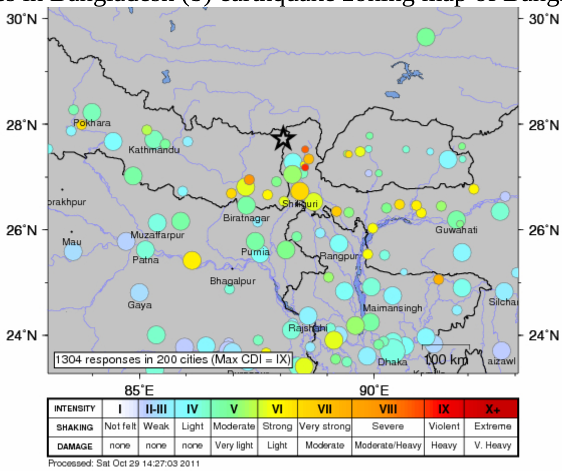

Figure 5: Intensity map of the 2011 Sikkim earthquake (Source: USGS Computer intensity map)

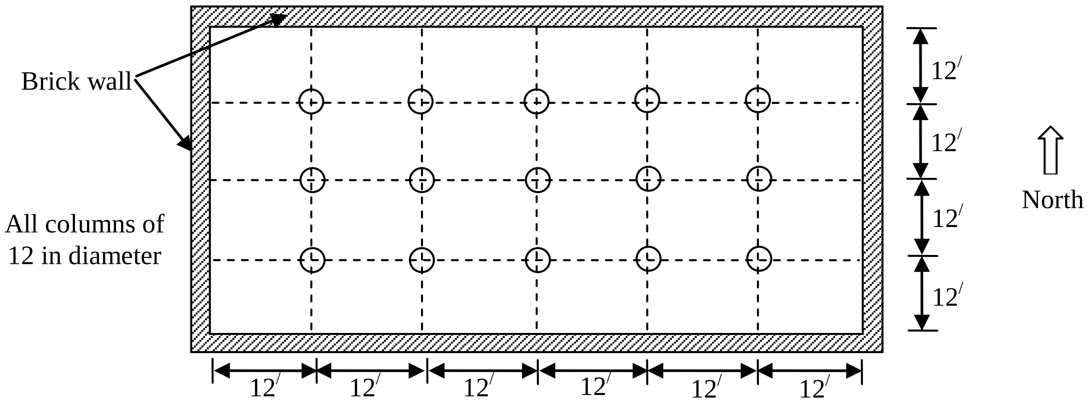

Figure 6: Plan of the dining hall

3.3 Observations from the Field Investigation

Visual inspection and photographic examination method was applied to evaluate the building. Some photographs from the investigation are shown in Figures 7-12. From the investigation cracks were found in the column and railing as well as crushing and spalling of concrete were also observed. It is interesting to note that the masonry wall was intact and no crack was found in any of the walls. The dining hall columns suffered most and out of fifteen columns thirteen columns were found medium to severe damaged. In some columns, the cracks extend upto four feet from the base (Figure7). The unsupported length of the dining hall columns is 13 ft and the diameter of the column is 12 inch. It is obvious that the slenderness ratio of the column is large enough to cause problem to the columns.

The veranda columns also suffered in the same way as it was observed in case of dining hall columns. Both spalling and crushing of concrete was noticed (Figures 7 and 8). In some cases the cracks extend up to four feet from the base. The crack was found in all three levels of the columns. The reinforced veranda railing also cracked during the earthquake. Spalling of concrete was also observed in veranda railing (Figure 12). All veranda columns are oriented in north-south direction and cracks were found on the inner side of the columns. No crack was found on the face that is exposed on the outer side. Later it was found that this type of cracking was mainly due to the formation of rust on the steel bars.

Close inspection of the damaged columns and spall out concrete clearly reveals that the primary reason of the failure of the columns (both dining and veranda columns) was mainly due to the rusting of the steel re-bars. Formation of rust was the primary reason for spalling of the concrete. The columns were initially weak and perhaps cracked inside. During the 2011 Sikkim earthquake, columns were overloaded and could not carry the extra load caused by the earthquake. As a result cracks were visible and widened and exposed on the surface were concrete was crushed. It is clear from the photograph (Figure 9) that the spirals were cracked. These failures of spirals may be due to the corrosion of steel re-bars or this may be the resultant of rusting and the shaking during the earthquake. Observation results also make it clear that the quality of the concrete used in the construction of the columns was not good. Additionally, there was ample evidence of insufficient consolidation of cast-in-place concrete i.e. honeycombing (Figure 10). The lack of proper seismic detailing was also evident in the damaged columns.

Comparable to the narrow dimension of the column the spacing of the transverse reinforcement (column ties) may be large, i.e the column ties are unable to provide sufficient confinement to the concrete core of the column. In addition to the poor quality concrete, formation of rust on the steel bars this large spacing of lateral reinforcement may also be responsible to accelerate the crushing of concrete and cracking of the column. However, it was not possible to create any more damage in the column and spacing of the ties could not be confirmed.

4. CONCLUSIONS

It is well known that "earthquakes don't kill people, buildings do.” The number of aged structures is increasing day by day all over the world. Many of these structures have been built without following any Code of practice. Also many of these structures fall out side the range of present building Codes. Therefore, it is obvious that to ensure safety monitoring of these structures as well as strengthening are very important issues. In this paper, a report based on the field investigation of a 45-year old building after the 2011 Sikkim earthquake is presented. The building suffered severe damage due to the earthquake. Cracks were developed in the columns as well as crushing and spalling of concrete was evident. Presence of crack in the columns due to the formation of crack may be the primary reason which made the column weaker and during the earthquake could not perform well. From the field investigation it is obvious that the building Code was not followed during the construction. It can also be concluded that to avoid future catastrophe, these types of buildings should be investigated and proper maintenance should be carried out.

Figure 7: Photograph of some damaged dining hall columns

Figure 8: Photograph of some damaged parts verandah column and part of railing

Figure 9: Breaking of the spiral bars of dining hall column

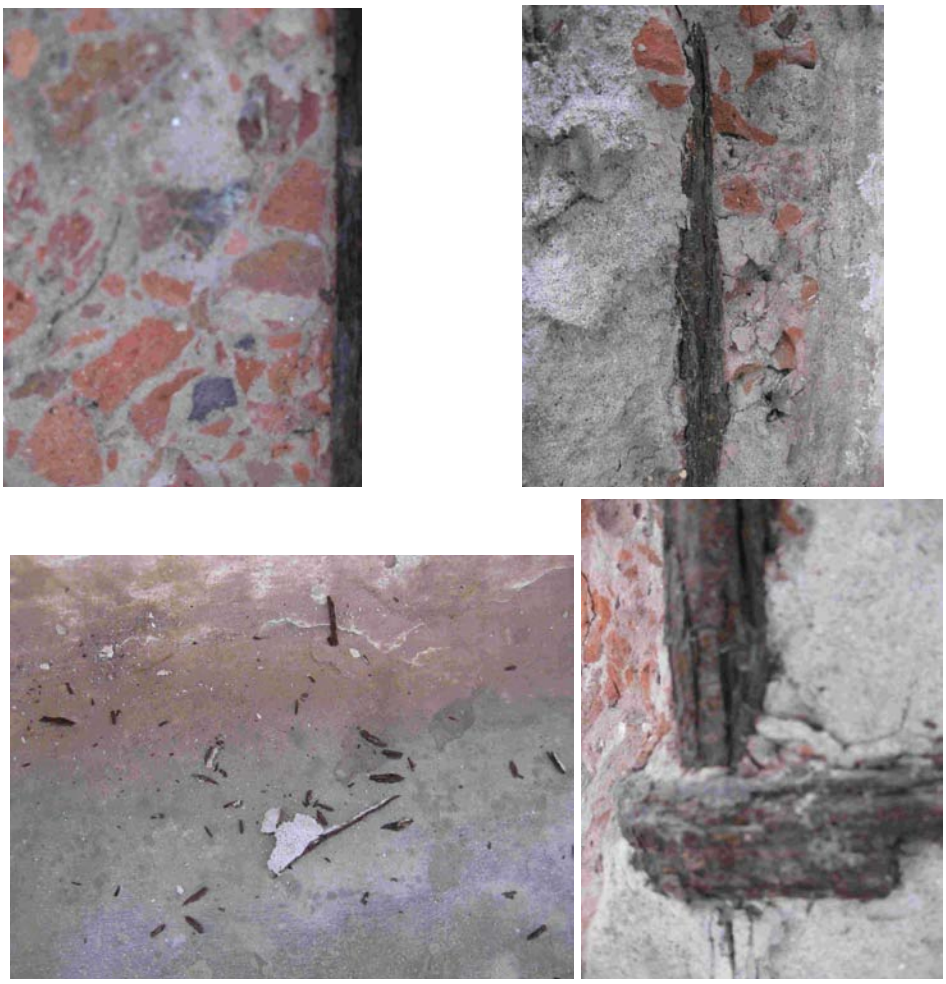

Figure 10: Detailed photographic view of dining hall columns

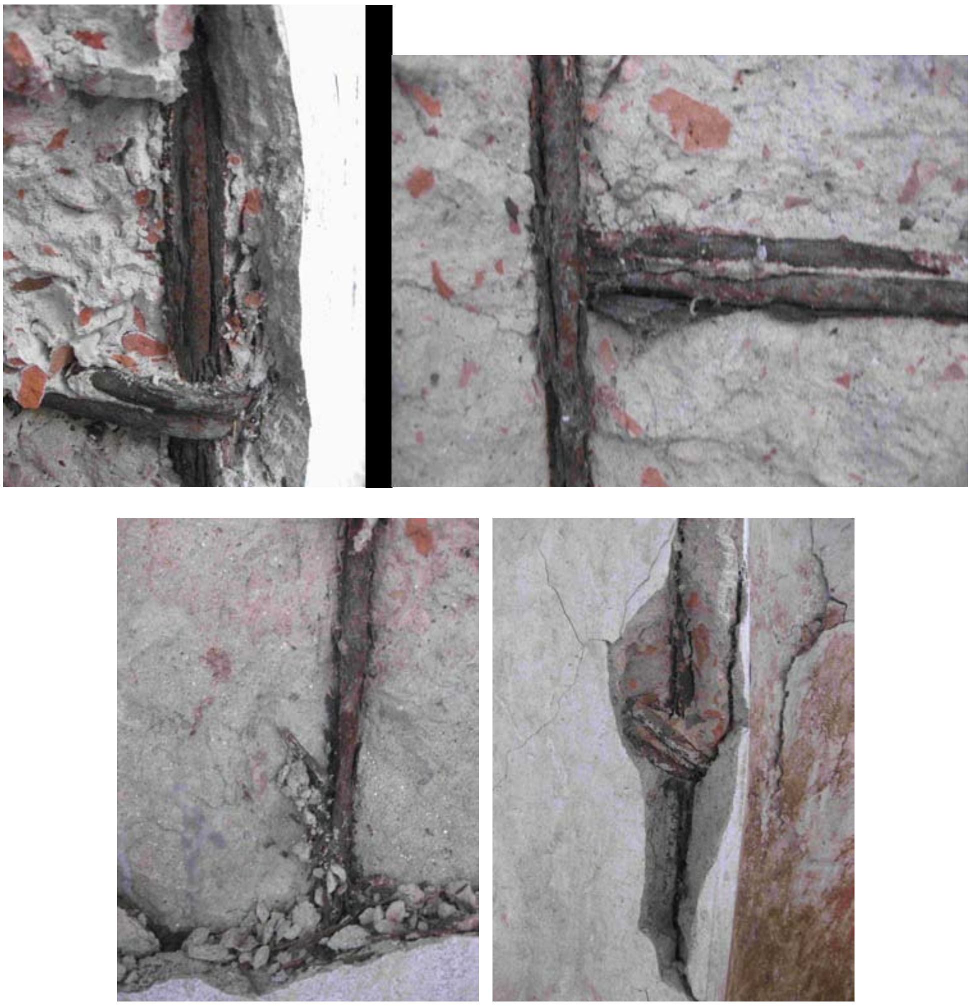

Figure 11: Detailed photographic view of veranda columns

Figure 12: Detailed photographic view of veranda railing

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

FINITE ELEMENT ANALYSIS OF COMPOSITE ROOF SYSTEM SUBJECTED TO VARIOUS LOADING BY ANSYS

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ABSTRACT

In recent years the use of advanced composite materials has gained wider space in the civil engineering sector, due to some favourable characteristics such as lightweight, high specific strength, resistance to corrosion and fatigue. In this work, a new slab system composed of brick masonry is analysed for finite element method by ANSYS 13.0. Brick masonry is one of the oldest building materials comparative superior to other alterative in terms of appearance, durability and cost. Brick masonry slab is durable, fire proofing, easy to construct, free from corrosion of reinforcement due to saline effect and in most cases it results in the increase of floor space due to adoption of brickwork or lesser thickness. In addition to tremendous use of bricks for low cost housing in Bangladesh, its use is getting popularity both in high rise structures and in factory buildings. As FEA (Finite Element Analysis) is a preferred method to analyse the behaviour of brick slab, commercial finite element analysis software ANSYS 13.0 is used to analyse the structural performance of the brick slab model under static and dynamic loading. Masonry slab under various load showed strong enough to tackle load induced on it.

Keywords: *composite roof system, masonry slab, static analysis, finite element analysis, ANSYS*

1. INTRODUCTION

The construction of stone, brick or tile which is hardened by heat is termed as masonry. It may be defined as building units bonded together with mortar [1]. Brick masonry is one of the oldest building materials comparative superior to other alternatives in terms of appearance, durability and cost (Hossain M. M. et al., 1997) [3]. In residential buildings, roof system is a vital part. Composite slab made of brick and plain concrete with or without beam are found to be practiced in locally made housing roof for low cost interest [2]. It was therefore felt to investigate the local carrying capacity of different type of masonry slab (without reinforcement). Different physical or mechanical properties are analyzed for brick slab by Finite Element Analysis (FEA) method under various static and dynamic loading by creating a model in ANSYS.

The Finite Element Analysis (FEA) method, originally introduced by Turner et al. (1956), is a powerful computational technique for approximate solutions having complex domains subjected to general boundary conditions [4]. In recent years, however, the use of finite element analysis has increased due to progressing knowledge and capabilities of computer software and hardware. It has now become the choice method to analyze structural components. The use of computer software to model these elements is much faster, and extremely cost-effective [5].

To fully understand the capabilities of finite element computer software, one must look back to experimental data and simple analysis. Executing the necessary checks along the way is a key to make sure that what is being output by the computer software is valid.

This study presents the mechanical properties of masonry slab. A masonry slab span of 5ft x 10ft with a thickness of 3 inches was analyzed by finite element method in ANSYS 13.0. The masonry slab was analyzed under uniformly distributed loading, concentrated loading, impact loading and cyclic loading. The results showed that deformation and stress level of masonry slab is within limit when subjected to more than eight times of standard slab loading. This makes the masonry slab safely usable in residential buildings as general slab without any application of reinforcement.

2. METHODOLOGY

Roof system of a residential building is an indispensable part .There are several type of roof system which are usually constructed in rural and urban areas namely, conventional R.C.C. slab beam, wooden rafter and beam covered with tile followed by lime surki mortar finish, brick masonry roof reinforced by MS bar or other indigenous material. Sometime unreinforced brick masonry is found to be constructed from long past. Effort of lowering cost has become burning need for low income group of people. Room with comparatively short span length is used in rural adobe buildings. For cost optimization and broaden utility, it possibility needs to be verified by full scale tests.

2.1 Experimental Details

Reinforced brick slab are widely used in low cost rural housing. Higher rate of corrosion in reinforcing steel and high cost of reinforcement has necessitated the study on brick slab without reinforcement for the interest of economy and durability of the slab. Rabbani and Nahid (2006) investigated the parametric study on more than 30 brick slabs without reinforcement [2]. Parameter included – brick line, span and filler. Figure 1 shows one of their typical laying pattern and Figure 2 shows the loading arrangement for the test of slab.

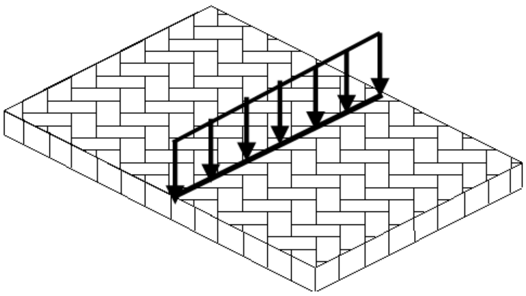

Figure 1: Model of herring bone bond slab of size (36 in×24 in×3 in)

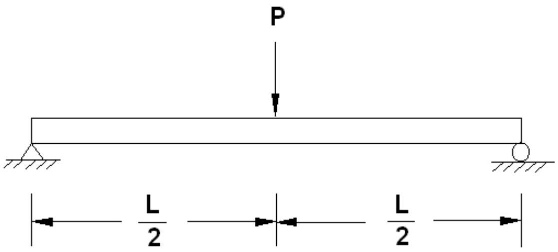

Figure 2: Loading arrangement of model slab

They concluded that herring bone bond masonry slab of 3 in thickness can resist maximum load, moment and stress. Therefore in this study a herring bone bond masonry slab of 120 in×60 in×3 in has been constructed and tested with uniform distributed load and repeated load.

2.1.1 Preparation of Test Slab

In this study, a two panel masonry slab each of 120 in×60 in×3 in are cast with brick module placed flat providing 3 in thickness for the slab. The interspaces between the modules (0.5 in) are sealed with mortar [12].

2.1.2 Materials Specifications

First class brick the average compressive strength 4000psi
Cement mortar ratio 1:1.5
Ordinary Portland Cement
Washed Local sand with fineness modulus of 1.5

2.1.3 Construction Sequences

First of all, wooden platform was prepared and leveled before laying the bricks. Bricks are then laid in staggering pattern placed with frog mark at to side keeping 0.5 in. Layout and support position of the masonry slab has shown in Figure 3. On the other hand, Figure 4 and Figure 5 show the detailing of the support size in cross-section and long section respectively. A 3 in thick slab was made keeping 0.5 in gap in between two adjacent bricks. Figure 6 shows a close view photograph of the same. Top surface of the slab was finished with

0.5 in mortar with neat finish. After 24 hours a 3 in height of brick border was made to store water for curing purposes. After completing 28 days of curing period the formwork was removed and the slab was prepared for test.

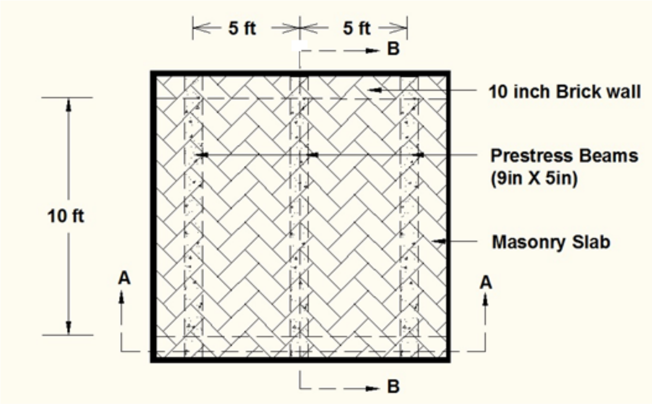

Figure 3: Layout and support position of slab

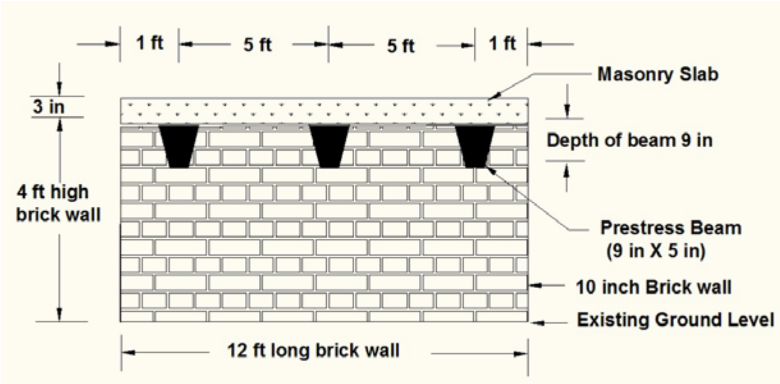

Figure 4: Section A-A of masonry slab

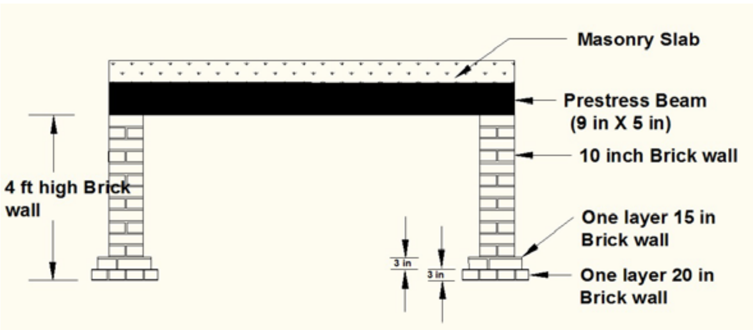

Figure 5: Section B-B of masonry slab

Figure 6: Photograph of Close view - showing the interspaces 0.5 in between bricks

2.1.4 Instrumentation and Testing

First Instrumentation and testing was performed in two phase. In first phase, only load bearing capacity of the full scale slab was tested and the test was done after 28 days of slab construction. Testing of second phase involved the application of static load and repeated load.

To perform the static load test, a brick wall of height 4 ft and 5 inch thickness was constructed around the 120in×60in slab. Then water pump was used to fill the 120in×60in×48in chamber on the slab. Linear Voltage Displacement Transducers (LVDTs), portable data logger and computer arrangements were used for data acquisition.

Figure 7: Application of water pressure on masonry slab.

Figure 8: LVDT setup

Figure 9: Portable Data Logger

Figure 10: Data acquisition devices

To perform the repeated load test, similar instrumentation was done. In this case, the height of water was increased again deceased gradually with respect to time and the reading changes in the data acquisition devices were observed. This was repeated 10 times. This has been done by data acquisition systems corresponding to deformation such as LVDT's and strain gauges in slab have been taken. Mainly two types of loading were induced on slab panels: static load and repeated load.

2.2 Finite Element Analysis

The Finite Element Analysis (FEA) method, originally introduced by Turner et al. (1956), is a powerful computational technique for approximate solutions [1]. The ANSYS program was introduced in 1970, by Swanson Analysis System (1995). Since then ANSYS Support Distributors have grown as part of a commitment to provide latest finite element analysis and design technology to engineers, worldwide. The primary unknowns (nodal degrees of freedom) calculated in a structural analysis are displacements. Other quantities such as strains, stresses, and reaction forces, are then divided from the nodal displacements. Especially, its graphical representations are very distinct [11].

2.2.1 ANSYS Finite Element Modeling

To create the finite element model in ANSYS there are multiple tasks that have to be completed for the model to run properly. Models can be created using command prompt line input or graphical user interface (GUI). For the simulation of the model, the GUI method was utilized to create the model. The section describes the different tasks and entries into used to create the FE calibration model.

2.2.2 Element Type

An element type is identified by a name, consisting of a group label and a unique, identifying number. In this study SOLID186 element is used to model brick and SOLID65 element is used to model mortar.

2.2.3 Real Constant

The real constants for this model are shown in Table 1. Note that individual elements contain different real constants. No real constant set exists for the Solid186 element. Real Constant Set 1 is used for the Solid65 element. It requires real constants for rebar assuming a smeared model. Values can be entered for Material Number, Volume Ratio, and Orientation Angles. In the present study the beam is modeled using discrete reinforcement. Therefore, a value of zero was entered for all real constants which turned the smeared reinforcement capability of the Solid65 element off.

Table 1: Real Constants for Model

Real Constant	Element Type	Constant			
			constant 1	constant 2	constant 3
2	SOLID65	Material Number	0	0	0
		Volume Ratio	0	0	0

2.2.4 Material Properties

As stated earlier, brick used in herringbone pattern is defined with SOLlD186 element in the model. The properties of SOLlD186element [7] are listed below:

Table 2: SOLlD186 Material properties

Material Model Number	Element Type	Material Properties	
1	SOLlD186	Linear Isotropic	
		EX	5.366 x 10^6psi
		PRXY	0.2
		Density	
		0.072 lb/in^3	

Mortar is defined with SOLlD65 element in the model. The properties of SOLlD65 element are listed below:

Table 3: SOLlD65 Material properties

Material Model Number	Element Type	Material Properties	
2	SOLlD65	Linear Isotropic	
		EX	2.906 x 10^6psi
		PRXY	0.25
		Density	
		0.054 lb/in^3	
		Concrete	
		ShrCf-Op	0.3
		ShrCf-Cl	1
		UnTensSt	410
		UnCompSt	3000
		BiCompSt	0
		HydroPrs	0
		BiCompSt	0
		TenCrFac	0

2.2.5 Modeling

Modeling is the most important part in the simulation through GUI method. The masonry slab model has been designed as herringbone pattern. Each brick and mortar block in the model has been designed individually then connected to each other by GLUE command [8].

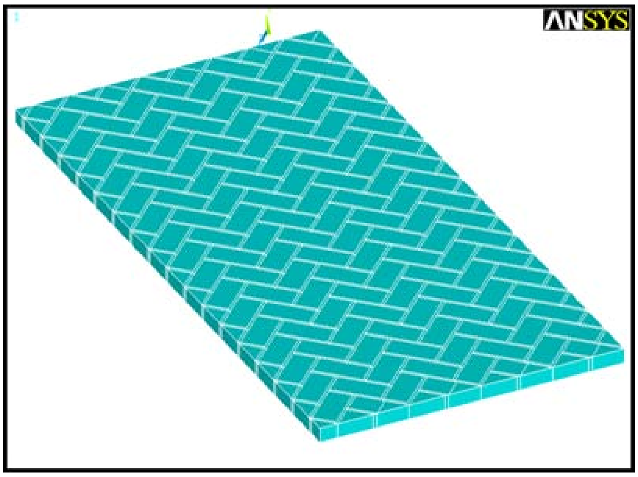

Figure 11: Finite element model of masonry slab

2.2.6 Finite Element Meshing

Meshing is the important part of this model. Size control of the mesh element is also necessary because, huge numbers of elements are prepared with many nodes, the matrix required to solve the nodal solution will be large, and its solution may fail for some circumstances. Meshing is an integral part of the computer-aided engineering (CAE) analysis process [9]. The mesh influences the accuracy, convergence and speed of the solution. Furthermore, the time it takes to create a mesh model is often a significant portion of the time it takes to get results from a CAE solution. Therefore, the better and more automated the meshing tools, the better the solution. From easy automatic meshing to highly crafted mesh, ANSYS, Inc. provides the ultimate meshing solution. ANSYS® technologies provide powerful pre- and post-processing tools for mesh generation from any geometry source, to produce almost any element type, for nearly any physics, for virtually any application.

The ANSYS meshing solution provides robust automatic tetrahedral meshing on even the most complicated geometries. With automatic contact detection and setup, a user requires little training to do sophisticated analysis. In addition, users can generate a pure hex mesh with the automatic body-fitted Cartesian meshing approach. This unique method will ignore geometric features to obtain a pure hex mesh on geometries that once were thought impossible to hex mesh. For users who want control over every step in the meshing process to create highly crafted brick meshes or to mesh directly on scan data, ANSYS provides state-of-the-art hex, tet and hybrid meshing tools to fit almost any need.

Element size is the size by which the whole volumetric area is to be meshed. It is essential to mesh the volume properly. At the time of making dimensions it is obvious to merge all the points selected from the lines and the volumetric areas. It is necessary to merge all the meshing elements properly. Otherwise no result will be found. The meshed finite element model is shown below.

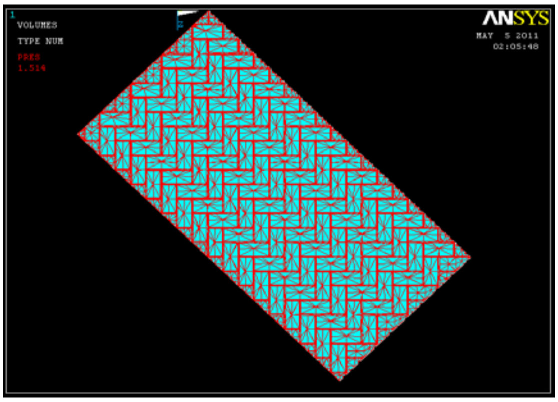

Figure 12: Meshed volume of the finite element model

2.3 Defining Loads

Various types of load have been applied on the finite element model of the masonry slab and then analyzed it by various methods [10].

2.3.1 Applying Static Load

A non-varying load; the basal pressure exerted by the weight of a mass at rest, such as the load imposed on a drill bit by the weight of the drill-stem equipment or the pressure exerted on the rocks around an underground opening by the weight of the superimposed rocks. Static loads were imposed on the structure. It was impose at the edge of the beam for both the exterior and the interior one. For keeping the full structure fixed displacement was applied on all four side of the masonry slab model as all DOF fixed. Then concentrated load such as 1000lb, 500lb etc. was applied on the slab model. In the same way uniformly distributed load of various magnitudes such as 218 psf, 500 psf etc. was applied in the slab model.

2.3.2 Applying Transient Load

Transient dynamic analysis is a technique used to determine the dynamic response of a structure under a time-varying load. The time frame for this type of analysis is such that inertia or damping effects of the structure are considered to be important. Cases where such effects play a major role are under step or impulse loading conditions, for example, where there is a sharp load change in a fraction of time. When loads are applied suddenly and when the loads are applied as impact loads the resulting stresses induced in the machine elements are much higher than if the loads are applied gradually.

Most ductile materials have strength properties which are a function of the loading speed. The more rapid the loading the higher is the tensile and ultimate strengths of the materials. Two standard tests, the Chirpy and Izod, measure the impact energy (the energy required to fracture a test piece under an impact load), also called the notch toughness. The detailed assessment of the strength of machine elements under impact loading regimes involves use of advanced techniques including Finite Element Analysis. Impact loads result in shock waves propagating through the elements with possible serious consequences. It is possible to complete relatively simply stress evaluation for suddenly applied and impact loads by using the principle of conservation of energy and assuming the materials considered respond to the loading elastically. The masonry slab model has been analyzed for ***an*** impact load of 50.706 lb and for varying time step such as .1 sec, .01 sec and .001 sec.

2.3.3 Applying Harmonic Load

Harmonic response analysis is a technique used to determine the steady-state response of a linear structure to loads that vary sinusoidally (harmonically) with time [6]. The idea is to calculate the structure's response at several frequencies and obtain a graph of some response quantity (usually displacements) versus frequency. "Peak" responses are then identified on the graph and stresses reviewed at those peak frequencies. Cyclic load is a load which is applied over and over; may cause a type of crack called a "fatigue crack." In materials science, fatigue is the progressive and localized structural damage that occurs when a material is subjected to cyclic loading. This cyclic load is one kind of harmonic load.

Before the implementation of load it is necessary to provide the density of the material. A harmonic load of 50.706 lb was applied in the finite element model. “Real part of force / moment” in this part the projected value is to be provided. By specifying a real and imaginary value of the load it provides information on magnitude and phase of the load. In this case the magnitude of the load is some value and its phase is 0. Phase information is important when two or more cyclic loads are being applied to the structure as these loads could be in or out of phase. For harmonic analysis, all loads applied to a structure must have the same frequency.

3. RESULTS AND DISCUSSION

This section focuses on the experimental results obtained from each test and analysis of the test results. ANSYS 13.0 gives many facilities to find out the desired results at any point of interest in the model. Corresponding sub-step of the result from result-list is to be read by pick. Some graphical representations of the results are shown in this section such as deformed shape, displacements, stress, strain etc. along with laboratory test results.

3.1 Experimental Results

3.1.1 Static loading on slab panel

From the laboratory test no significant change in deformation was recorded from the data acquisition devices. However the slab carried a water column height of 4 ft on the area of 10 ft×5 ft which equivalent to 218 psf. Hence the slab carried a uniform distributed load 4 times than traditional load of residential buildings. The maximum y component displacement found for this magnitude of load is 6.13×10^{-3} inch, which is very low. Moreover no crack and leakage of slab panel was observed.

3.1.2 Repeated loading on slab panel

Slab carried repeated load 10 times while varying the height of water pressure. However no change in deformation and no cracks were found when repeated load was induced on slab panel.

3.2 Finite Element Analysis Results

3.2.1 Results for Uniformly Distributed Load of 218 psf

For static analysis of the masonry slab for 218 psf uniformly distributed loading a maximum y component displacement of 5.1×10^{-3} inch was found and no crack was found for this magnitude of load. This value of displacement varies with the experimental result (6.13×10^{-3} inch) by 16.8%.

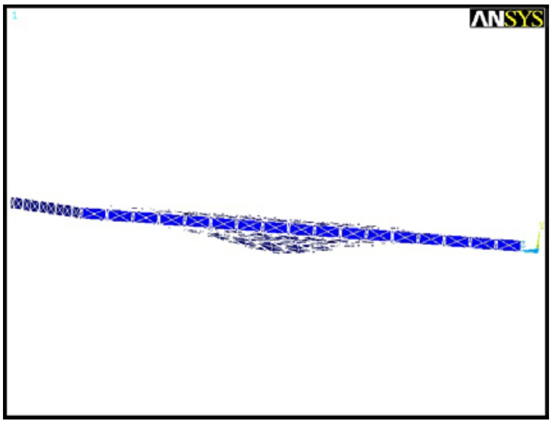

Figure 13: Deformed shape of masonry slab model for UDL magnitude of 218 psf

Figure 14 shows the contour output of degree of freedom solutions for y component displacement. The color bar shown at bottoms of each plot indicates the ranges of desired values.

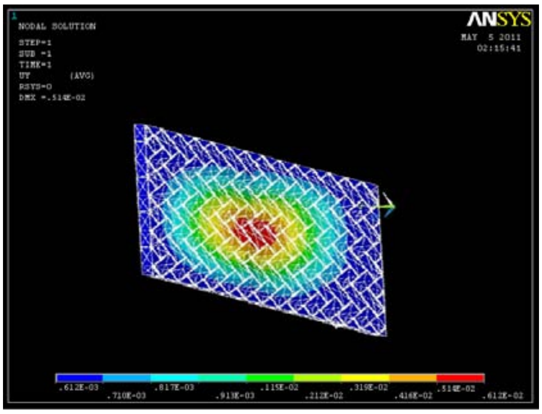

Figure 14: Contour output of DOF solutions for y component displacement

3.2.2 Results for Uniformly Distributed Load of 500 psf

For static analysis of the masonry slab for 500 psf uniformly distributed loading a maximum y component displacement of 7.78×10^{-3} inch was found and no crack was found for this magnitude of load.

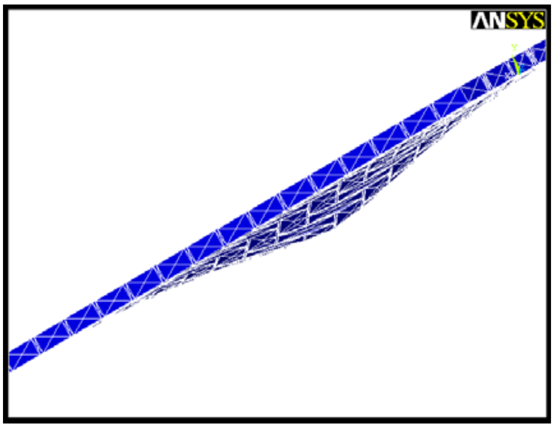

Figure 15: Deformed shape of masonry slab model for UDL magnitude of 500 psf

Figure 16 shows the contour output of degree of freedom solutions for y component displacement. The color bar shown at bottoms of each plot indicates the ranges of desired values.

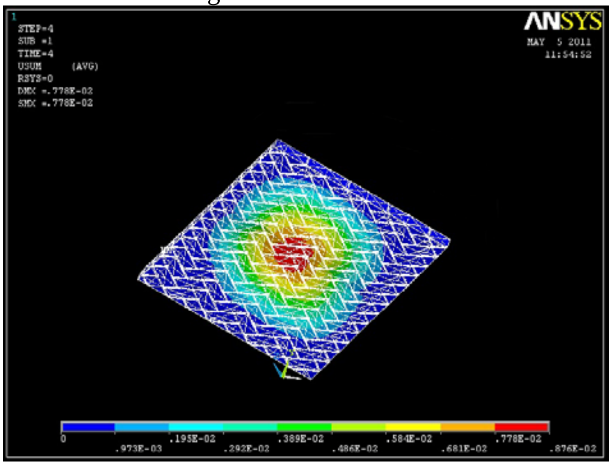

Figure 16: Contour output of DOF solutions for y component displacement

3.2.3 Results For Impact Loading

Impact analysis was done on the finite element model of masonry slab for an impact load of 50.706 lb. Impact analysis was done for three different time sub steps such as 0.1 sec, 0.01 sec and 0.001 sec. Impact analysis of the masonry slab model shows a maximum displacement of 1.33×10^{-3} inch when using a time sub step size of 0.1 sec. The analyses show a displacement of 4.1×10^{-5} inch and 6.1×10^{-5} inch when using time sub step size of 0.01 sec and 0.001 sec respectively. The graphical representations of the impact analysis results are shown here:

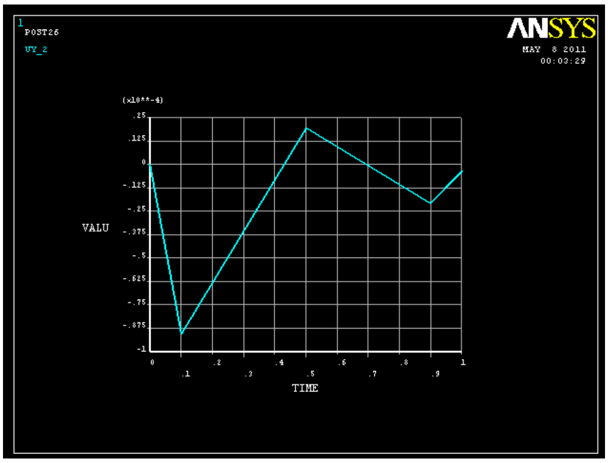

Figure 17: Displacement vs time plot of impact analysis for time sub step size of 0.1 sec

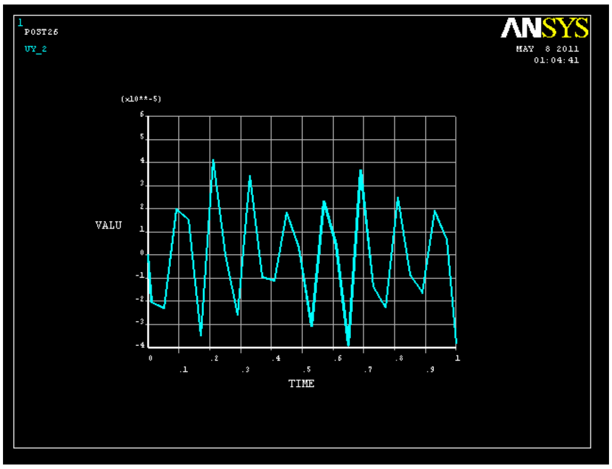

Figure 18: Displacement vs time plot of impact analysis for time sub step size of 0.01 sec

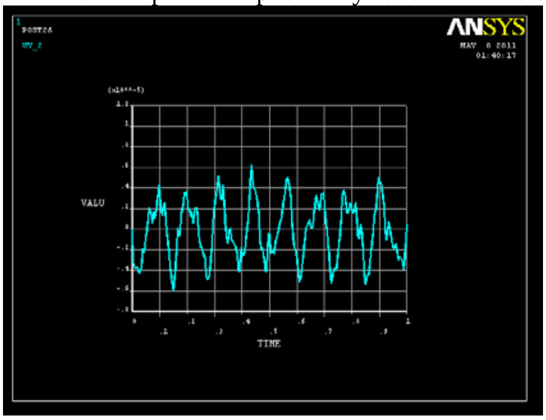

Figure 19: Displacement vs time plot of impact analysis for time sub step size of 0.001 sec

This case study shows a research on the impact load caused the masonry slab through dynamic analysis developed with programming methods based on the finite element method. Through this analysis the maximum displacement that is produced by the impact on the slab has been determined.

3.2.4 Results For Harmonic Loading

Harmonic analysis was done on the finite element model of masonry slab for a cyclic load of 50.706 lb. Frequency step set for this analysis from 1 Hz to 100 Hz. 100 sub steps were selected for this analysis. Harmonic analysis of the masonry slab model shows a maximum displacement of 2.82×10^{-3} inch when using a frequency sub step size from 1Hz to 100 Hz. The graphical representation of the harmonic analysis results are shown here:

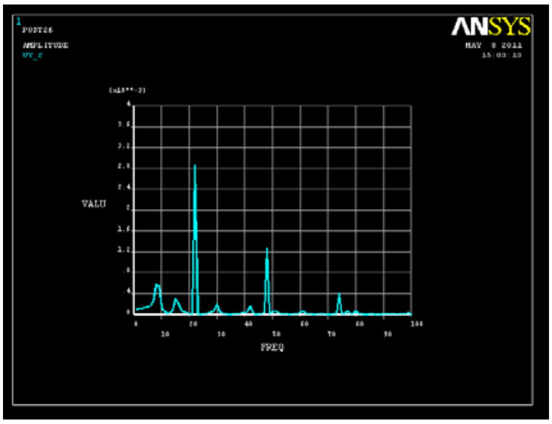

Figure 20: Displacement vs frequency plot of harmonic analysis

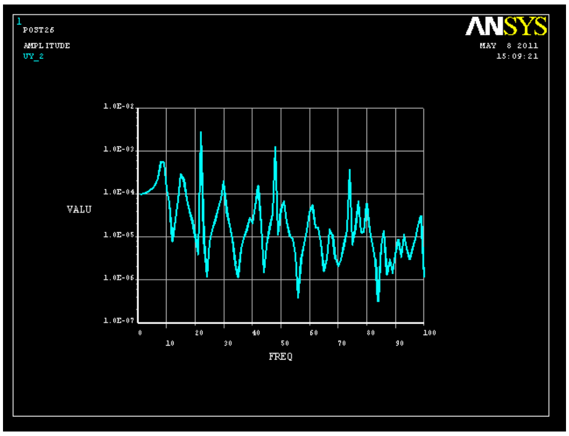

Figure 21: Semi log plot for Displacement vs frequency of harmonic analysis

For determining the response of a structure to harmonically time-varying loads, such as those typically seen in rotating machinery, harmonic analysis is done. ANSYS harmonic analysis is employed to predict the sustained dynamic behaviour of structures to consistent cyclic loading.

4. CONCLUSIONS

The aim of this study was to investigate the mechanical behaviour of herringbone pattern masonry slab model. From results it has been concluded that herring bone bond masonry slab can resist maximum load, moment and stress when constructed without reinforcement. A maximum y component displacement of 6.13×10^{-3} inch was found by LVDT and Data logger when tested a 10ft by 5ft herring bone bond masonry slab under uniformly distributed load of 218 psf. A 10ft by 5ft finite element model of masonry slab without reinforcement has been analysed for the same magnitude of uniformly distributed load and a maximum y component displacement of 5.1×10^{-3} inch was found. Hence the experimental and finite element analysis result show that the slab could carry uniform distributed load of 4 times than conventional residential building. Hence negligible amount of deformation was observed. No sign of crack was found. For static analysis of the masonry slab for 500lb and 1000lb concentrated loading a maximum y component displacement of 1.05×10^{-3} inch and 2.09×10^{-3} inch were found respectively and no crack was found for these magnitude of load. Satisfactory results were also found for transient and harmonic analysis. So herringbone pattern masonry slab can be easily used in residential building construction.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

FINITE ELEMENT ANALYSIS FOR THE ASSESMENT OF MASONRY BUCKLING BEHAVIOUR

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ABSTRACT

Load bearing masonry is among the most ancient construction technologies, yet continues to provide boundless opportunities for both traditional and modern design. Masonry load bearing wall subjected to vertical concentric and eccentric loading may collapse through instability. In this Paper the buckling behavior of masonry load bearing wall of different slenderness ratio were investigated via testing a series of scale masonry wall subjected to concentric and eccentric vertical loading. A total of thirty six masonry walls were tested in the Laboratory of Technical University of Catalonia (UPC), which was the basis of this numerical study. For better understanding of buckling failure of the masonry load bearing wall and to simulate the response of walls tested in laboratory, a numerical finite element model was developed based on the simplified micro model approach. The numerical model was calibrated by using those results found from experimental study. The numerical model was developed, which takes into account both geometrical and material nonlinearities. The influence of tensile strength of units, nonlinear behavior of interface element, slenderness ratio and various end conditions have been investigated together with the effect of different end eccentricity of vertical load.

Keywords: *Masonry load bearing wall, buckling failure, eccentric load, slenderness ratio, micro-modeling*

1. INTRODUCTION

The micro-modelling strategy, is considered at present as one of the most accurate tools available to model the behaviour of masonry structures, and has been adopted in the present research in order to carry out the needed numerical simulations. Micro-modelling allows, in particular, an appropriate simulation of the buckling response taking into account joint tensile cracking in combination with masonry crushing in compression. In this study, the predictions of the ultimate capacity of walls obtained by means of micro-modelling approach and compared with experimental results obtained from the experimental study in structural engineering laboratory of Technical University of Catalonia (UPC). Moreover, results obtained in the parametric studies by considering different end support condition and effect of tensile stress on failure loads of walls were investigated. Conclusions are drawn on the relative importance of tensile strength, non-linear geometrical and material properties with different end eccentricity.

2. ADOPTED MODELING STRATEGY

The numerical simulation presented is performed with the well-known micro-model proposed by Lourenco & Rots (1997) requires more specific software oriented to masonry analysis. For all cases, micro-models assume 2D plain-stress and a hinged-hinged configuration. The hinges are modeled by means of stiff triangular objects placed at the bottom and at top of the wall, whose end vertex is allowed to freely rotate. In addition, a minimum eccentricity of 1mm is always applied in order to account for possible irregularities of the wall geometry of the load positioning. Basically, the model assigns an elastic behavior to the units whereas masonry inelastic behavior is transferred to the joints. This analysis was performed with DIANA software. The integration schemes used are 2x2 points Gauss integration for the continuum elements and 3 points Lobato integration for the interface elements. An interface allows discontinuities in the displacement field and its behavior is described in terms of a relation between the traction t and relative displacement u across the interface. In the multisurface interface model for the masonry proposed by Lourenco and Rots, the quantity of traction and displacement is denoted as generalized stress σ and generalized strain, ε. In this case the elastic constitutive relation between stresses and strain is given by:

$$\sigma = D\varepsilon \tag{1}$$

For 2D configuration $D = diag\{k_n, k_s\}, \sigma = (\sigma, \tau)^\tau and \varepsilon = (u_n, u_s)$

Where, n and s is the normal and shear components respectively. The terms in the elastic stiffness matrix can be obtained from the properties of both masonry components and thickness of the joint as:

$$k_n = \frac{E_u E_m}{t_m(E_u - E_m)}; k_s = \frac{G_u G_m}{t_m(G_u - G_m)} \tag{2}$$

Where, E_u and E_m = Young's moduli; G_u and G_m = Shear moduli and t_m = thickness of the joint.

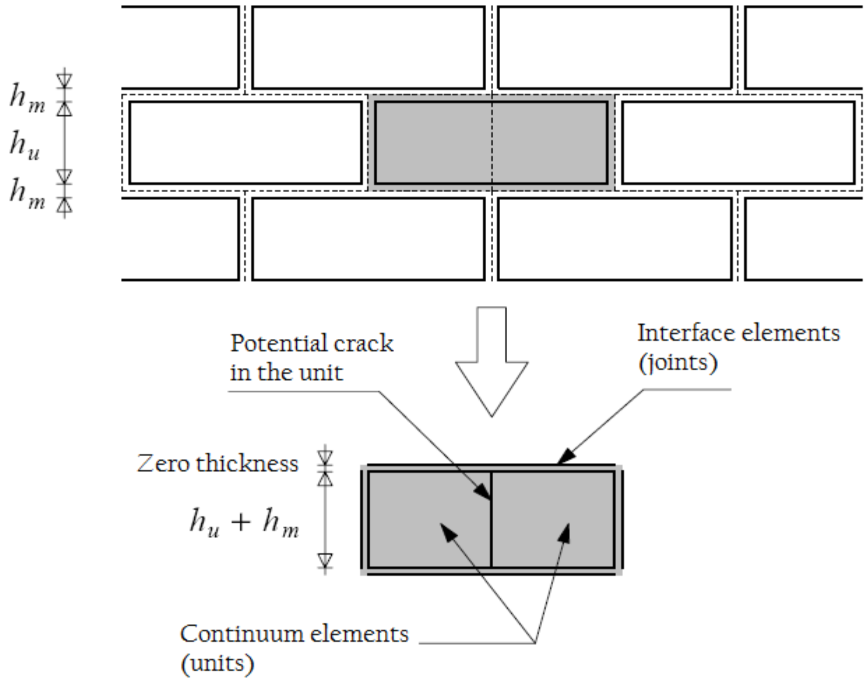


Figure 1: Proposed modeling strategy. Units (u), which are expanded in both directions by the mortar thickness, are modeled with continuum elements. Mortar joints (m) and potential cracks in the units are modeled with zero-thickness interface elements.

The interface model includes a compressive cap where the complete inelastic behavior of masonry in compression is lumped. This is a phenomenological representation of masonry crushing because the failure process in compression explained by the microstructure of units and mortar and the interaction between them. In the model the failure mechanism represented in such way that the global stress strain diagram is captured.
The model was justified by Lobato et al. and found that the model is efficiently able to reproduce the experimental results. For this reason, the proposed micro-model was selected by the author to simulate the wall for buckling failure.

3. MODEL DESCRIPTION

In the numerical simulation, the units were modelled by using plain-stress continuum 8-node elements and for the mortar joints adopted 6-node zero-thickness line interface elements. In addition, hinges are modelled by means of stiff triangular objects. Each unit was modelled with 12 x 3 elements. The geometry and meshing of the wall for slenderness ratio 6 and eccentricity 0, t/6 and t/3 are shown in the Figure 2. For all cases, micro-models of wall considered hinged-hinged configuration. The hinges are modelled by means of stiff triangular objects placed at the bottom and at top of the wall, whose end vertex is allowed to freely rotate. The vertical load was applied concentrically and eccentrically as unit deformation. The boundary condition and loading configuration is shown in the Figure 2.

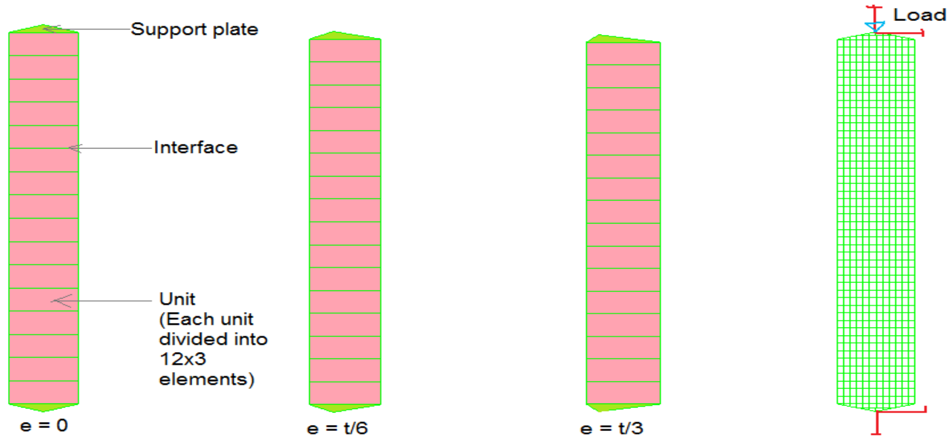


Figure 2: Geometry, Meshing and Load of walls for slenderness ratio 6

3.1 Material Properties

The material parameters used for the numerical simulation are shown in the Table 1. Some parameters such as $G^I{}_f$ and C_S have been taken directly from the previous research The fracture energy for mode I, $G^I{}_f$ have been taken from the test carried out by Van der Pluijm (1992) and for the parameter of shape of elliptical cap C_S a value of 9 has been adopted from Lourenco (1996). The interface elastic stiffness values were calculated from thickness of the joint h_j, the Young's moduli of unit and joint E_u and E_j , respectively, and the shear moduli of unit and joint G_u and G_j , respectively as CUR (1994):

$$k_n = \frac{E_u E_j}{h_j (E_u - E_j)}; k_t = \frac{G_u G_j}{h_j (G_u - G_j)} \tag{3}$$

The different strength values f_f, c and f_m have been obtained from the experimental study carried out in UPC (2009). The compressive fracture energy G_{fc} and equivalent relative displacement k_p calculated according to Model Code 90 and Eurocode 6, respectively by using followings formula (Lourenco, 1996):

$$G_{fc} = 15 + 0.43 f_m - 0.0036 f_m{}^2; k_p = \{0.002 - f_m (\frac{1}{E_u} + \frac{1}{k_n (h_u + h_j)})\} \tag{4}$$

Table 1: Material parameters adopted for numerical analysis.

Components	Parameter	Symbol	Units	Values
Brick	Elastic modulus	E_b	N/mm^2	4800
	Poison ratio	v	-	0.15
	Tensile strength	f_{tb}	N/mm^2	3.95
Joint	Normal stiffness	k_n	N/mm^2	2800
	Shear stiffness	k_t	N/mm^2	1900
	Bond tensile strength	f_t	N/mm^2	0.554
	Mode – I fracture energy	$G^I{}_f$	Nmm/mm^2	0.02
	Cohesion	c	-	0.45
	Mode – II fracture energy	$G^{II}{}_f$	Nmm/mm^2	0.175
	Angle of internal friction	$tan\varphi$	-	0.812
	Angle of dilatancy	$tan\Psi$	-	0.009
	Compressive strength of masonry	f_m	N/mm^2	14.20
	Compressive fracture energy	G_{fc}	Nmm/mm^2	20.38

3.2 Validation of Model

The micro-models were validated next by a comparison with experimental results obtained from UPC (2010). Usually, experiments on load bearing walls have been adopted by the masonry community as the most common axial load test and the tensile capacity of masonry has been neglected. As a result, the clear understanding of the buckling characteristics of masonry load bearing walls under concentric and eccentric vertical load was absent. In this study, special attention is given to the load bearing wall tests carried out in the UPC (2010), because most of the parameters necessary to characterize the material model are available from micro-experiments. The main concern of this work was, to demonstrate the ability of the model to capture the behaviour observed in the experiments and close quantitative reproduction of the experimental results. The large number and variability of the material parameters necessary to characterize the developed model permits to adopt a set of parameters suitable to closely fit the experimental capacity slenderness ratio diagrams. For the numerical analyses, units are represented by plane stress continuum elements (8-noded) while line interface elements (6-noded) are adopted for the joints. Each unit is modelled with 12 × 3 elements. For the joints, the composite interface model described in this study is adopted.

3.3 Results of Numerical Simulation

The comparison between the experimental collapse load and collapse load obtained from the micro- model is presented in the Figure 52, 53 and 54.

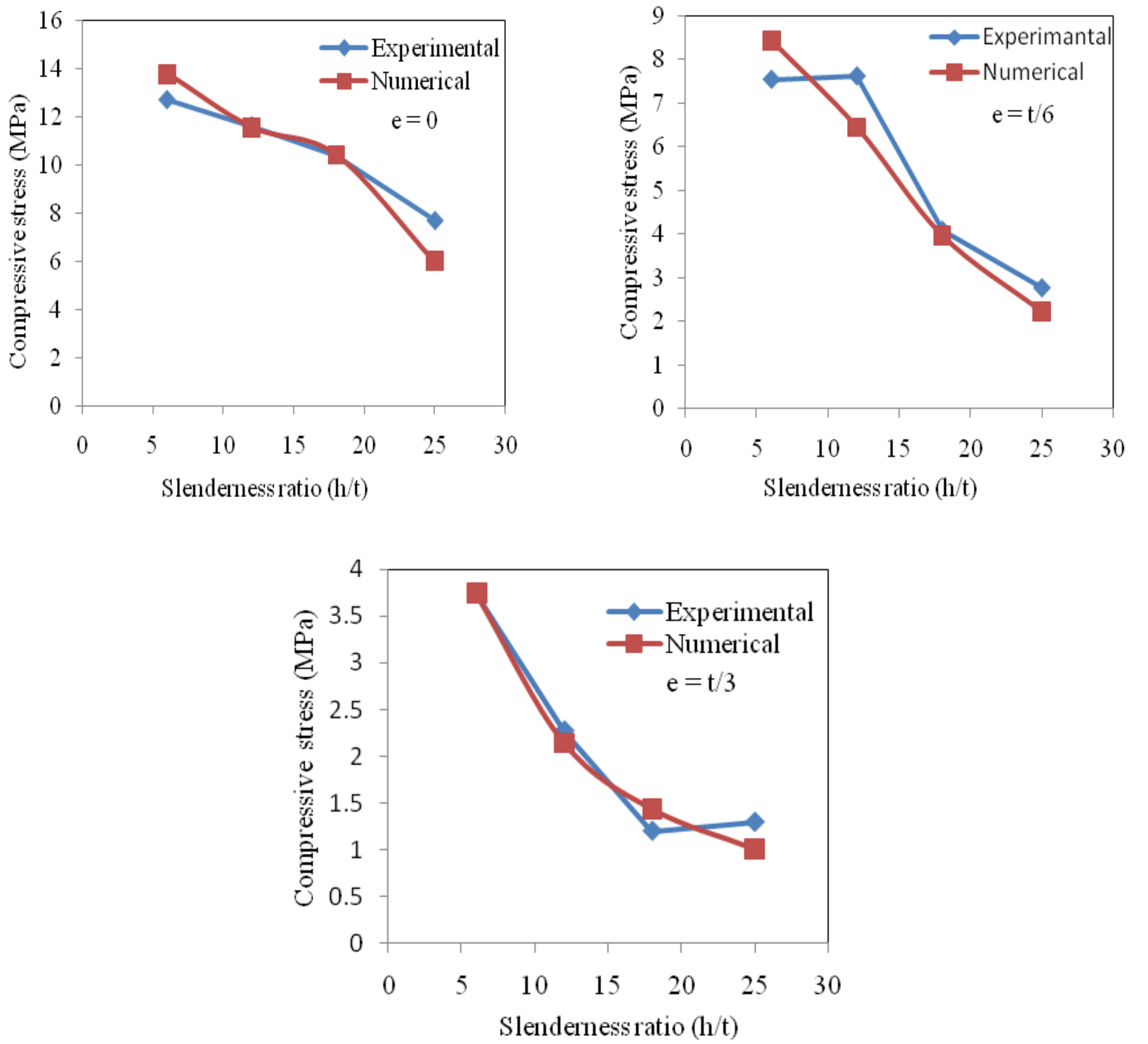


Figure 3: Comparison of compressive stress for different load eccentricity

The above figure shows that the experimental behavior is satisfactorily reproduced and the collapse load estimated within a 15 % range of the experimental values. The micro-modeling approach is being able to provide a very satisfactory estimation of the experimental capacity of the walls particularly, for the case with e = 0. The average errors are 7.85%, 12.6% and 11.93% for the eccentricity of 0, t/6 and t/3 respectively. For all cases, one tendency is clear that with the increasing of slenderness ratio and application of load eccentricity the capacity of the wall decreased.

4. PARAMETRIC ANALYSYS

For this parametric study vertical load is applied with different eccentricity and different boundary conditions such as hinge-hinge, hinge-fixed and fixed-fixed configurations. Four variables are investigated in the parametric study, namely, wall slenderness, loading conditions, boundary conditions and effect of tensile strength.

4.1 Effect of Boundary Condition

In this study an investigation was carried out to better understanding the effect of boundary conditions on the strength and buckling behavior of masonry load bearing walls by using hinge-hinge, hinge-fixed and fixed-fixed configurations. The results of collapse load for different end conditions are shown in the Figure 4.

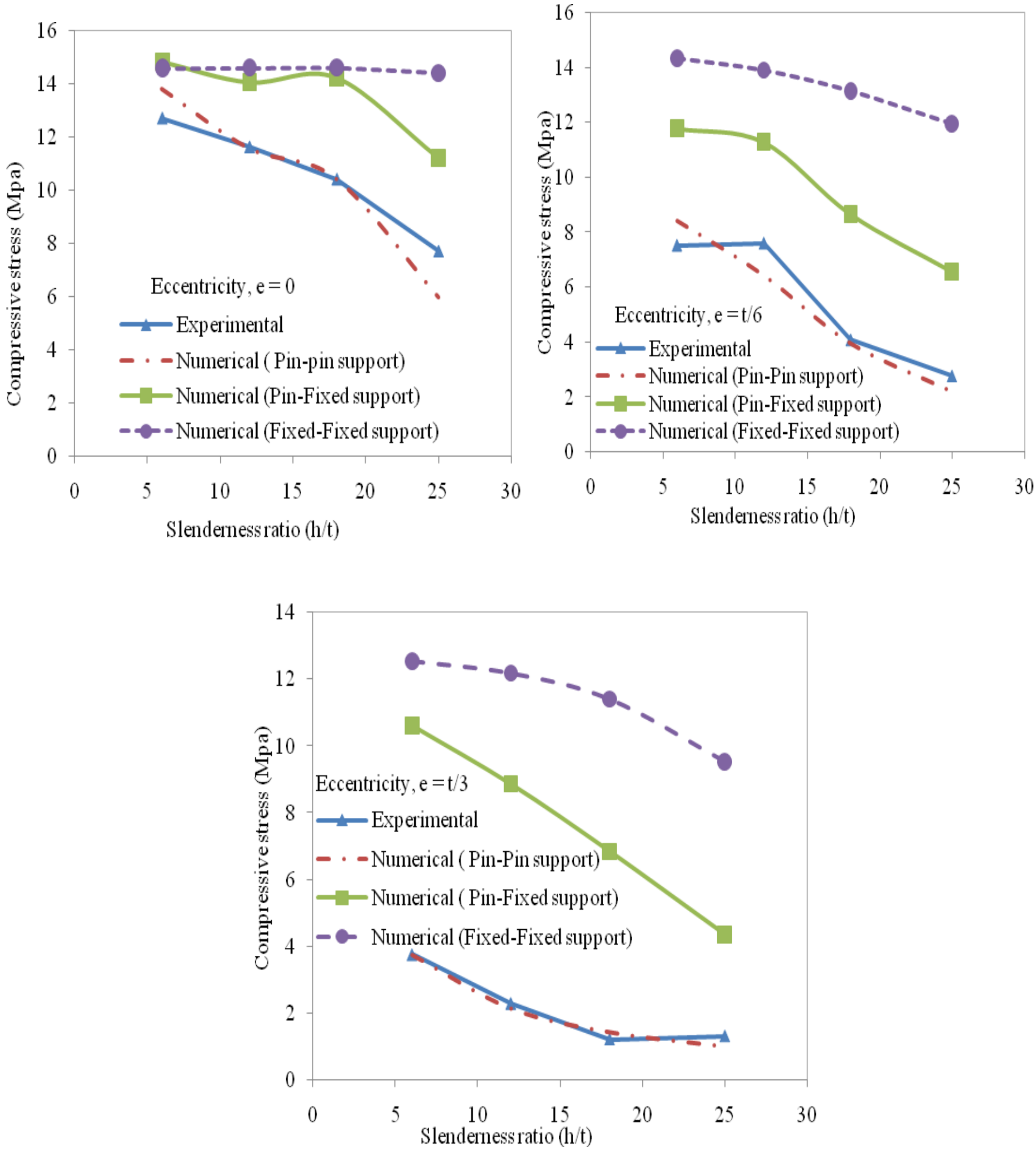


Figure 4: Capacity of wall for different end conditions with e = 0, t/6 and t/3.

The above figure shows that due to the change of end condition from hinge-hinge to fixed-fixed, the ultimate capacity of wall increases significantly. This increment has small value for lower slenderness ratio and rate of increment increased with the increasing of slenderness ratio. In the case of eccentricity,e = 0 and slenderness ratio 25, when the support changes from hinge-hinge to hinge-fixed and fixed-fixed, the ultimate load capacity of wall increase more than two times and very close to three times respectively and for slenderness ratio 18 capacity increase around 1.5 times of hinge-hinge support. The capacity of the wall increases significantly for both cases of hinge-fixed and fixed-fixed support for eccentricity = t/6, however, the higher increment obtained for fixed-fixed support. Also found the similar tendency of higher increment rate for higher slenderness ratio. The ultimate capacity for slenderness ratio 25 increased three times and five times for hinge-fixed and fixed-fixed supports respectively and about two times and around four times respectively when consider slenderness ratio 18. For the other slenderness ratio the capacity increases within the ranges between 1.2 to 2 times of hinge-hinge support. For the case of eccentricity t/3, the increment of capacity is higher than the other two eccentricities. The ultimate load increased around ten times for fixed-fixed support in cases of both slenderness ratio 25 and 18 respectively, while for hinge-fixed support about four times in cases of both slenderness ratio 25 and 18 respectively. On the other hand, the collapse load increased 4 to 6 times for slenderness ratio 6 and 12 with hinge-fixed and fixed-fixed end conditions respectively when compared with hing-hinge support.

4.2 Effect of Tensile Stress

For better understanding the effect of tensile strength of masonry on the collapse load and buckling behavior, the parametric analysis was carried out by using tensile strength, f_t of 0.001, 0.284, 0.568, 0.852, 1.136 and 1.42 MPa which was 1%, 2%, 4%, 6%, 8% and 10% of masonry prism compressive strength respectively. The analysis was performed for the boundary conditions of hinge-hinge, hinge-fixed and fixed-fixed. The results obtained from this analysis are shown in the following Figures below.

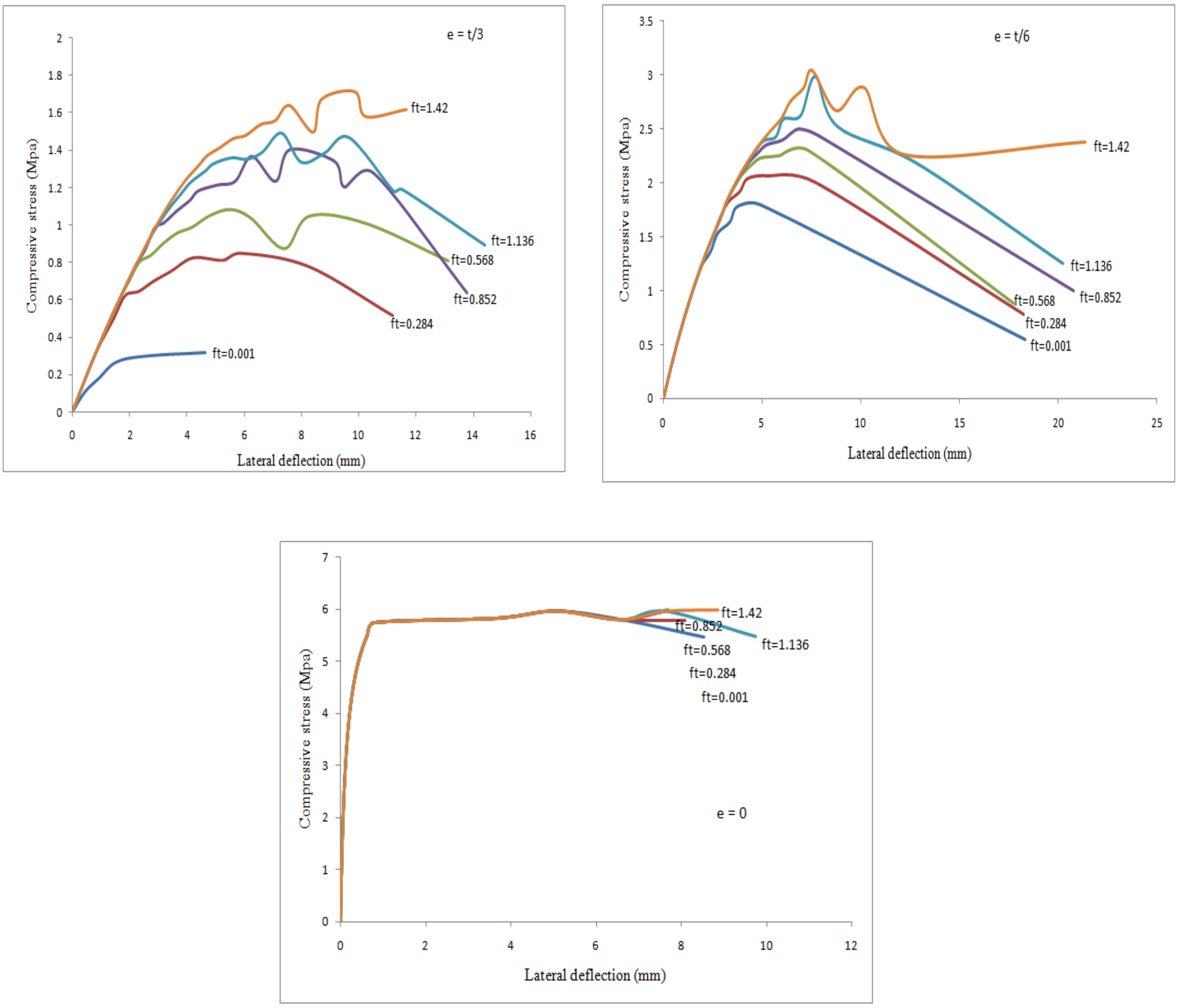
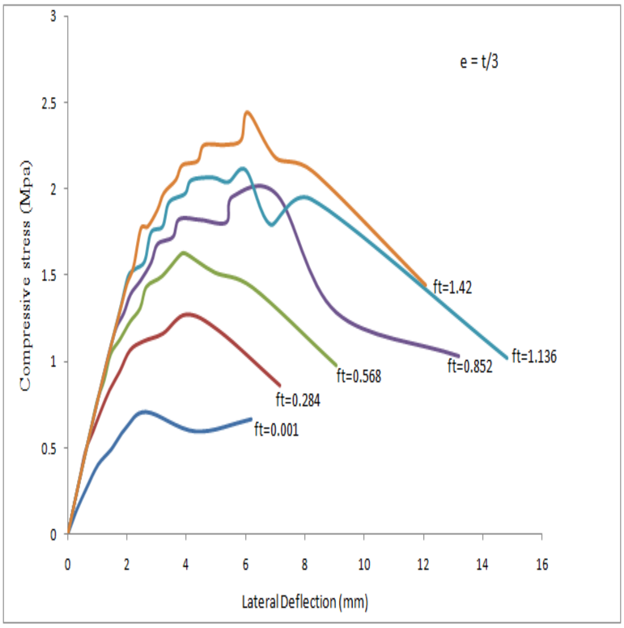


Figure 5: Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 25.

The Figure 5 shows that if the tensile strength varies from 0.001 to 1.42 MPa, the compressive stress of masonry increased from 0.31 to 1.49 MPa, 1.79 to 2.98 MPa and 5.96 to 5.98 MPa for the eccentricity t/3, t/6 and 0, respectively. The influence of tensile strength on ultimate capacity of masonry wall is very high in the case of higher load eccentricity and the influence decreases with the decreasing of load eccentricity.



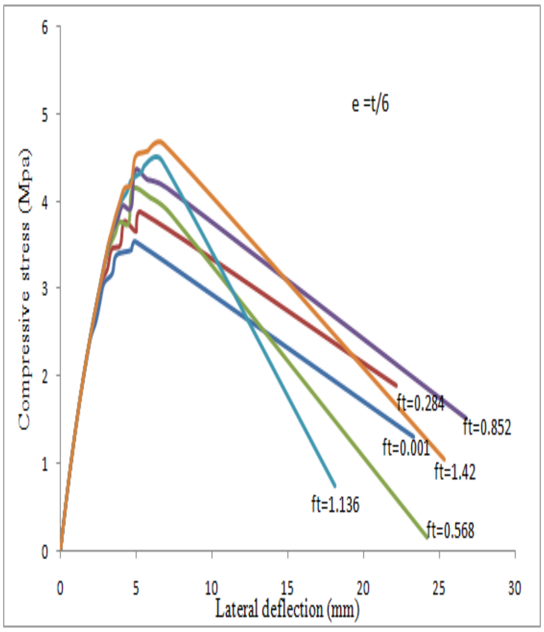
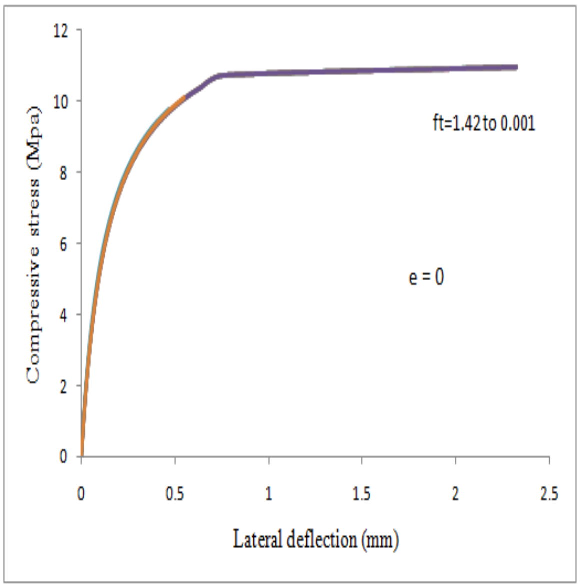


Figure 6: Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 18.

In case hinge-hinge support, Figure 6 and 7 (left) shows that the tensile strength has low effect on compressive strength for low eccentricity. The ultimate load increased 0.93 MPa and 1.40 MPa when the tensile strength of masonry varies from 1% to 10% of masonry prism compressive strength for eccentricity of load t/6 and t/3, respectively while the no effect of tensile strength on capacity in the case of eccentricity equal to zero.



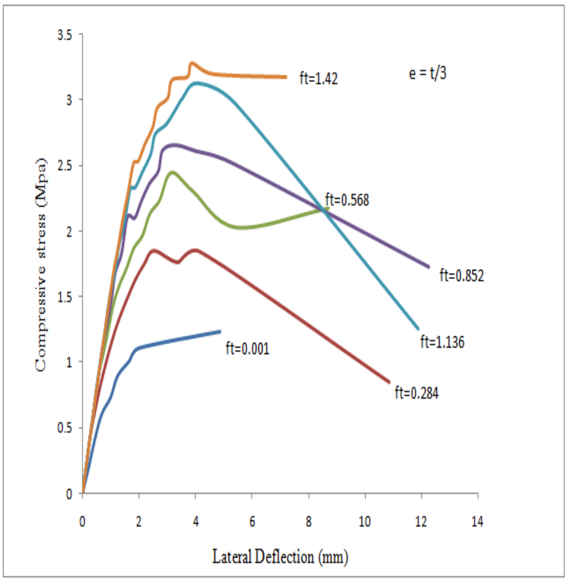


Figure 7: Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 18 (left) and slenderness ratio 12 (right).

Figure 7 (right) shows that when tensile strength changes from 0.001 to 1.42 MPa the failure load changes from 1.23 to 3.28 MPa with irregular variation of lateral deflection.

5. CONCLUSIONS

A set of experimental tests on the buckling failure of masonry walls has been numerically simulated by means of simplified micro-modeling approach. The micro-model described the nonlinear response of masonry in compression in an indirect way by localizing it to the units. In all cases, the non-linear response in tension is localized to the joints. The simplified micro-models afford a satisfactory prediction of the ultimate load of walls taking into account the buckling behavior. Simulations carried out by the micro-model provide the best fits for the test results with an acceptable error. It must be noted that some difference with respect to the experimental results is unavoidable because of the influence of possible non-reported accidental eccentricities. In the case of fixed support, the load capacity increased 2 to 6 times higher than hinge support depending on slenderness ratio and eccentricity. The capacity of wall for hinge-fixed support lies between the both end hinge and both end fixed support. In the case of hinge-hinge support with high eccentricity, the influence of tensile strength is higher than the other support conditions. Most of the cases, negligible effect was found for null eccentricity. The influence of tensile strength follow a common tendency from higher to lower values when the support condition and load eccentricity moves from hinge to fixed and higher to lower eccentricity respectively.

ACKNOWLEDGEMENTS

In completing this research, I would like to express special thanks to Professor Paulo Lourenco, Professor Enrico Garbin and PhD student Cristian Sandoval, for nice cooperation, frequent attention and encouragement during whole research period. A financial support from European Commission through Erasmus Mundus Scholarship is gratefully acknowledged.

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DEVELOPMENT AND EVALUATION OF AN OPEN SOURCE FINITE ELEMENT ANALYSIS FRAMEWORK

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ABSTRACT

The importance of engineering simulation is increasing day by day with the increase of computing power and the finite element analysis method is one of the widely used approaches for this purpose. To achieve optimum simulation, there is no alternative to take complete control over the code which proprietary commercial codes fail to offer. This paper focues on the development of a finite elemnt analysis framework using freely available python libraries and wraping legacy C libraries around python; and its evaluation as a viable finite element solution.

Keywords: *Finite Element Analysis, Numerical Modeling, Engineering Simulation, Scientific Computing, Python.*

1. INTRODUCTION

The finite element analysis is a computer-based numerical technique that is widely used to solve engineering problems. Major applications for FEA include static, dynamic and thermal characterizations of mechanical phenomenon occuring in nature. Advances in computer hardware have made FEA easier and very efficient into solving complex engineering problems on desktop computers (Abaqus/CAE, 2011). In spite of the great power of FEA, the disadvantages of computer solutions must be kept in mind when using this method. They do not necessarily reveal how the stresses are influenced by important problem variables such as materials properties and geometrical features, and errors in input data can produce wildly incorrect results that may be overlooked by the analyst. Perhaps the most important function of theoretical modeling is that of sharpening the user's intuition, users of finite element codes should plan their strategy toward this end by supplementing the computer simulation with as much closed-form and experimental analysis as possible (Roylance, 2001). Moreover, To avoid input errors and increase the simulation reliability, the user must have the access to the source code level.

2. PRACTICES IN ENGINEERING SIMULATION

Basically, there are three possible approaches by which finite element analysis is performed for an engineering phenomena.

Firstly, one can use a propritery finite element analysis software package, such as ABAQUS (Abaqus/CAE, 2011), ANSYS (ANSYS - Simulation Driven Product Development, 2011) or even general purpose prototyping framework MATLAB (MATLAB - The Language Of Technical Computing, 2011). The main advantage of these FEA software packages is that they come with both pre and post-prossesor equiped. The operation of a specific software is usually detailed in the documentation accompanying the software, and vendors of the more expensive codes will often offer workshops or training sessions as well to help users learn the intricacies of code operation. One problem users may have even after this training is that the code tends to be a "black box" whose inner workings are not understood by the end user. Thus, although ABAQUS, ANSYS are good for finite element analysis, they are not suitable for full-blown research projects, where complete control is the top most priority. Furthermore, they have fixed keywords, are not open source, very expensive and not wildly accepted in academia.

Secondly, one could develop their own FEA program, in a high-level language. However, one should realize that, Most of the high level language is scripting type, so the concern for speed is great if the whole program is coded in that language alone. FEA involves solution of large matrices which require high computing power as well as efficient algorithm. In many cases, pre-written library routines offer solutions to numerical problems which are pretty hard to improve upon.

Thirdly, coding can be done in a high-level language like python (Python Programming Language – Official Website, 2011), but using calls to pre-written, pre-compiled routines in commonly available subroutine libraries, such as UMFPACK (UMFPACK: unsymmetric multifrontal sparse LU factorization package, 2011), LAPACK (LAPACK - Linear Algebra PACKage, 2011) and BLAS (BLAS, 2011), to perform all of the real numerical

work. This approach is most beneficial for the coding easiness and the performance point of view. This approach is used by the majority of researchers & scientists in their modern FEA simulation program.
So, it is evident that the use of open source FEA software is beneficial for both the user and research, and it will further enhance the simulation reliability if it is a self coded one.

3. OPEN SOURCE FINITE ELEMENT ANALYSIS FRAMEWORK

To overcome all the difficulties and errors associated with commercial FEA packages, an open source FEA framework named "simple finite element in python" or "Sfepy" has been developed. SfePy is a finite element analysis software written almost entirely in Python (Millman, 2011), with exception of the most time demanding routines, those are written in C and wrapped by Cython or written directly in Cython (SfePy, 2011). SfePy is a software for solving systems of coupled partial differential equations by the finite element method in 2D and 3D. Solution of linear, nonlinear problems are possible in sfepy. It can be viewed both as black-box PDE solver, and as a Python package which can be used for building custom applications. As a black box PDE solver, After preparing a problem description file in Python Sfepy will solve it where no real programming is involved.
The word "simple" means that complex FEA problems can be coded very easily and rapidly. Sfepy is multi-platform, it runs in Linux, Mac OS X, Windows. The collection of modules sfepy carries are FE engine, problem description facilities, interfaces to various solvers, post-processing utilities.
For Installation Sfepy prerequisites are recent NumPy (van der Walt, 2011), SciPy with Umfpack wrapper, Cython (SciPy, 2011) (Cython: C-Extensions for Python, 2011).
Sfepy depends on matplotlib, pyparsing, Umfpack, pytables for smooth running. Some tests and functions use sympy. For post-processing, mayavi2 is required (Ramachandran, 2011). All these libraries are available as single package from Pythonxy (pythonxy - Scientific-oriented Python Distribution based on Qt and Spyder - Google Project Hosting, 2011) and Enthought python distribution (Enthought Python Distribution :: Products :: Enthought, Inc., 2011).
The typical working mechanism of Sfepy is described below.

1. A top-level script, usually *simple.py* reads in an input file. Following the contents of the input file, a *ProblemDefinition* instance is created; this is the input file coming to life. The problem sets up its domain, regions, fields, the equations and the solvers. The equations determine the materials and variables in use, only those are fully instantiated, so the input file can safely contain definitions of items that are not used actually.
2. Prior to solution, *problem.time_update()* function has to be called to setup boundary conditions, material parameters and other potentially time-dependent data. This holds also for stationary problems with a single “time step”.
3. The solution is then obtained by calling *problem.solve()* function.
4. Finally, the solution can be stored using *problem.save_state()*
5. Step 2, 3 & 4 are essentially repeated for each time step. So using the code a black-box PDE solver, shields the user from having to create the *ProblemDefinition* instance by hand.

3.1 Open Source vs. Closed Source

Optimum simulation depends on complete control over the finite element analysis framework but commercial products failed do so as they are proprietary, closed source. So a user is unable to see what is going on behind the scene during the simulation. Developing a finite element analysis framework, or at least using an open source finite element analysis software will help the end user by achieving complete control over the simulation process. Open source software is community driven, so large number of users can contribute to the continuing development of the software. And the uses of freely available modules ensure least cost in both usability and maintenance.

3.2 Development

The finite element method is comprised of three major phases: (1) Pre-processing, in which the analyst develops a finite element mesh to divide the subject geometry into subdomains for mathematical analysis, and applies material properties and boundary conditions, (2) Solution, during which the program derives the governing matrix equations from the model and solves for the primary quantities, and (3) Post-processing, in which the analyst checks the validity of the solution, examines the values of primary quantities such as displacements and stresses, and derives and examines additional quantities such as specialized stresses and error indicators.

3.2.1 Pre-processing

The goals of pre-processing are to develop an appropriate finite element mesh, assign suitable material properties, and apply boundary conditions in the form of restraints and loads. The finite element mesh subdivides the geometry into elements, upon which are found nodes. The model's degrees of freedom are assigned at the nodes. The assignment of nodal dof also depends on the class of analysis. For a thermal analysis, only one temperature dof exists at each node. Developing the mesh is usually the most time-consuming task in FEA. In the past, node locations were keyed in manually to approximate the geometry. The more modern approach is to develop the mesh directly on the CAD geometry, which will be (1) wireframe, with points and curves representing edges, (2) surfaced, with surfaces defining boundaries, or (3)solid, defining where the material is. Solid geometry is preferred, but often a surfacing package can create a complex blend that a solids package will not handle. As far as geometric detail, an underlying rule of FEA is to "model what is there", and yet simplifying assumptions simply must be applied to avoid huge models. A badly distorted element will cause a matrix singularity, killing the solution. A less distorted element may solve, but can deliver very poor answers. Acceptable levels of distortion are dependent upon the solver being used. Material properties required vary with the type of solution. A linear statics analysis, will require an elastic modulus, Poisson's ratio and perhaps a density for each material. Thermal properties are required for a thermal analysis. Examples of restraints are declaring a nodal translation or temperature. Loads include forces, pressures and heat flux.

Gmsh provides a reliable pre-processing solution and is the pre-processor of the developed FEA framework (Gmsh, 2011). The simplest way of using SfePy is to mesh the geometry using Gmsh and then solve a system of PDEs describing the physical problem in python programming language defined in a problem description file, also referred to as input file. In such a file, the problem is described using several keywords that allow one to define the equations, variables, finite element approximations, solvers, solution domain and subdomains etc. The syntax of the problem description file is very simple yet powerful, as the file itself is just a regular Python module that can be normally imported, no special parsing is necessary (Comparing Python to Other Languages, 1997). The keywords are regular Python variables, usually of the *dict* type with special names. The different components of problem description file are shown in Table 1.

Table 1: Structure of Problem description file

Serial No.	Keyword	Description
1	Finite Element Mesh	FE meshes defines domain geometry. Four types of geometry elements ar e supported (Figure 1).
2	Regions	Regions serve to select a certain part of the computational domain. It is the selection of nodes and elements of a FE mesh. They are used to define the boundary conditions, the domains of terms and materials etc.
3	Fields	Fields correspond to Finite Element spaces.
4	Integrals	Define the integral type and quadrature rule. This keyword is optional.
5	Variables	Variables use the Finite Element approximation given by the specified field.
6	Boundary Conditions	The boundary conditions apply in a given region given by its name, and, optionally, in selected times. The times can be given either using a list of tuples (t0, t1) making the condition active for t0 <= t < t1, or by a name of a function taking the time argument and returning True or False depending on whether the condition is active at the given time or not.
7	Initial conditions	Initial conditions are applied prior to the boundary conditions; no special care must be used for the boundary dof’s.
8	Materials	Materials are used to define constitutive parameters such as stiffness, permeability, or viscosity, and other non-field arguments of terms such as known traction or volume forces. Depending on a particular term, the parameters can be constants, functions defined over Finite Element mesh nodes, functions defined in the elements, etc.
9	Equations	Equations can be built by combining terms listed in Term Table. User can create the required term in Sfepy before building equation.
10	Solvers	In Sfepy, a non-linear solver has to be specified even when solving a linear problem. The linear problem is thus solved by one iteration of the nonlinear solver.
11	Functions	Functions are a way of customizing Sfepy behavior. They make it possible to define material properties, boundary conditions, parametric sweeps, and other items in an arbitrary manner. Functions are normal Python functions declared in the Problem definition file, so they can invoke the full power of Python. In order for Sfepy to make use of the functions, they must be declared using the function keyword.
12	Miscellaneous	The options can be used to select solvers, output file format, output directory, to register functions to be called at various phases of the solution, and for other settings.

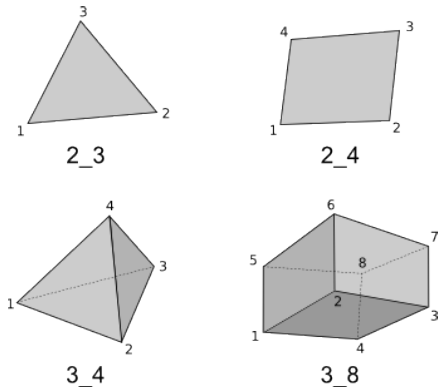

Figure 1: Supported geometry elements

3.2.2 Solution

While the pre-processing and post-processing phases of the finite element method are interactive and time-consuming for the analyst, the solution is often a batch process, and is demanding of computer resource. The governing equations are assembled into matrix form and are solved numerically. The assembly process depends not only on the type of analysis such as static or dynamic, but also on the model's element types and properties, material properties and boundary conditions.

The dataset prepared by the preprocessor is used as input to the finite element code itself, which constructs and solves a system of linear or nonlinear algebraic equations in the form $Kd = r$, where K is the system stiffness matrix, d is the nodal degree of freedom dof displacement vector, and r is the applied nodal load vector. The strain-displacement relation may be introduced into the stress-strain relation to express stress in terms of displacement. Under the assumption of compatibility, the differential equations of equilibrium in concert with the boundary conditions then determine a unique displacement field solution, which in turn determines the strain and stress fields. The chances of directly solving these equations are slim to none for anything but the most trivial geometries, hence the need for approximate numerical techniques presents itself.

Solution methods for finite element matrix equations are plentiful. In the case of the linear static $Kd = r$, inverting K is computationally expensive and numerically unstable. A better technique is Cholesky factorization, a form of Gauss elimination, and a minor variation on the "*LDU*" factorization theme. The K matrix may be efficiently factored into LDU, where L is lower triangular, D is diagonal, and U is upper triangular, resulting in $LDUd = r$. Since L and D are easily inverted, and U is upper triangular, d may be determined by back-substitution. Another popular approach is the wavefront method, which assembles and reduces the equations at the same time. Some of the best modern solution methods employ sparse matrix techniques. Because node-to-node stiffnesses are non-zero only for nearby node pairs, the stiffness matrix has a large number of zero entries. This can be exploited to reduce solution time and storage by a factor of 10 or more. Improved solution methods are continually being developed. The key point is that the analyst must understand the solution technique being applied.

Sfepy employs sparse matrix solver integrated in scipy library, hence it is quite fast to solve large matrices. Optimized mathematical library such as Umfpack, BLAS can also be used in sfepy during solution to obtain maximum speed (UMFPACK: unsymmetric multifrontal sparse LU factorization package, 2011).

3.2.3 Post-processing

After a finite element model has been prepared and checked, boundary conditions have been applied, and the model has been solved, it is time to investigate the results of the analysis. This activity is known as the post-processing phase of the finite element method. A typical postprocessor display overlays colored contours representing the unknown parameters on the model, showing a full-field picture.

Once the solution is verified to be free of numerical problems, the quantities of interest may be examined. Many display options are available, the choice of which depends on the mathematical form of the quantity as well as its physical meaning. For example, the displacement of a solid linear brick element's node is a 3-component spatial vector, and the model's overall displacement is often displayed by superposing the deformed shape over the undeformed shape. Dynamic viewing and animation capabilities aid greatly in obtaining an understanding of the deformation pattern. Stresses, being tensor quantities, currently lack a good single visualization technique, and thus derived stress quantities are extracted and displayed. Principal stress vectors are displayed as color-coded arrows, indicating both direction and magnitude. The magnitude of principal stresses or of a scalar failure stress such as the Von Mises stress are displayed on the model as colored bands.

Sfepy uses matplotlib for 2D and Mayavi for 3D visualization of solution results (Ramachandran, 2011). Paraview is also used as a fully functional post-processor (ParaView - Open Source Scientific Visualization, 2011).

3.3 Evaluation

The typical process to solve a problem using the developed framework is as followes, a model is meshed in Gmsh, a problem definition file is drafted in python programming language, SfePy is run to solve the problem and finally the results of the analysis are visualised. The tensile strength test of concrete which is also known as split cylinder test is simulated with the developed finite element analysis framework (Figure 2(a)). In this test a cylindrical specimen is loaded across its diameter to failure. To model this problem using finite elements the indirect tensile test can be simplified to represent a diametrically point loaded disk as shown in Figure 2(b).

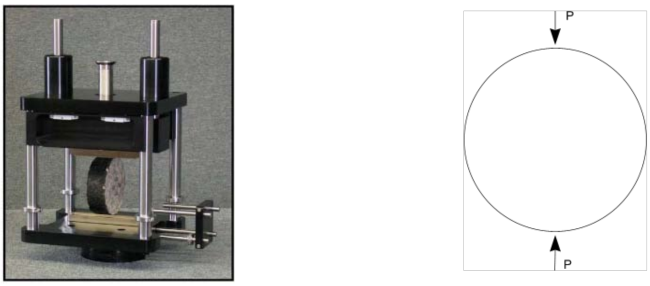

Figure 2: (a) Indirect Tesnsile Strength test setup, (b) Simplified 2D diametrically point loaded disk

The tensile and compressive stresses that develop in the specimen as a result of the point loads *P* are a function of the diameter (*D*) and thickness (*t*) of the cylindrical specimen. At the centre of the specimen, the compressive stress is 3 times the tensile stress and the analytical formulation for these are, respectively:

$$\sigma_t = \frac{2P}{\pi t D} \tag{1}$$

$$\sigma_c = \frac{6P}{\pi t D} \tag{2}$$

These solutions are approximated using finite element methods and the finite element solution is finally compared to the analytical solution to verify the developed FEA framework.

3.3.1 Pre-processing

The specimen has a diameter of 150 mm. Assuming plane strain conditions, the indirect tensile test may be modelled using a 2D finite element mesh. Furthermore, the geometry of the model is symmetrical about the x- and y-axes passing through the centre of the circle. To take advantage of this symmetry only one quarter of the 2D model will be meshed and boundary conditions will be established to indicate this symmetry. The meshing program Gmsh is used here to very quickly mesh the model (Gmsh, 2011). In meshing, Triangle elements are used (Figure 1).

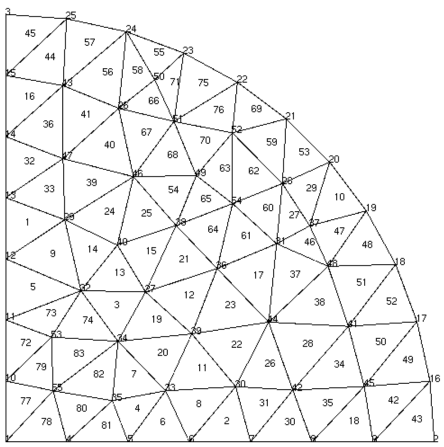

Figure 3: meshed geometry

3.3.2 Solution

For the analysis it is assumed that the material of the test specimen is linear elastic and isotropic. The problem definition file is named as *its2D_1.py* and is in the same directory as the *simple.py* script. The problem definition file is shown in Appendix A. Two material constants are defined, Young's modulus and Poisson's ratio. The material is assumed to be asphalt concrete having a Young's modulus of 2,000 MPa and a Poisson's ration of 0.4.

Four regions are defined:

1. Omega: all the elements in the mesh
2. Left: the y-axis
3. Bottom: the x-axis
4. Top: the topmost node. This is where the load is applied.

To ensure symmetry about the x-axis, the vertical or y-displacement of the nodes in the Bottom region are prevented or set to zero. Similarly, for symmetry about the y-axis, any horizontal or displacement in the x-direction of the nodes in the Left region or y-axis is prevented. Material constants are provided in terms of Young's modulus and Poisson's ratio, but the linear elastic isotropic equation used requires as input Lamé's parameters. The *youngpoisson_to_lame* function is thus used for conversion.

One option to solve the problem is to run the SfePy *simple.py* script from the command shell:

```
$ ./simple.py its2D_1.py
```

SfePy solves the problem and outputs the solution to the output path provided in the problem description file. The output file will be in the vtk format by default if this is not explicitly specified and the name of the output file will be the same as that used for the mesh file except with the vtk extension.

3.3.3 Post-processing

SfePy includes a post-processing script to quickly view the solution.

```
$ ./postproc.py its2D.vtk
```

The *postproc.py* script generates the image shown below, which by default shows the displacements in the model as arrows and their magnitude as color scale.

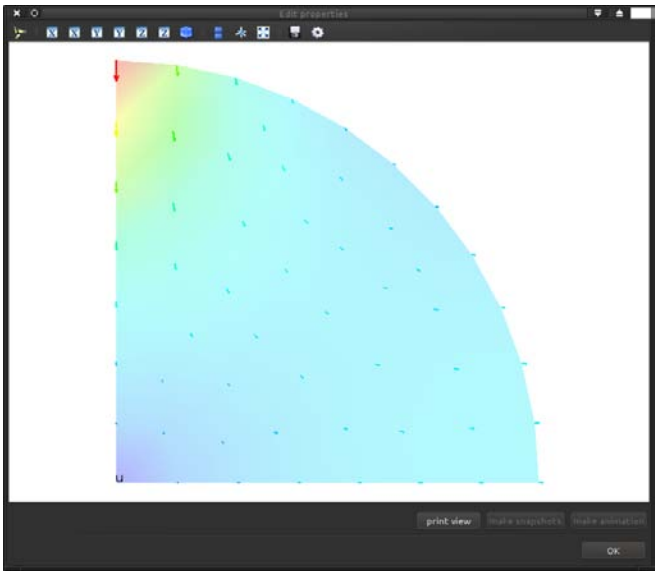

Figure 4: Visualization of the solution

The stresses in the model had been calculated but these were averaged from those calculated at Gauss quadrature points within the elements. It is possible to provide custom integrals to allow the calculation of stresses with the Gauss quadrature points at the element nodes. This will provide a more accurate estimate of the stress at the centre of the specimen located at node 0. The new problem definition file is named as *its2D_2.py* and is executed in the same directory as the *simple.py* script. The analytic solution is independent of mesh refinement influence, so it'll always be same for as long as the whole geometry remains intact. On the other hand, the finite elemnt analysis solution is influenced by mesh refinement. The finer the mesh is, the more accurate the result will be. A fine mesh of the same geometric property is drafted and again *its2D_2.py* script is executed to obtain the solution. The FEA result for both the coarse and fine mesh is shown in Table 2.

Table 2: Analysis summary

<table>
<tr><th>Mesh type</th><th>Applied load (N)</th><th>Analytic horizontal tensile stress (MPa/mm)</th><th>FEA horizontal tensile stress (MPa/mm)</th><th>% error w.r.t Analytic solution</th><th>Analytic vertical compressive stress (MPa/mm)</th><th>FEA vertical compressive stress (MPa/mm)</th><th>% error w.r.t Analytic solution</th></tr>
<tr><td>Coarse - 83 Elements</td><td>2000</td><td rowspan="2">8.48826</td><td>7.57220</td><td>+10.79</td><td rowspan="2">25.4648</td><td>25.8660</td><td>-1.57</td></tr>
<tr><td>Fine - 5568 Elements</td><td>2000</td><td>8.50042</td><td>-0.14</td><td>25.4300</td><td>+0.13</td></tr>
</table>

From Table 2 it is established that the developed FEA framework works correctly as the FEA approximate solution is very close to the analytical solution. It has also been observed that with certain amount of mesh refinement, the FEA solution will be well converged. From table 2, results obtained using fine mesh converges better towards zero error.

4. CONCLUSION

Taking the advantages of full control over the Engineering simulation using custom coded finite element analysis software along with its structure and working procedure is focused in this paper. It has been established that by using open source finite element analysis software developed from freely available scientific python tools, one can optimize the code by great extent. As the source code is freely available to use and modify, many user can contribute their individual knowledge regarding various aspect of numerical modeling. The use of one of the most popular scripting language python, results in readability increment of the source code.

This paper also focuses on the necessity & easiness of developing engineering simulation framework. The evaluation of the developed framework is presented with comparison to an analytical solution. It has been observed that development and use of open source engineering simulation software offers four main benefits:

1. Anyone can contribute to the continuing development of the software, as it is open source.
2. Optimum simulation is obtained as the user can see what is running behind the scene.
3. The use of legacy mathematical libraries written in static language offers great code reuse and saves time; also it is easier to change the underlying legacy library.
4. Use of freely available libraries provides maximum economy.

ACKNOWLEDGMENTS

The Author is an active developer of “Sfepy” open source finite element analysis framework and would like to extend thanks to co-developers. The author make extensive use of freely available software such as GNU/Linux operating system, GCC compilers, Python, SciPy, Pythonxy, Enthought python distribution and Lapack, Blas, Umfpack and others. Many thanks to open source community for those excellent products.

APPENDIX A: Problem definition file named *its2D_1.py*

```
from sfepy.mechanics.matcoefs import youngpoisson_to_lame
from sfepy.fem.utils import refine_mesh
from sfepy import data_dir

# Fix the mesh file name if you run this file outside the SfePy directory.
filename_mesh = data_dir + '/its2D.mesh'

refinement_level = 0
filename_mesh = refine_mesh(filename_mesh, refinement_level)

output_dir = '.' # set this to a valid directory you have write access to

young = 2000.0 # Young's modulus [MPa]
poisson = 0.4  # Poisson's ratio
```

```
options = {
    'output_dir' : output_dir,
}

regions = {
    'Omega' : ('all', {}),
    'Left' : ('nodes in (x < 0.001)', {}),
    'Bottom' : ('nodes in (y < 0.001)', {}),
    'Top' : ('node 2', {}),
}

materials = {
    'Asphalt' : ({
        'lam' : youngpoisson_to_lame(young, poisson)[0],
        'mu' : youngpoisson_to_lame(young, poisson)[1],
    },),
    'Load' : ({'.val' : [0.0, -1000.0]},),
}

fields = {
    'displacement': ('real', 'vector', 'Omega', 1),
}

equations = {
   'balance_of_forces' :
   """dw_lin_elastic_iso.2.Omega(Asphalt.lam, Asphalt.mu, v, u )
      = dw_point_load.0.Top(Load.val, v)""",
}

variables = {
    'u' : ('unknown field', 'displacement', 0),
    'v' : ('test field', 'displacement', 'u'),
}

ebcs = {
    'XSym' : ('Bottom', {'u.1' : 0.0}),
    'YSym' : ('Left', {'u.0' : 0.0}),
}

solvers = {
    'ls' : ('ls.scipy_direct', {}),
    'newton' : ('nls.newton', {
        'i_max' : 1,
        'eps_a' : 1e-6,
        'problem' : 'nonlinear'
    }),
}
```

APPENDIX B: Problem definition file named *its2D_2.py*

```
from its2D_1 import *

from sfepy.mechanics.matcoefs import stiffness_tensor_youngpoisson
from sfepy.fem.geometry_element import geometry_data
from sfepy.fem import Field,FieldVariable
import numpy as nm

gdata = geometry_data['2_3']
nc = len(gdata.coors)
```

```
def nodal_stress(out, pb, state, extend=False):
    '''
    Calculate stresses at nodal points.
    '''

    # Point load.
    mat = pb.get_materials()['Load']
    P = 2.0 * mat.get_data('special', None, 'val')[1]

    # Calculate nodal stress.
    pb.time_update()

    stress = pb.evaluate('dq_cauchy_stress.ivn.Omega(Asphalt.D, u)', mode='qp')
    sfield = Field('stress', nm.float64, (3,), pb.domain.regions['Omega'])
    svar = FieldVariable('sigma', 'parameter', sfield, 3,
                    primary_var_name='(set-to-None)')
    svar.data_from_qp(stress, pb.integrals['ivn'])

    print '\n================================================================'
    print 'Given load = %.2f N' % -P
    print '\nAnalytical solution'
    print '==================='
    print 'Horizontal tensile stress = %.5e MPa/mm' % (-2.*P/(nm.pi*150.))
    print 'Vertical compressive stress = %.5e MPa/mm' % (-6.*P/(nm.pi*150.))
    print '\nFEM solution'
    print '============'
    print 'Horizontal tensile stress = %.5e MPa/mm' % (svar()[0][0])
    print 'Vertical compressive stress = %.5e MPa/mm' % (-svar()[0][1])
    print '=============================================================='
    return out

asphalt = materials['Asphalt'][0]
asphalt.update({'D' : stiffness_tensor_youngpoisson(2, young, poisson)})
options.update({'post_process_hook' : 'nodal_stress',})

integrals = {
    'ivn' : ('v', 'custom', gdata.coors, [gdata.volume / nc] * nc),
}
```

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

INVESTIGATION OF BAMBOO AS A POTENTIAL REINFORCEMENT IN THE CONCRETE

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ABSTRACT

This research was undertaken to investigate the possibility of using bamboo as a potential reinforcement in the concrete beam to compensate the low tensile property of the concrete. Even though steel reinforcement is a very suitable material for complementing concrete's low tensile strength, considering the cost, some parts of the world people build their houses by using only concrete which is very vulnerable. To overcome these problems, bamboo may be the alternative materials to substitute the reinforcing bar in concrete for less important structures. Two beams with same cross section were constructed and tested under two point loading considering reinforcement ratio is the main parameter. To improve the bond strength, pierced nails were used for both ends. Split bamboo was used as shear reinforcement. From bending tests after fifty days, the ultimate failure load was obtained which was compared with the corresponding failure load of steel reinforced concrete beam that was found up to forty percent. The calculated initial cracking load and failure load has been found to be alike to the experimental load. From the results it can be opined that bamboo reinforced beams follows the constitutive laws of the corresponding steel reinforced concrete beams and can be used with some limitations as substitute of steel reinforcement.

Keywords: *Bamboo reinforcement, beam, bending property, cracking load, flexural strength.*

1. INTRODUCTION

One of the properties that would make bamboo a good substitute to steel in reinforced concrete is its strength. The strength of bamboo is greater than many timber products which are advantageous, but it is quite less than the tensile and bending property of steel. Bamboo is easily accessible as it is available in almost every tropical and subtropical regions, this lowers the cost of construction and increases the strength of the buildings that would otherwise be unreinforced. In any reinforced concrete member, strength and deformation under load are very critical criteria. If bamboo is used as reinforcement in the concrete it would have to follow the constitutive laws of concrete structures. Therefore, it was decided to investigate the bending property of bamboo reinforced beam under two point loading.

2. OBJECTIVES OF THE RESEARCH

Whereas the mechanical properties and behavior of steel reinforcement have been thoroughly studied and well documented, there exists no comprehensive data describing bamboo reinforcement. Therefore, the aim of this study is to provide a preliminary contribution toward the collection of the mechanical properties and behaviors of bamboo reinforcement. Some of the previous researches are mentioned in connection with this. The mechanical properties vary with height and age of the bamboo culm. Research findings indicate that the strength of bamboo increases with age. The optimum strength value occurs between 2.5 and 4 years. The strength decreases at a later age (Amada and Untao , 2001). Amada et al. (1997) investigated the mechanical and physical properties of bamboo. They conducted a thorough investigation into the structure and purposes of the nodes, which they found to strengthen the bamboo culm (Amada et al., 1997). Lo et al. (2004) gave a detailed description of the mechanical properties of bamboo in their study. They found that the physical, as well as mechanical attributes vary with respect to diameter, length, age, type, position along culms, and moisture content of bamboo (Lo et al., 2004). Amada and Untao (2001) studied the fracture properties of bamboo. In contradiction to other studies, this study states that the tensile strength of bamboo fibers almost corresponds to that of steel (Amada and Untao, 2001). Ghavami (1995) discussed the mechanical properties of bamboo, specifically pertaining to bamboo in concrete. This study showed that the ultimate strength of a concrete beam reinforced with bamboo is approximately 4 times when compared with un-reinforced concrete (Ghavami, 1995).

3. SAMPLE PREPARATION

First a bamboo was divided into two piece length wise as shown in Figure 1 and Figure 2 with the carpenter's tools like hammer, chisel etc. Each of the two halves was further divided into three pieces. It was then cleaned and finally rounded to shape of a rod as shown in Figure 3 and Figure 4.

Figure 1: Hammering bamboo with a chisel

Figure 2: Bamboo splitting into two pieces

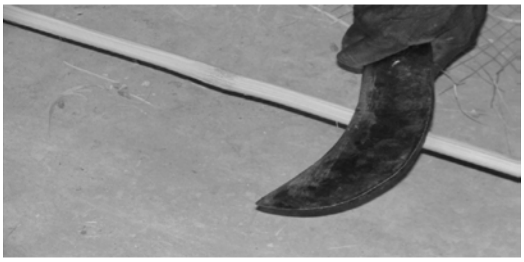

Figure 3: Preparation of sample bamboo by using fem

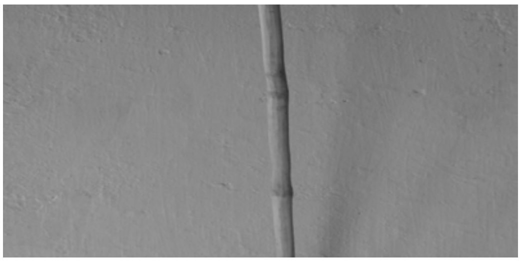

Figure 4: Prepared bamboo sample as reinforcement

4. TENSION TEST

Proper gripping is an important factor for tensile test. Bamboo is relatively soft material than the materials used for gripping purpose in universal testing machine. At the time of tension tests, early failure was observed at the gripping end possibly due to high stress developed from lateral compression. Moreover, the surface of the bamboo is very slippery and therefore the samples in some cases experienced slip at the time of tension test. To solve this gripping problem GI wires (2mm diameter) were wringed spirally at both ends of the specimen. The test setup is shown in Figure 5. The stress-strain characteristics of bamboo has been derived from the results of this tension test following the standard procedure (ASTM E6) as shown in the Figure 6. Therefore, from the figure, the yield strength f_y = 109 MPa. To be on the conservative side the value of f_y=105.7 MPa was used to calculate the cracking load and also the ultimate load that the bamboo reinforced beam can sustain. The modulus of elasticity was found to be 51428.6 MPa.

5. VARIABLES CONSIDERED FOR BENDING TEST

Due to unknowns associated with the behavior of bamboo reinforced concrete, the percentages of reinforcement were taken as 1.5% and 2.5%. Considering the laboratory facilities, a beam of 2438 mm length and 203 mm $\times$ 406 mm cross section was chosen for bending test. The variables considered are shown in Table 1. To increase the bond strength at the end pierced nails were used through the 762 mm from both ends since the end anchorage is not possible for bamboo reinforcement. Finally, the whole bamboo structure was coated with tar for water proofing.

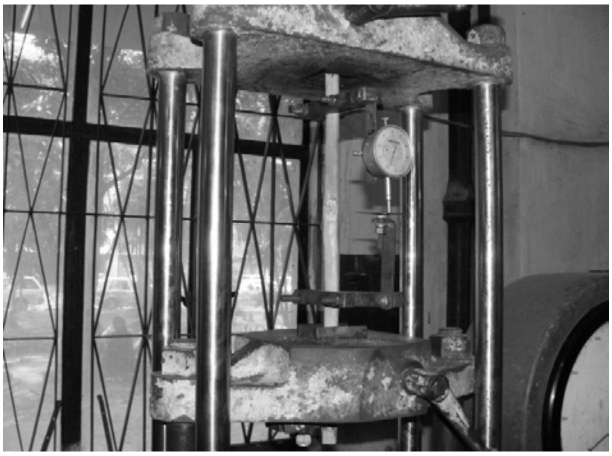

Figure 5: Test setup for tension test

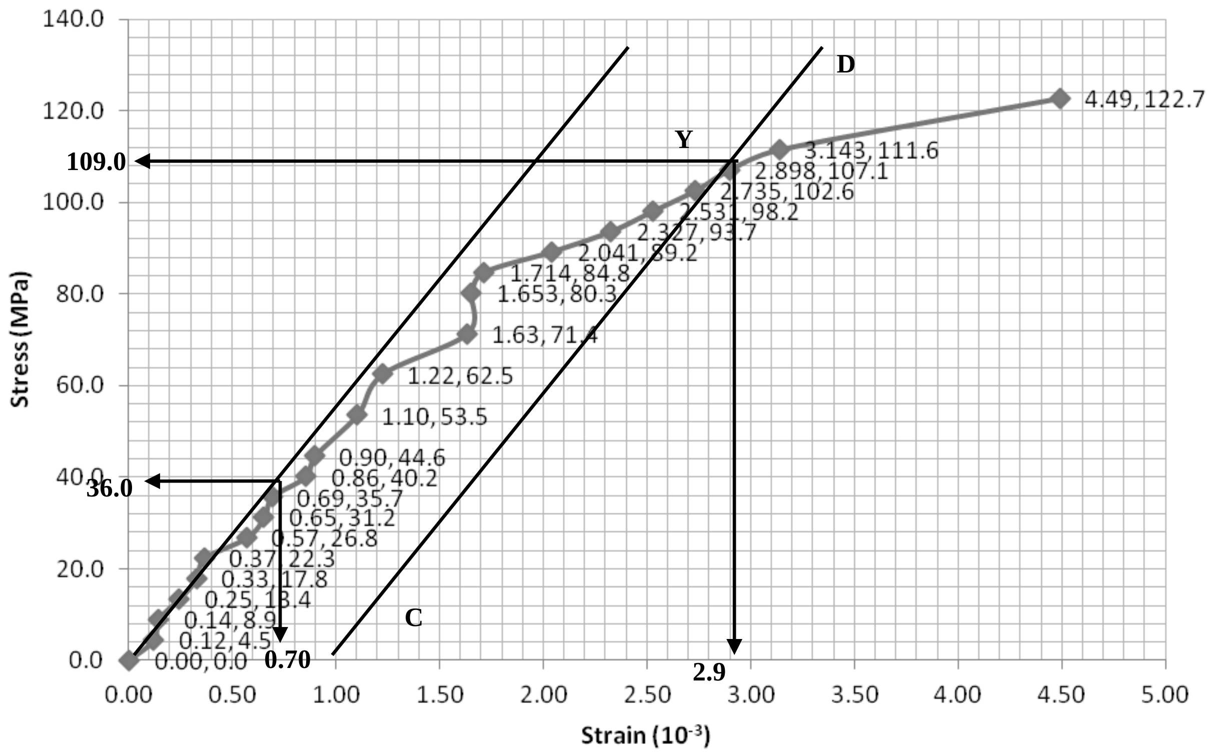

Figure 6: Stress- strain curve of bamboo samples

Table 1: Parameters considered for beam tests

	Test 1	**Test 2**
Reinforcement type	Bamboo	Bamboo
Main reinforcement (%)	1.5%	2.5%
Shear reinforcement	Split bamboo	Split bamboo
Effective depth (d), mm	359	329

6. PREPARATION OF BAMBOO REINFORCED BEAM

Reinforcement ratio: 1.5%

Dimension: 203 mm width × 406 mm depth × 2438 mm length

Reinforcement Amount: The total amount of required reinforcement (1.5%) = $\frac{1.5\times 203\times 406}{100}$ = 1237 mm^2

The sequence of preparation is illustrated step by step as follows:
i. At first the average diameter of each bamboo reinforcement was measured and from this the average area was calculated. It was observed that total 5 nos of bamboo rod was required. The area of individual bamboo specimen is shown in Table 2. The clear cover in each direction was 1 in and clear spacing between reinforcements was at least 25 mm.

Table 2: Area of individual bamboo reinforcement (1.5% reinforcement)

Reinforcement no.	**Avg. reinforcement area(mm^2)**
1.	361
2.	168
3.	342
4.	187
5.	181
Total	1237

ii. To increase the bond strength, at the end pierced nails were used through the 762 mm from both ends since the end anchorage is not possible for bamboo reinforcement. The nails are pierced at 51 mm spacing and any two adjacent nails were driven at right angle to each other as shown in the Figure 7.

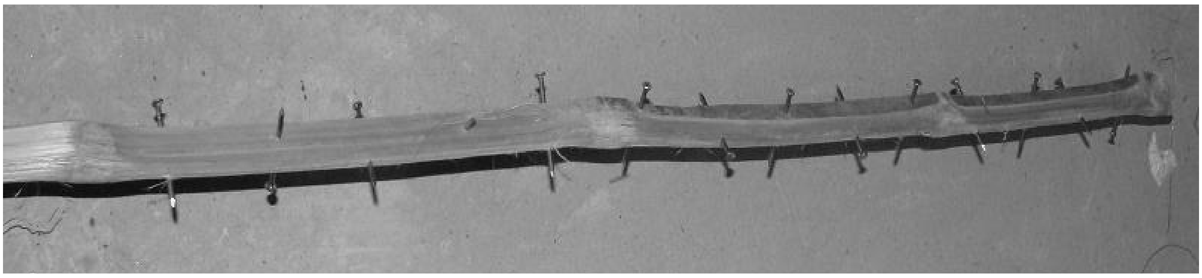

Figure 7: Pierced nails at the end of bamboo reinforcement

iii. Shear reinforcements were used at 102 mm spacing from both ends upto 914 mm and at 152 mm spacing for the middle 610 mm. Split bamboos were used for shear reinforcement. The thickness of this split bamboo was 3 mm and width was approximately 13 mm. In dry state it was not possible to provide U shape for shear reinforcement because of its brittle behavior. To make the specimen ductile it was soaked in water for two days as shown in the Figure 8. At this stage, the split bamboos were able to sustain any form of bending. A wooden form was made according to the shape of the shear reinforcement and the split water soaked bamboos were wound around this form and tied at the end to make the shear reinforcements as shown in Figure 9.

Figure 8: Split Bamboos are immersed in water

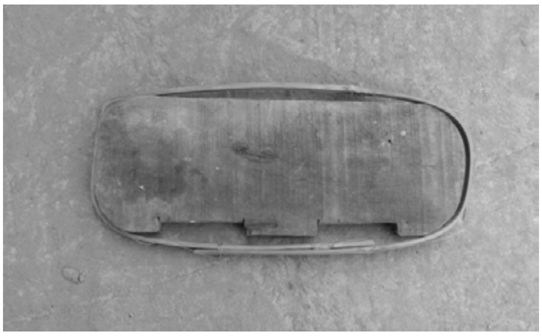

Figure 9: Water soaked split bamboo was wounded around the wooden form

iv. The main bamboo reinforcement (long bars) were placed alternatively head and tail to satisfy the uniform cross section requirement throughout the beam and tied with shear reinforcement as shown in Figure 10 and Figure 11.
v. Finally, the whole bamboo structure was coated with tar for water proofing as shown in Figure 12.

Figure 10: Alternatively placed bamboo rod

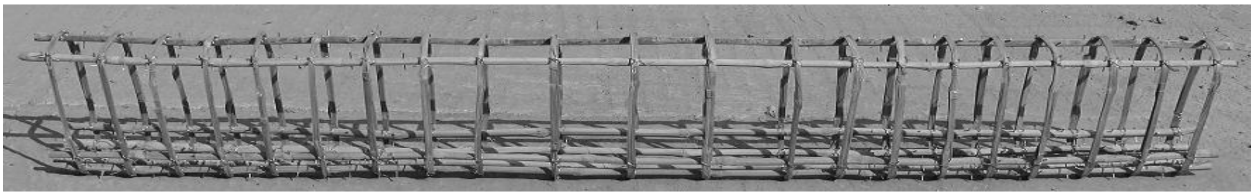

Figure 11: Prepared bamboo beam reinforcement

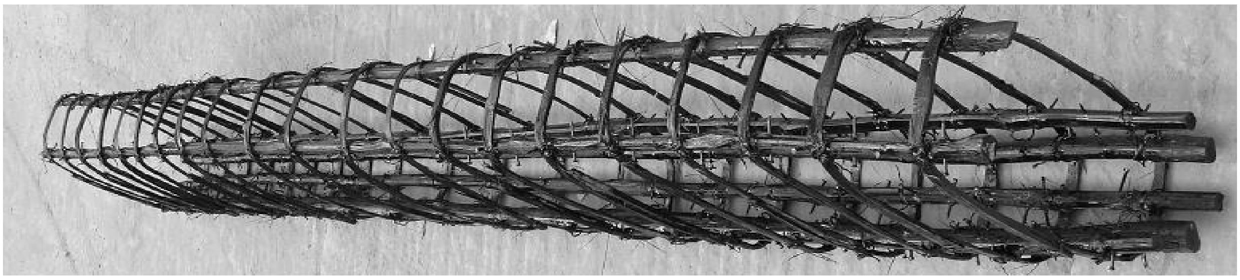

Figure 12: Bamboo reinforcement coated with tar

Reinforcement ratio: 2.5%

Dimension: 203 mm width $\times$ 406 mm depth $\times$ 2438 mm length

Reinforcement amount: The total amount of required reinforcement(2.5%)= $\frac{2.5 \times 203 \times 406}{100}$= 2060 mm^2

The sequence of preparation is illustrated step by step as follows:

i. At first the average diameter of each bamboo reinforcement was measured and from this the average area was calculated. It was observed that total 9 nos of bamboo rod was required. The area of individual bamboo specimen is shown in Table 3. The clear cover in each direction was 1 in and clear spacing between reinforcementss was at least 25 mm.

Table 3: Area of individual bamboo reinforcement (2.5% reinforcement)

Reinforcement no.	**Avg. reinforcement area(mm^2)**
1.	277
2.	168
3.	265
4.	174
5.	271
6.	181
7.	258
8.	206
9.	277
Total	2077

ii. The rest of the procedures are same as the preparation of 1.5% bamboo reinforcement preparation. Some figures are shown below.

Figure 13: Alternatively placed bamboo reinforcement

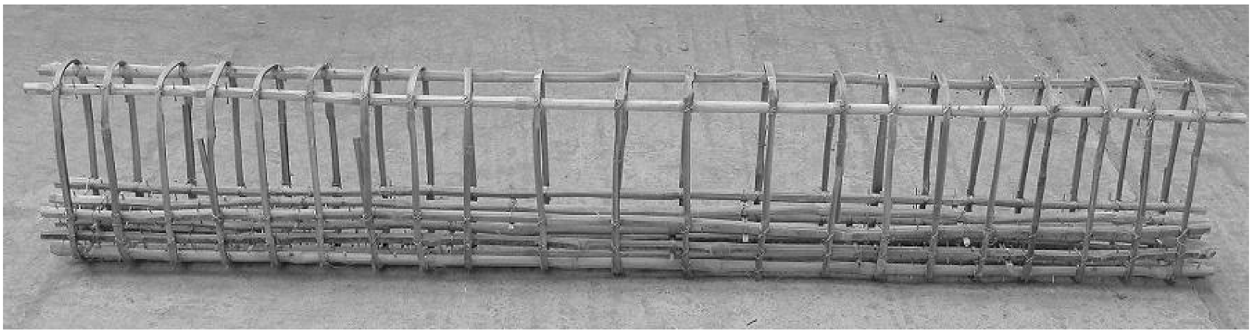

Figure 14: Prepared 2.5% bamboo reinforcement structure

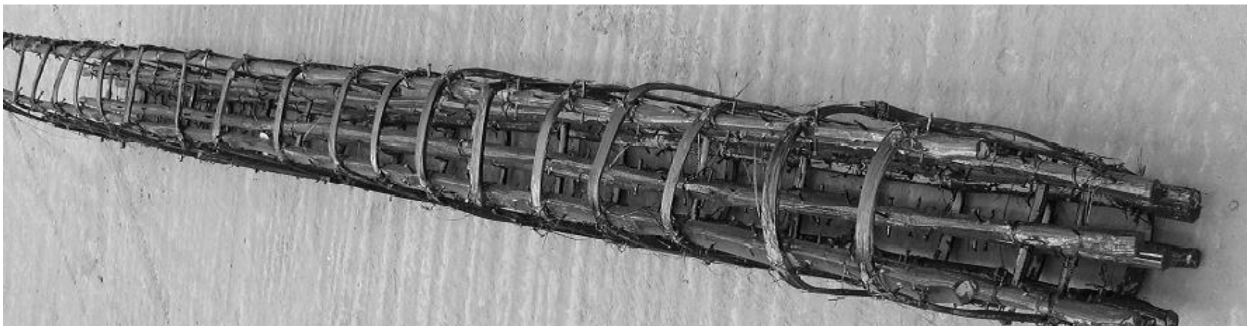

Figure 15: Bamboo reinforcement coated with tar

7. CONCRETE MIX PROPORTION

The concrete used for the beams was made using the Portland cement, Sylhet sand as the fine aggregate and brick chips as the coarse aggregate with the maximum size of 19 mm. The unit weight of Sylhet sand= 1450 kg/m^3. The unit weight of Brick chips = 1100 kg/m^3. The concrete mix proportion was 1: 1.5 : 3 (cement : fine aggregate : coarse aggregate) and water cement ratio was 0.41. The slump value was approximately 1.5 in. The concrete was mixed in two batches and then poured into the formwork which is shown in the Figure 16. Three cylinders (102 mm dia and 203 mm height) were also prepared (as per ASTM) for compression tests. This was done by pouring them full of the same concrete used in the beam and they were cured in water. The cylinder strength was taken at the day of testing of the beams. After three days the wooden shutter was removed and covered with wet jute sack to maintain the moisture. The beams were cured with water twice in every day until the testing date.

Figure 16: Smoothly finished concrete

8. TEST SET-UP, INSTRUMENTATION AND DATA ACQUISITION

After 48 days the beams were tested. Before testing all the beams were all through white washed and 51 mm blocks were drown on one side of the beam to find out the crack and their absolute location. The test set-up began with picking up the beam by the crane and then placed under the testing machine as shown in the Figure 17. The left support was provided as hinge and right support was provided as roller support. Deflection gages were placed in the critical areas of the beam to follow and record the deflection behavior. One deflection gage was placed at L/2 distance from the support, in the area of maximum bending moment and the second deflection gage was placed at L/4 distance from the left support. All tests were conducted at a/d = 2. The loading steps were selected at 4.4 kN increment. After cracking occurred, the beam was loaded up to failure. The crack formation was observed carefully with loading.

9. RESULTS OF BENDING TESTS FOR BAMBOO REINFORCED BEAMS

Two beams of similar cross section with different reinforcement ratio were tested by two point loading. The results of testing of these beams are presented in the following sections.

9.1 Beam with 1.5% Bamboo Reinforcement

The first test was conducted with water proofed bamboo reinforcement 1.5% with a/d ratio of 2 and the distance between the loads is 699 mm. The failure mode and the corresponding loads are shown in Figure 18. The load was applied incrementally from zero to ultimate failure.

The first crack appeared due to combined action of bending and shear at 39.6 kN which propagated to 381mm from the bottom of the beam at 61.6 kN load and the location of the crack is 432 mm right from the center of the beam. The second crack mainly appeared due to bending at 44 kN at the bottom surface which propagated to 381 mm from the bottom of the beam at 110 kN load and the location of the crack is 127 mm left from the center of the beam. Another crack appeared at 44 kN which is possibly due to combined action of bending and shear which propagated to 381 mm at 79 kN load and its location was 406 mm left from the centre of the beam. The third crack appeared due to combined action of bending and shear at 70.4 kN which propagated to 381 mm from the bottom of the beam and the location of the crack is 737 mm right from the center of the beam. The fourth crack appeared due to combined action of bending and shear at 110 kN which propagated to 381 mm from the bottom of the beam and the location of the crack is 813 mm right from the center of the beam. It was a typical inclined shear crack pattern. The formation and propagation of all cracks is similar to reinforced concrete beam. The maximum load recorded was 110 kN. The failure load was defined at crack propagation upto approximately 90% depth of the beam. The final failure pattern has been shown in Figure 19.

The load deflection curve at mid span and quarter span has been presented in Figure 20. At 39.6 kN load the first crack appeared and after this the slope of the curve deviated slightly. From this curve it can be seen that the beam stiffness reduces considerably for every crack formation and propagation.

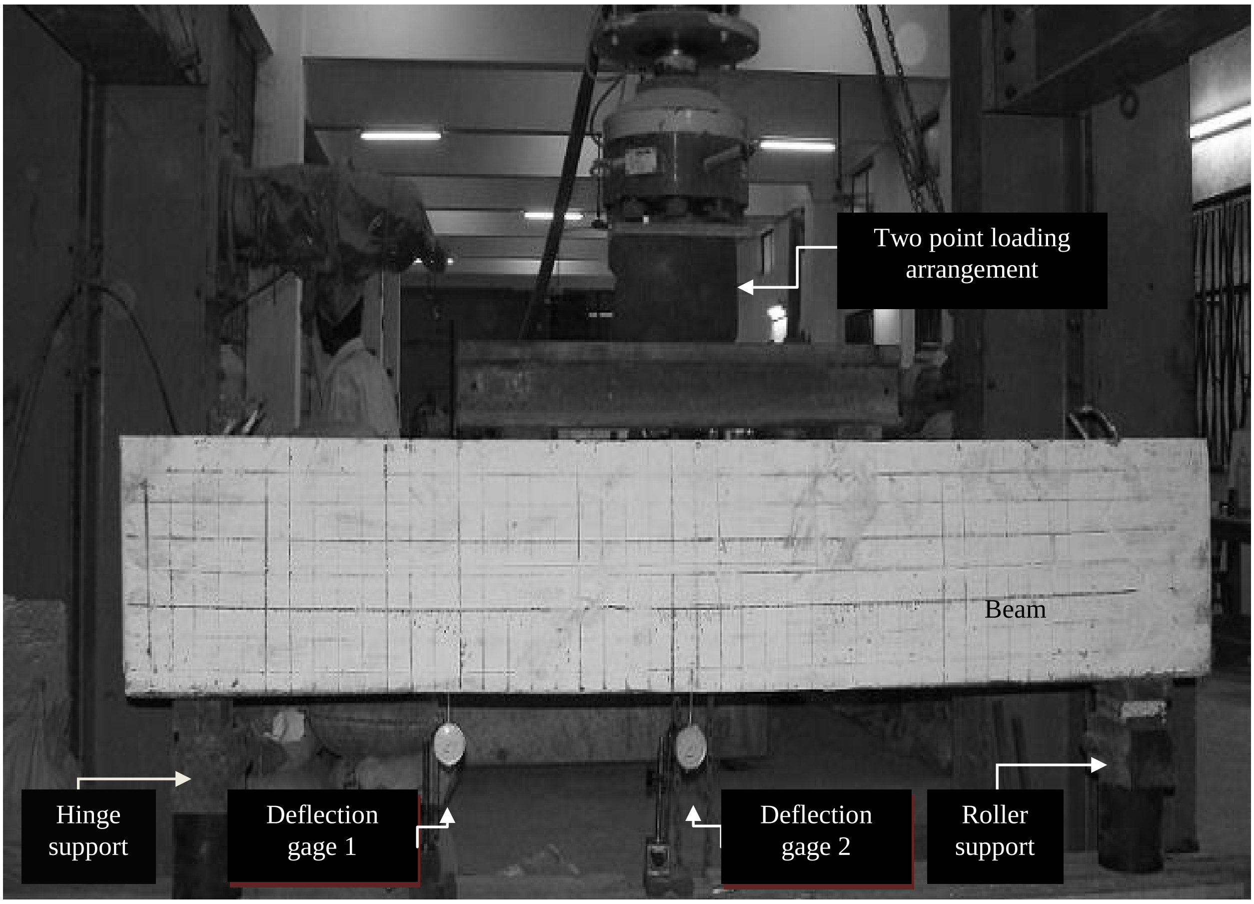

Figure 17: Test set-up

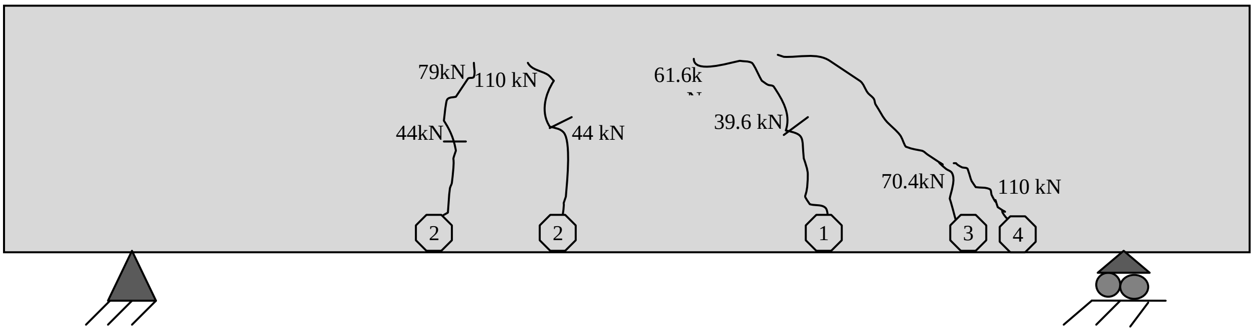

Figure 18: Final crack patterns for1.5% bamboo reinforced beam

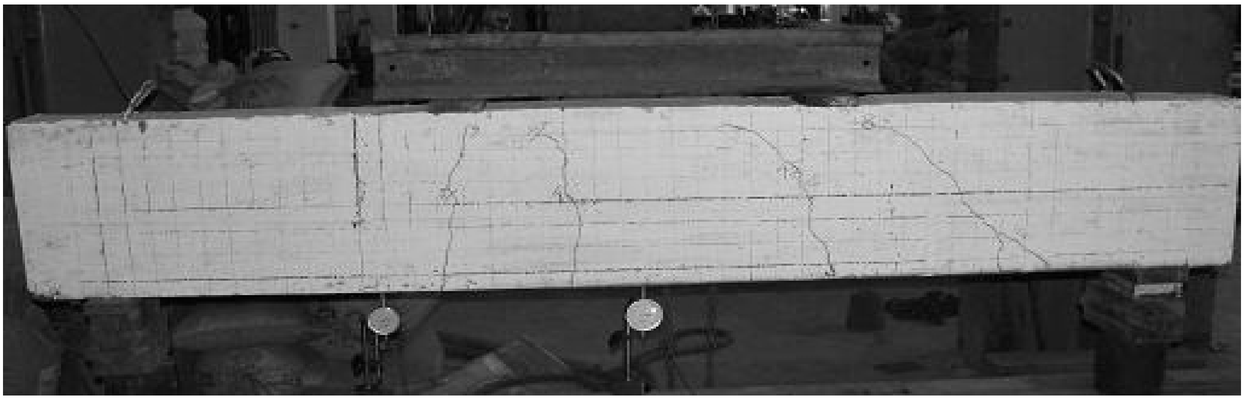

Figure 19: Failure pattern for 1.5% bamboo reinforced beam

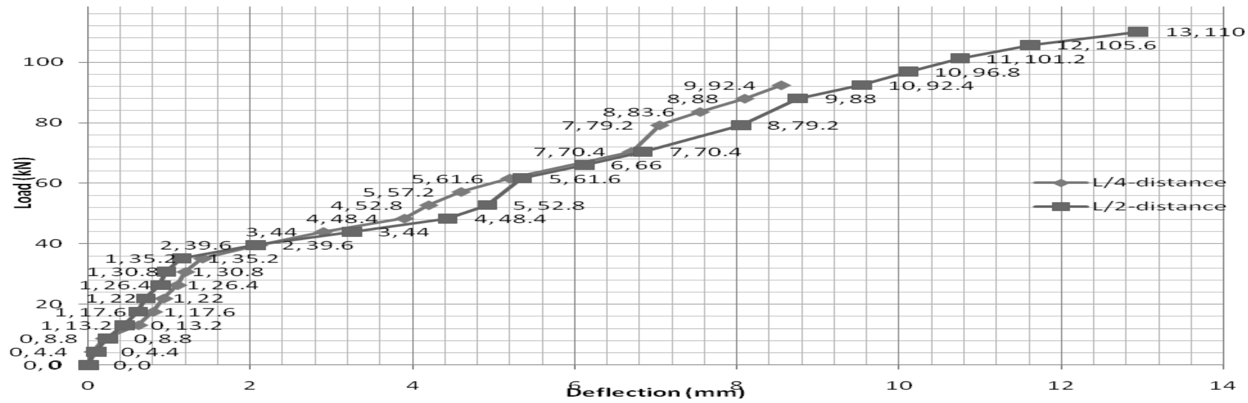

Figure 20: Load deflection curve for 1.5% bamboo reinforced beam

9.2 Beam with 2.5% Bamboo Reinforcement

The test was conducted with water proofed 2.5% bamboo reinforcement with a/d ratio of 2 and the distance between the loads is 817 mm. The failure mode and the corresponding loads are shown in Figure 21. The load was applied incrementally from zero to ultimate failure.

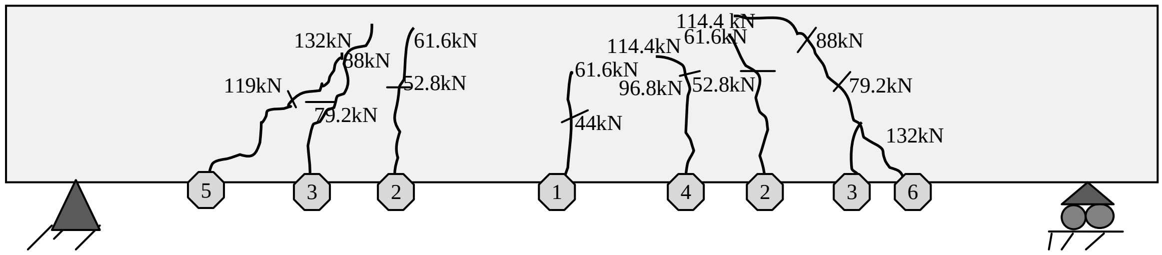

Figure 21: Final crack pattern of 2.5% bamboo reinforced beam

The first crack appeared due to bending at 44 kN which propagated to 254 mm from the bottom of the beam at 61.6 kN load and the location of the crack is 25 mm left from the center of the beam. The second crack appeared due to bending at 52.8 kN which propagated to 381 mm from the bottom of the beam at 61.6 kN load and the location of the crack is 381 mm left and right from the center of the beam. The third crack appeared due to combined action of bending and shear appeared at 79.2 kN which propagated to 381 mm from the bottom of the beam at 88 kN load. The location of the crack is 584 mm left from the center of the beam and another crack propagated to 381 mm from the bottom of the beam at 114.4 kN which is 584 mm right from the center of the beam. The fourth crack appeared due to bending at 96.8 kN which extended to 305 mm from the bottom of the beam at 114.4 kN load and the location of the crack is 203 mm right from the center of the beam. The fifth crack appeared due to shear at 119 kN which propagated to 356 mm from the bottom of the beam at 132 kN load and the location of the crack is 31 mm left from the center of the beam. It was a perfectly inclined shear crack pattern. The sixth crack appeared due to combined action of bending and shear appeared at 132 kN which propagated to 381 mm from the bottom of the beam and the location of the crack is 711 mm left from the center of the beam. The maximum load recorded was 132 kN. The failure load was defined at crack propagation up to approximately 90% depth of the beam. The final failure pattern has been shown in Figure 22. The load deflection curve at mid span and quarter span has been presented in Figure 23. At 44 kN load the first crack appeared and after this the slope of the curve deviated significantly. From this curve it can be seen that the beam stiffness reduces considerably for every crack formation and propagation.

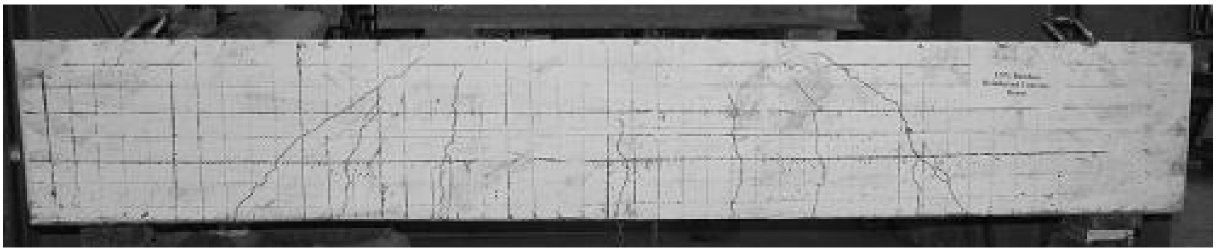

Fig.ure 22: Failure pattern for 2.5% bamboo reinforced beam.

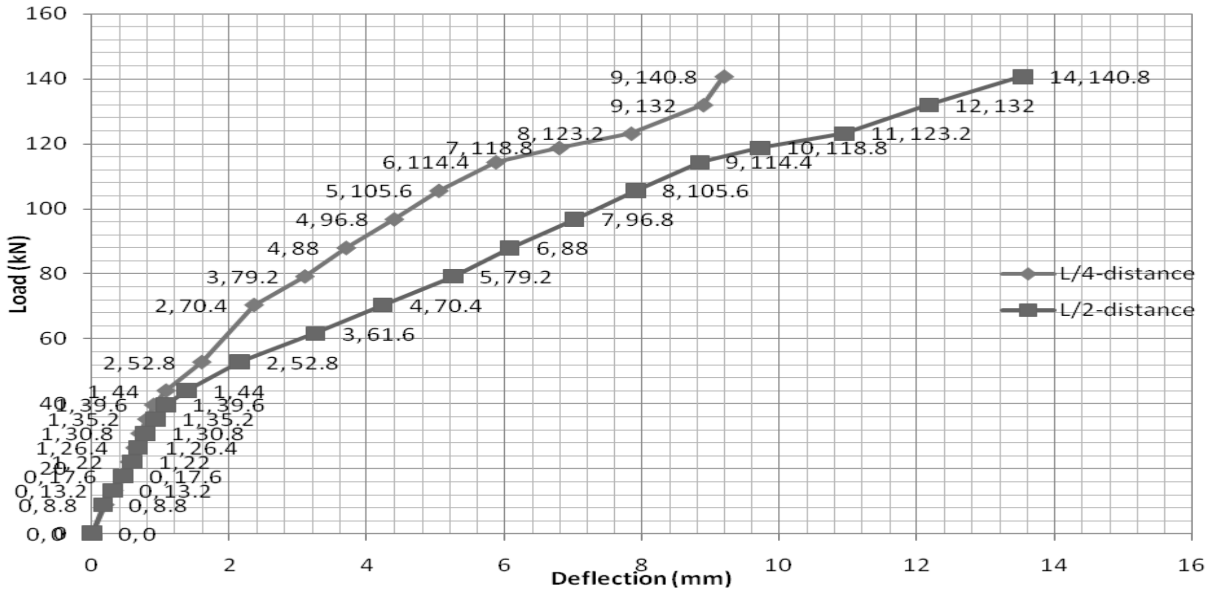

Figure 23: Load deflection curve for 2.5% bamboo reinforced beam

The results of two point bending test of beams with different bamboo reinforcement ratios are summarized in Table 4 and Table 5 respectively.

Table 4: Comparison of two point bending beam test results

Test designation	**1.5% Bamboo reinforced beam**	**2.5% Bamboo reinforced beam**
Test results		
Failure load (kN)	110	140.8
Moment at failure (kN-m)	39.4	46.3
Load at first crack (kN)	39.6	44

Table 5: Calculated values for load/moment

Test Designation	**1.5% Bamboo Reinforced Beam**	**2.5% Bamboo Reinforced Beam**
Calculated failure load (kN)	112.2	182.6
Calculated Failure Moment (kN-m)	40.6	60.1
Modulus of Rupture, $f_r = 7.5\sqrt{f'_c}$ (MPa)	32	32
Calculated First Cracking Load (kN)	50.6	55

Yield strength, (f_y), of bamboo = 105.7 MPa
Concrete strength of concrete = 27.3 MPa.
The load-deflection curve for bamboo reinforced beam and their comparison are shown in Figure 24 and Figure 25 respectively
At 1.5% reinforcement the 1st crack initiated by combine shear and bending but for 2.5% reinforcement the 1st crack initiated by bending alone. From Table 4 and Table 5 it can be seen that the calculated failure load is almost same with the experimental load for 1.5% bamboo reinforcement but a small difference occurred for 2.5% reinforcement. The expected 1st crack load is some what similar to the experimental load. Figure 24 and Figure 25 shows that bamboo reinforcement with 1.5% had greater deflection than 2.5% reinforcement.

10. COMPARISON OF ULTIMATE LOADS FOR BAMBOO AND STEEL REINFORCED CONCRETE BEAMS

By using the compressive strength of concrete obtained from the cylinder tests, the capacity of each beam was calculated according to USD method of ACI Code by replacing bamboo with Grade 40 steel and Grade 60 steel. Table 6 shows the properties of reinforced concrete and Table 7 shows the comparison between the experimental failure loads of bamboo reinforced beams with those calculated for the equivalent reinforced concrete beam as per ACI Code.

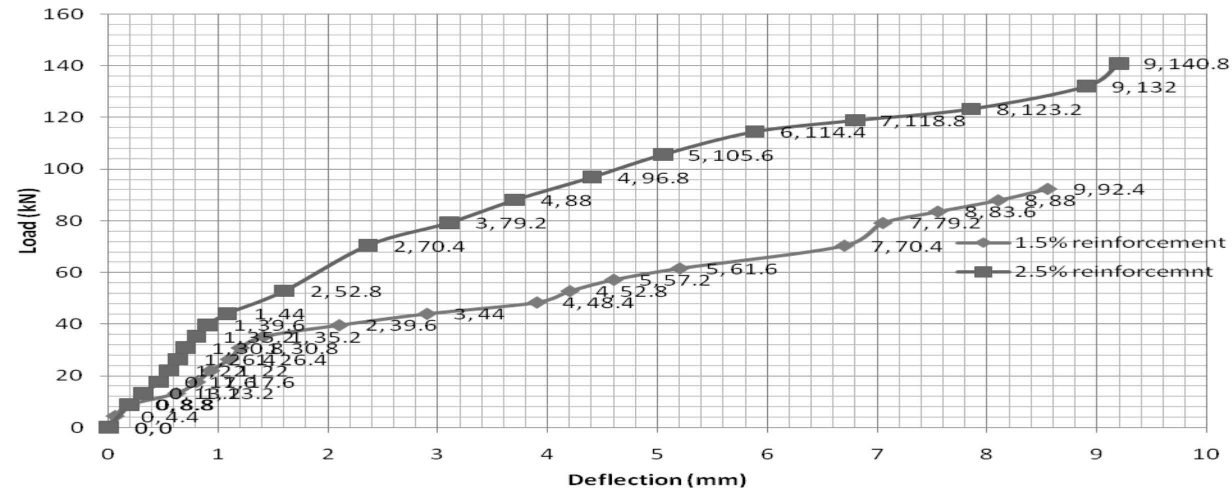

Figure 24: Load-deflection curve at L/4 distance for bamboo reinforced beam

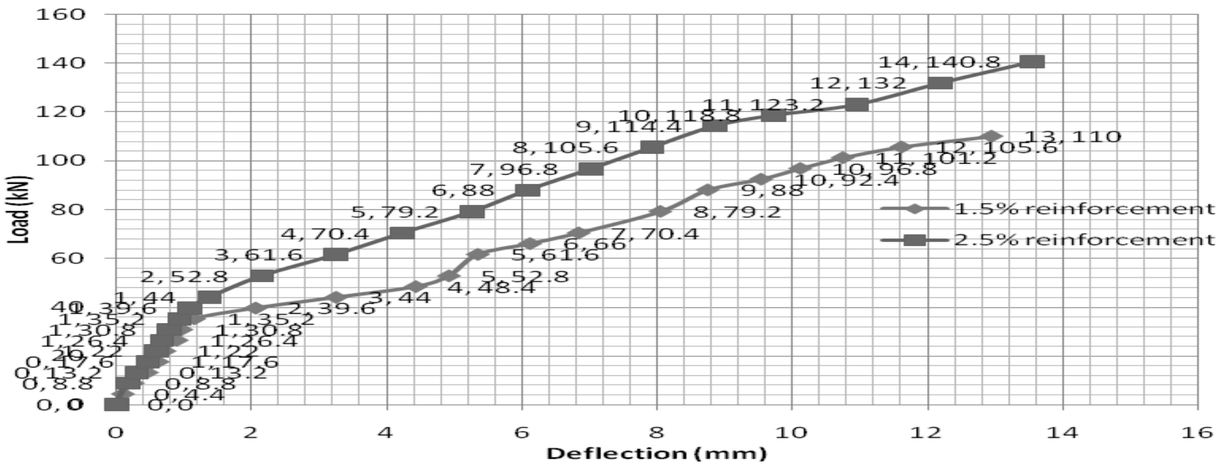

Figure 25: Load-deflection curve at L/2 distance for bamboo reinforced beam

Table 6: Property of reinforced concrete

Beam Type	Cross section (mm x mm)	Length (mm)	Steel area, A_s (mm^2)	Depth, d (mm)
1.5% Reinforcement	203 x406	2438	1239	354
2.5% Reinforcement	203 x406	2438	2065	332

Table 7: Comparison of failure loads between bamboo reinforced beam and steel reinforced beam

Percentage of reinforce-ment of concrete	Compre ssi-ve strength, f'_c (MPa)	Bamboo reinforced beam (kN)	Steel reinforced beam (40 Grade) (kN)	Steel reinforced beam (60 Grade) (kN)	Ratio of bamboo to Steel (40 Grade)	Ratio of bamboo to steel (60 Grade)
1.5% Reinforcement	27.3	110	303.7	409.7	0.36	0..27
2.5% Reinforcement	27.3	140.8	462	617	0.31	0.23

These table shows that the ultimate load capacity of bamboo reinforced concrete beam is about 36% (average) when compared with ultimate load of steel (40 Grade) reinforced concrete beam and the ultimate load capacity of bamboo reinforced concrete is about 27% (average) when compared with ultimate load of steel (60 Grade) reinforced concrete beam.

11. INVESTIGATION OF POST FAILURE

After completing the tests, all beams were broken to check the condition of bamboo reinforcement within concrete which are shown in the Figure 26.

Figure 26: Post failure condition of 1.5% bamboo reinforced concrete beam

The reinforcements were in concrete for about 50 days. The condition of bamboo reinforcements were found to be in satisfactory condition. From the figures it was seen that the bamboo reinforcements were in dry condition and no fungus growth was observed. Therefore, it can be said that the tar coating is effective to inhibit moisture penetration. In most of the cases, the bond between bamboo and concrete was found to be in good condition. However, after breaking, some local bond failure (slippage) was observed in some beams.

12. CONCLUSION

1.The flexural behaviour of bamboo reinforced beam has been found to be similar to steel reinforced beam.
2.The crack patterns of bamboo reinforced beams were found to be very similar to the corresponding reinforced concrete beams. The flexural crack initiates vertically near the mid span whereas the shear crack originates near the support at approximately forty five degree angle.
3.With the decrease of reinforcement, the deflection increases for bamboo reinforced beams.
4.The ultimate load capacity of bamboo reinforced beam has been found to be less than half of ultimate load for corresponding steel reinforced beam.
5.The calculated ultimate load has been found to be very close to the experimental failure load. The calculated initial cracking load has been found to be in close agreement to the experimental load.
6. The tar coated bamboo reinforcements were found to be in dry condition in concrete after fifty days and no fungus growth was observed. Therefore, the use of tar could be very effective as a protective coating.
7.Some slippage was observed between concrete and reinforcements but overall bonding was found satisfactory.
8.If the bamboos are soaked in water for two days, it can be easily bend for shear reinforcement.

ACKNOWLEDGEMENTS

The authors are thankful to Professor Dr. Sk. Sekender Ali of Department of Civil Engineering, Bangladesh University of Engineering and Technology for his valuable suggestions, consultations and continuous inspiration.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE OF PET BOTTLE FIBER TO ENHANCE THE MECHANICAL BEHAVIOUR OF CONCRETE

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ABSTRACT

Concrete is an indisputable material for the construction of various types of structures in the contemporary advancement of civil infrastructures. Concrete is strong in compression but weak in tension. To get rid of this problem, the introduction of fiber was brought in as a substitute to developing concrete in view of enhancing its tensile strength as well as enhancing its ductile property. Therefore, the purpose of this study was to investigate the mechanical behavior of concrete reinforced with Polyethylene Terephthalate (PET-Bottle) synthetic fibers. Polyethylene Terephthalate (PET) fibers of 40mm long, 1.5mm width and 0.6mm thickness were added to concrete in various percentages, such as- 0.0, 0.3, 0.5 and 0.75% of fiber as volume fractions. Specific gravity and unit weight of hardened concrete was measured and it was found that both were reduced insignificantly when percentages of PET fiber were increased. A total of 24 cylinder specimens (each size 6"×12") were cast to investigate compressive and splitting tensile strength. Test results after 90 days of curing demonstrated that inclusion of 0.5% PET fiber volume fraction enhanced compressive and tensile strength a maximum value of about 23% and 20% respectively. Finally, optimum dosages of PET fiber volume fractions; such as - 0.47% to attain maximum compressive strength and 0.44% to attain maximum tensile strength were found for the mix.

Keywords: *Fiber reinforced concrete (FRC), polyethylene Terephthalate fibers (PET fibers) , compressive strength test, splitting tensile test, synthetic fibers, fiber volume fraction.*

1. INTRODUCTION

Concrete and cement based materials have been implemented in structural members since prehistoric times. Day by day the implication of concrete has been developed and the limitations of concrete have been slowly but surely eliminated which increases the durability of concrete allowing a higher performance value to be achieved. However, concrete is strong in compression but weak in tension. To overcome this weakness in concrete, steel reinforcement is utilized to carry the tensile forces and prevent any cracking or by pre-stressing the concrete so it remains largely in compression under load. Therefore, the introduction of fibers was brought in as an alternative to developing concrete in view of enhancing its flexural and tensile strengths. Although the basic governing principles between conventional reinforcement and fiber systems are identical, there are several characteristic variations; such as - fibers are generally short, closely spaced and dispersed throughout a given cross section but reinforcing bars or wires are placed only where required (Kosmatka S. et al. 2002). For this reason fibers have been used to improve the toughness and ductility of concrete. It is used in industrial floors, tunneling, mining, security structures, heavy duty pavements, slab types members, runways of airport where conventional reinforcement are impractical.

Polyethylene Terephthalate (PET) is one of the most important synthetic fibers for industrial production. The largest use of PET currently is in containers. In this area, beverage and mineral water bottles are standing in prime position. Last decade, few studies were done on mechanical behavior of PET-FRC and fiber itself. Semiha A. et al. (2009) investigated PET bottle granules as a light weight aggregate in mortar and reported some advantages; such as – reduction in the death weight of a structural concrete member of a building which help to reduce the seismic risk of the building, reduction in the use of natural resources, disposal of wastes, prevention of environmental pollution and energy saving. Santos P. and Pezzin H. (2009) performed an experimental study on recycle PET (r-PET) and observed that the incorporation of r-PET fibers in Polypropylene (PP) is an efficient way to recycle PET as well as enhancing the mechanical properties of PP. Frigione M. (2010) carried out an study on r-PET as a fine aggregate and found that the r-PET concretes display similar workability characteristics and compressive strength, but splitting tensile strength slightly lower than the conventional concrete and a moderately higher ductility. Therefore, it has abundant scope to do research on PET fiber in conjunction with concrete as a discrete fiber by the various percentages of fiber volume fractions and carry out the laboratory investigation on the mechanical behavior of PET-FRC.

2. METHODOLOGY

The purpose of this study was to investigate the mechanical behaviour of concrete reinforced with PET fibers. In pursuit of this aim, the following objectives had been set:

a. To investigate the physical properties; such as-specific gravity and unit weight of the casted specimens prepared by various percentages of PET fiber volume fractions used in the mixes.
b. To investigate the compressive strength of PET-FRC.
c. To investigate the tensile strength of PET-FRC.
d. To compare the results with reference specimen.
e. To find out the optimum fiber dosage in concrete.

2.1 Types of Synthetic Fiber

- Polyethylene Terephthalate (PET) fibers
- Carbon fibers - Ceramic, Nylon, Polyester
- Polypropylene fibers – micro and macro fibers
- Acrylic
- Aramid and others.

2.2 Sources And Generations of Pet Fibers

- House hold: Carry bags bottles, containers and trash bags.
- Health and medical: Disposable syringes, glucose bottles, tubes, catheters and surgical gloves.
- Hotel and catering: Packaging items, mineral water bottle, plastic bottle and glass etc.

Figure 1: Natural PET- fiber

Figure 2: Resized PET-fiber

2.3 Significance of Using Pet Fiber in Concrete

Polyethylene Terephthalate (PET) fiber can be used in concrete for the following advantages:

a. It could increase impact and scatter resistance, fatigue endurance, durable and corrosion resistant.
b. Good insulation for cold, heat, sound saving energy and reducing noise pollution.
c. It is economical and has a longer life.
d. It is hygienic and clean.
e. Ease of processing and makes concrete light weight.
f. Provide multi directional concrete reinforcement.

2.4 Experimental Investigation

2.4.1 Materials and Mixes

The main components of the polymeric fiber used in the study were Polyethylene Terephthalate (PET) fibers (Figure 3). This fiber was prepared by cutting the used MUM mineral water bottle with designated size such as - nominal length of 40 mm(1.58 in.), average width of 1.5 mm(0.06 in.) and average thickness of 0.6mm(0.02 in.). The fiber had an aspect ratio of 90 and specific gravity of approximately 1. The average tensile strength of the fiber was 100 MPa (14.5 ksi) and tested by performing the Pullout test of briquette specimens (Figure 5).

Figure 3: PET(MUM mineral water) bottle

Figure 4: PET fibers produced from bottle

Figure 5: Pullout test of fiber

Portland composite cement confirming 28 days (ASTM C109) cube strength 5983 psi, initial setting time (ASTM C191) 126 minutes, final setting time 250 minutes was used as a binding material collected from the local market. Sylhet sand of angular and partially rounded shape having a fineness modulus of 3.18 was used as a fine aggregate. Stone chips maximum particle size of 20mm, well graded, fineness modulus of 8.38 were used as coarse aggregate. Tap water for mixing was used to cast specimens where water/cement ratio of 0.42 was used throughout the research. However no super plasticizing admixture was used in mixes. PET fibers with the fiber volume fractions of 0.0, 0.3, 0.5 and 0.75% were used where fiber containing no fiber was used as reference specimens. Mix ratio was 1:2:2.5:0.42 (Cement: Fine Aggregate: Coarse Aggregate: w/c ratio) in reference specimens. A total of 24 number cylinders specimens each size of $6' \times 12''$ were cast and then tested in the laboratory.

2.4.2 Mixing Sequence

A rotary drum mixture machine was used to get the good quality of concrete. In the mixer machine, firstly the coarse aggregate and fine aggregate were added prior to the PET fibers. These dry ingredients were mixed for about two minutes so that the fibers were evenly distributed throughout the mix. Special care was taken so as to ensure no fiber balls were formed. After that cement was added and these dry ingredients were mixed for about one minute. Water was added after one minute and was mixed for about 5 minutes so that a good mix was achieved. Concrete was then placed in the molds in three layers and a tamping rod (ASTM C 31/C 31M) of 600mm (24 in.) long and 16mm (5/8 in.) diameter was used to compact each layer. The number of rodding was 25 and falling height was 300mm (12 in.) from top surface of layer. After finishing the compaction, a trowel was used to make the top surface smooth. The molds were then kept for 24 hours under a temperature of 25^0 C to 32^0 C to set the concrete. After 24 hours the specimens were demoulded and kept in the water tank for 28 days.

Figure 6: Resized Fibers for use. Figure 7: Fibers added with aggregates

Figure 8: Concrete with PET- Fiber Figure 9: 24-Specimens after 28 days curing.

2.4.3 Compressive Strength Test

Compressive strength test procedure was carried out in accordance to ASTM C 39/C 39M. The prepared cylinders were capped so that load can transmit uniformly. Specimen to measure the compressive strength was instrumented as shown in figure 10 and then test was performed by the compression testing machine.

Figure 10: Instrumentation of cylinder specimen to test the compressive strength

A maximum crushing load (P) was measured. Compressive strength was then calculated by the equation

$$f'_c = \frac{4P}{\pi D^2} \qquad (1)$$

Where, f_c' = compressive strength (psi),
P = maximum crushing load resisted by the specimen before failure (lb),
D = diameter of the cylinder (in).

2.4.4 Splitting Tensile Strength Test

An indirect tensile test procedure was carried out in accordance to ASTM C 496/C 496M. The prepared cylinders were marked (Figure 11) after completing 90days curing and instrumented as shown in figure 12. In this test, concrete cylinder was placed with its axis horizontal in a compression testing machine.

Figure 11: Marking the cylinders in progress

Figure 12: Test set up for splitting tensile strength

The load was applied uniformly along two opposite lines on the surface of the cylinder through two plywood pads (each 13 in. long, 1 in. wide and ⅛ in. thick). The tensile strength was then calculated by the equation

$$T = \frac{2P}{\pi LD} \tag{1}$$

Where, T = maximum splitting tensile strength (psi),
L = length of cylinder (in)
D = diameter of the cylinder (in).

3. RESULTS AND DISCUSSION

3.1 Specific Gravity and Unit Weight of Hardened Concrete

Average value of specific gravity and unit weight of hardened concrete is shown in Table 1.

Table 1: Specific gravity and unit weight of hardened concrete

Percentage of PET Fibers used as volume fractions	Average Specific Gravity	Average Unit Weight (kg/m^3)
0.0%	2.424	2461
0.3%	2.415	2445
0.5%	2.395	2422
0.75%	2.342	2294

From Table1, it can be demonstrated that inclusion of PET fiber in concrete reduced both specific gravity and unit weight of hardened concrete. However, the reduction was varying within small ranges, such as - 0.35 - 3.35% for specific gravity and 0.65 - 6.75% for unit weight. As a result, addition of PET fiber made the concrete slightly lightweight compared to the specimen containing no PET fibers.

3.2 Compressive Strength Test Result

A total 12 numbers of cylinder with each size of $6'' \times 12''$, four different percentages of PET fiber volume fractions, such as - 0, 0.3, 0.5 and 0.75% were tested. Table 2 shows the average compressive strength test results and the change in compressive strength for each type of specimens.

Table 2: Compressive strength test result

Specimen Designation	% fiber volume fractions used	Average Compressive Strength (psi)	Change in Compressive Strength (%)
C_1	0.0	5300	Reference Specimen
C_2	0.3	5910	11.5
C_3	0.5	6370	20.2
C_4	0.75	4960	-6.5

Test results reveal that addition of PET fiber in concrete enhanced the compressive strength of the specimens. It was improved by at least 11% for the specimen C_2 and gradual improvement was found maximum value by at least 20% for the specimen C_3 relative to control specimen. Beyond the dosages of 0.5% PET fiber volume fractions, it was declined. Hence for the specimen C_4, compressive strength was declined to 6.5% relative to reference specimen.

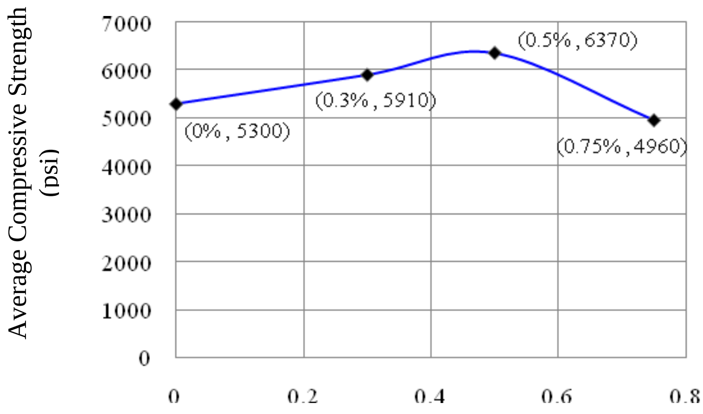

Figure 13: Compressive strength relative to the specimens of various % fiber volume fractions used

Figure 13 shows the variation of compressive strength with respect to various percentages of fiber used. It was observed that fiber enhanced the compressive strength up to the inclusion of 0.5% PET fiber volume fraction. The reduction beyond this percentage may be due to the weak bonding of fiber to concrete matrix. The fiber may not have sufficient paste volume so that it can coat itself and strengthen the fiber-matrix interaction.

Figure 14: Brittle failure of specimen C_1

Figure 15: Wedge failure were observed in specimen C_2 and C_3

Figure 16: De-bonding of fiber-matrix of specimen C_4

Failure pattern of the specimen in the figure 14 shows that concrete without PET fiber failed suddenly and combined failure was found. However the strength value is acceptable when low to moderate strength is required. Wedge type failure was observed for specimen C_2 and C_3 (Figure 15). The specimens in this case did not fully separate. On the other hand, debonding of the fiber matrix had happened due to the slip of fiber when compressive strength of C_4 specimen was tested (Figure 16).

3.3 Splitting Tensile Strength Test Result

Table 3 below shows the average of indirect tensile strength of three cylinder specimen in each case recorded during the test and the percentage change in tensile strength for all mix batches relative to the control batch.

Table 3: Splitting Tensile Strength Test Result

Specimen Designation	**% fibers volume fractions used**	**Average Tensile Strength (psi)**	**Change in Splitting Tensile Strength (%)**
T_1	0.0	546	Reference Specimen
T_2	0.3	655	20
T_3	0.5	672	23
T_4	0.75	582	7

Figure 17 below shows a graphical representation of the average indirect tensile strength for concrete containing no fibers and concrete containing different amounts of PET fibers.

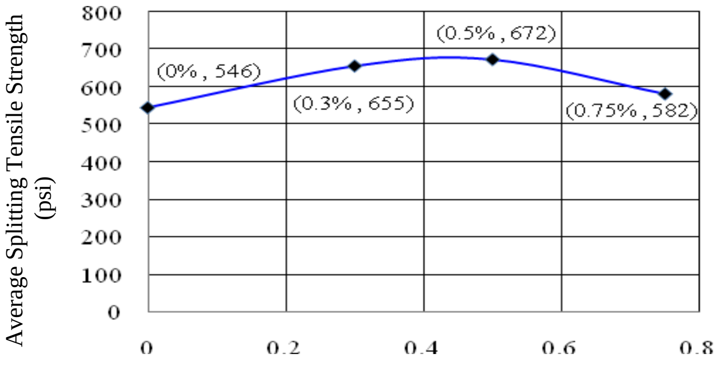

Figure 17: Variation of Splittingg Tensile Strength with different percentage of PET fiber used in concrete

Table 3 and Figure 17 show that the indirect tensile strength was increased with the addition of PET fibers. The tensile strength of the concrete for the cylinder samples T_2 and T_3 were increased by at least 20 and 23%, respectively relative to the sample T_1. The maximum tensile strength was recorded as 672 psi for the cylinder with PET fiber volume fraction of 0.5%. This increase in tensile strength was due to the fiber bridging properties in the concrete. The reinforced concrete was split apart in the tensile strength test and as a result the load was transferred into the fibers as pull out behaviour when the concrete matrix began to crack where it exceeded the pre-crack state. The control batch specimens containing no fibers failed suddenly (Figure 18) once the concrete cracked, while the PET fiber reinforced concrete specimens exhibited cracks but did not fully separate (Figure 19). This shows that the PET fiber reinforced concrete has the ability to absorb energy in the post-cracking state. However, the tensile strength of the cylinder specimen was increased for the sample T_4 7% compared to the reference specimen T_1. The reason for this downward trend for T_4 specimen may be due to the inadequate concrete's workability (fibers are known to decrease workability) for higher dosages and full compaction may not have been achieved. It can be improved by a slight increase of fine aggregate to have sufficient paste volume for coating the fibers and the addition of super plasticizer to offset the possible reduction in the slump, particularly for the mixtures with high fiber content.

Figure 18: Brittle failure of specimen T_1

Figure 19: Fiber bridging of specimens T_2, T_3 and T_4

3.4 Stress Versus Strain Relationship

The stress-strain diagrams for the four types of specimens such as 0.0%, 0.3%, 0.5%, 0.75% were shown in Figure 20 below.

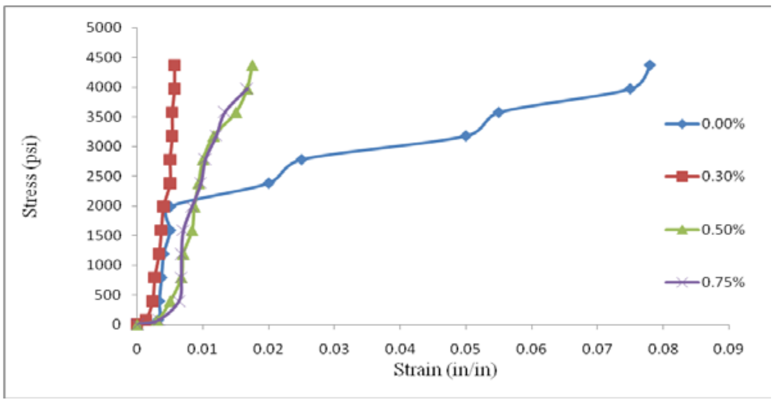

Figure 20. Stress-strain relationship

From the figure 20, it is exhibited that stress-strain diagrams for 0.3%, 0.5% and 0.75% are almost similar but diagram of specimen in which no fiber incorporated is different from that of specimens in which fibers are incorporated. It means that the mechanical properties of concrete are changed due to addition of PET-fibers with concrete. Stress-strain diagrams of fiber reinforced concrete are not exactly as like as typical stress-strain diagrams of fresh concrete. It may be due to miss setup of instrumentation and due to variation of temperature or over hardening of specimens. Tests were completed after some days of curing period. So, further investigations are required for the stress-strain diagram.

4. CONCLUSIONS

The conclusions and specific findings of the research are summarized as follows:

- Analyzing all the test results it is observed that the specific gravity and unit weight and concrete decreased as the fibers volume fractions increased. Addition of fibers with the concrete decreases the self-weight of concrete which is important for the safety of structure during natural disaster.
- From the laboratory test results it is found that the concrete compressive strength increases due to increases of fibers with the concrete. But after certain limit it is declined. When 0.5% volume fractions of PET-fibers are incorporated to the concrete, compressive strength increased by 20.2% compared to the plain concrete reference cylinders. When 0.75% volume fractions of PET-fibers are incorporated to the concrete, compressive strength decreased by 6.5% compared to the plain concrete reference cylinders.
- The addition of PET-fibers to concrete improved the tensile strength. Tensile strength of concrete increased by 20%, 23% and 7% due to addition of PET-fibers of 0.3%, 0.5%, 0.75% respectively, compared to the plain concrete specimen. These results indicate the fact that macro synthetic fiber reinforcement enhanced the tensile strength although the 0.47% fiber volume fraction is seems to be optimal. No benefits were noted when the fiber volume fraction was increased beyond 0.47%. Moreover, the control batch specimens containing no fibers failed suddenly once the concrete cracked, while the PET-fiber reinforced concrete specimens were still remain as a unique. This shows that the macro fiber (PET-Fiber) reinforced concrete has the ability to absorb energy in the post-cracking state.
- Stress-strain relationship investigated in this report shows satisfactory result. Stress- strain diagrams of fiber reinforced concrete are totally different from that of normal fresh concrete but poison's ratio investigated in this report did not satisfy from that of normal concrete.
- The empirical assumption that tensile strength of concrete is approximately one-tenth of compressive strength was verified. Hence precision of laboratory works might be agreed.

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ULTIMATE LOAD CAPACITY OF AXIALLY LOADED VERTICAL PILES FROM FULL SCALE LOAD TEST RESULTS INTERPRETATIONS-APPLIED TO 20 CASE HISTORIES

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ABSTRACT

Accurate evaluation of ultimate load capacity of pile is extremely complex and difficult. Capacity of pile is derived from the resistance developed by the soils surrounding the shaft and underneath the tip of the pile. Soil, by nature, inherits/exhibits non-homogeneity, anisotropy and in-elasticity. Prediction of accurate engineering properties of the surrounding particulate three-phase soil matrix becomes even more complex due to installation of piles. Thus, available semi-empirical/empirical methods for determination of ultimate resistance of pile pose unreliability and, thereby, may significantly affect safety and economy of a project. Therefore, although expensive, conduction of limited number of full-scale pile load tests becomes inevitable prior to commencement of construction. On most occasions, the results of this test do not show a distinct plunging ultimate load, therefore the results need interpretation to estimate pile capacity or ultimate load. Several techniques have been developed by different researchers for this purpose. In this paper, twenty pile load tests performed in different parts of Bangladesh were analyzed with six common interpretation methods and compared with the actual capacities evaluated from the plunging failure. The comparison uses Log-Normal distribution, cumulative probability as well as measured versus predicted figures to represent the precision of the methods. The result shows that the Davisson's method (1972) has the highest accuracy and also the lowest scatter as opposed to the Rebound Elastic (1956) approach.

Keywords: *Pile capacity, pile load test, measured capacity, predicted capacity, statistical analysis*

1. INTRODUCTION

Rapid urbanization has become an ongoing issue of Bangladesh during the last one-and-half decades. Tremendous pressure is being experienced by the big cities due to movement of population from rural to urban areas. Severity of housing problem is increasing everyday. Big cities are growing at the fastest rate ever happened before. Priority of vertical expansion to horizontal one is strongly realized. Government agencies strongly realize the need of tall-storied buildings and consequently new 'Building Acts' are approved. To a great extent, pile foundation has become an indispensable choice for the foundation system of tall-storied buildings.

Piles are relatively long and generally slender structural foundation members that transmit superstructure loads to deep soil layers. In geotechnical engineering, piles usually serve as foundations when soil conditions are not suitable for the use of shallow foundations. Therefore, safety and stability of pile supported structures depend on the behavior of piles. Pile behavior is significantly dependent on the properties of soils surrounding and underlying the shaft and tip/end of the pile, respectively. Accurate evaluation of the properties of these soils is quite complex. Generally, soil is non-homogeneous, inelastic and anisotropic. Again, the behavior of these soils changes significantly during the installation process, in case of driven piles, depending upon the driving method, energy required, overburden pressure, pile geometry etc. As the dissipation of excess pore water pressure generated during the driving process is significantly affected by hydro-geologic condition and time, the properties of these soils become also time and hydro-geology dependent. Consequently, determination of the exact capacity of pile becomes almost impossible.

The prediction/estimation of load carrying capacity of pile can be achieved using different methods such as pile load test, dynamic analysis and static analysis based on soil properties from field and laboratory tests, and static analysis utilizing the results of in situ tests such as cone penetration. Due to the uncertainties associated with pile design, available semi-empirical/empirical methods for determination of ultimate resistance of pile may pose un-reliability and, thereby, may significantly affect safety and economy of a project. Therefore, although expensive, conduction of limited number of full-scale pile load tests becomes inevitable prior to commencement of construction. On most occasions, the results of this test do not show a distinct plunging (Figure 1B) ultimate load, therefore the results need interpretation to estimate pile capacity or ultimate load. Several techniques (Terzaghi 1942, Housel 1956, Hansen 1963, De Beer 1968, Fullar and Hoy 1970, Chin 1970, Davisson 1972, Mazurkiewicz 1972, Butler and Hoy 1977, Chin Fung Kee 1977, Brierley 1979, Decourt 1999) have been proposed in the geotechnical literature. Since uncertainty is not only existing in the pile design process, but also in the procedure of confirmation and verification for pile capacity based on static load testing, the study on the static load testing and interpretation methods draw special attentions of researchers and engineers back to decades ago and the trend continues.

In this paper, twenty pile load tests performed in different parts of Bangladesh were analyzed with six common interpretation methods (Double Tangent Method on an Arithmetic Plot, Maximum Curvature method, The Hansen 80% Criterion, Davisson's method, Rebound Elastic method, and Decourt Extrapolation method) and compared with the actual capacities evaluated from the plunging (Figure 1A) failure (Shariatmadari et. al 2008). This study aims at bridging this gap between prediction and reality. The research effort is focused on the applicability of the most suitable interpretation methods for predicting the axial load carrying capacity of piles into the geo-hydrologic, cohesive and granulometric matrix of dominantly prevailing alluvial deposits of the Bengal basin.

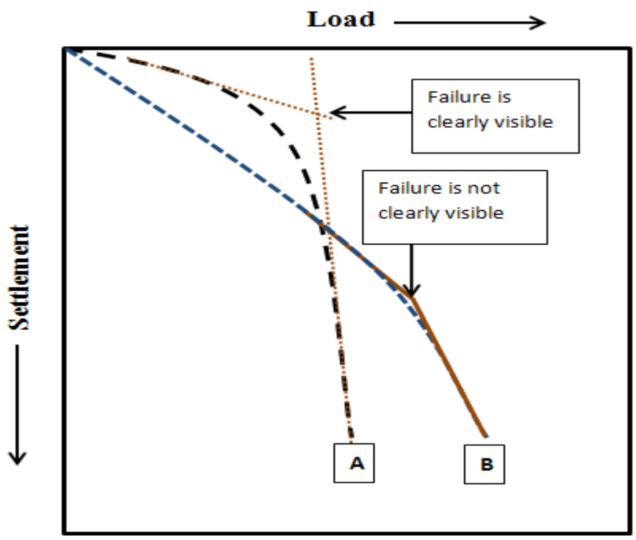

Figure 1: (A) Ideal plunging, (B) graph with unclear failure point

2. PILE LOAD TEST DATABASE

A database has been compiled including 20 pile case histories where 17 were driven piles and rest 3 were bored piles. These case histories were collected from 10 different sites. For each load test, emphasis was placed on the completeness of the required information which includes: test pile size (length and x-section/diameter), complete records of the load-deflection data and availability of the subsurface exploration data for the site. Pile load tests with missing information were discarded. The length of the driven piles varies from 8.24m to 27.45m and length of the bored piles varies from 13.7m to 15.25m. All driven piles used in this research were square with x-section varying from 254mm x 254mm to 355mm x 355mm. On the other hand, all three bored piles were circular with diameter of 600mm.

All the tests were conducted according to ASTM D 1143 test procedure. The main characteristics of these pile load tests were reaching to ultimate point of bearing capacity by plunging concept. The results of load tests are summarized in Table 1. Due to the space limitations the load settlement curves are not included in this paper.

Table 1: Summary of database records

Sl. No.	Test Pile No.	Pile Dimension		Total Settlement	Net Settlement
		Length M	X-Sectional Dimension mm	mm	mm
1	TP 1	18.3	300 x 300	29.84	21.4
2	TP 2	12.2	300 x 300	31.43	15.23
3	TP 3	13.7	300 x 300	31.9	18.3
4	TP 4	18.3	300 x 300	31.3	19.46
5	TP 5	12.2	300 x 300	22.37	15.07
6	TP 6	13.7	300 x 300	30.18	15.6
7	TP 7	9.15	254 x 254	55.57	48.95
8	TP 8	15.25	355 x 355	19.99	9.95
9	TP 9	8.24	355 x 355	42.14	23.68
10	TP 10$^{\psi}$	14.65	600*	129.09	128.41
11	TP 11	9.15	254 x 254	56.84	50.36
12	TP 12	15.25	355 x 355	52.49	43.03
13	TP 13	12.2	300 x 300	64.17	57.14
14	TP 14	9.15	254 x 254	56.35	51.10
15	TP 15	15.25	355 x 355	28.91	17.47
16	TP 16	27.45	355 x 355	76.73	72.55
17	TP17	13.7	300 x 300	47.97	40.77
18	TP 18$^{\psi}$	13.7	600*	71.68	67.33
19	TP 19	13.7	300 x 300	53.77	48.69
20	TP 20$^{\psi}$	15.25	600*	74.78	62.6

ψ Bored pile, * Circular cross-section

3. INTERPRETATION APPROACHES

Without a proper definition, interpretation of ultimate load from load-test result becomes a meaningless scheme. To be useful, a definition of failure load must be based on some rules. It is difficult to make a cogent choice of the best capacity criterion to use, because the preferred criterion depends heavily on one's past experience and conception which establish the ultimate resistance of a pile. To reach ultimate bearing capacity, large deformation is needed. Because of this reason some failure criteria is based on a specified settlement. In other hand some designer preferred that to achieve allowable capacity of piles, use the ultimate bearing capacity and a safety factor. The interpretation methods for ultimate capacity of piles based on axial static load testing data stipulated in international codes can be grouped to three types:
1) Based on the load grade generating load-settlement (P-S) curve;
2) Based on the settlement under certain load; and
3) Based on the total settlement of the pile head.

Until now, several interpretation approaches (Terzaghi 1942, Housel 1956, Hansen 1963, De Beer 1968, Fullar and Hoy 1970, Chin 1970, Davisson 1972, Mazurkiewicz 1972, Butler and Hoy 1977, Chin Fung Kee 1977, Brierley 1979, Decourt 1999) have been established to estimate the ultimate capacity of piles. Among them six common interpretation method have been selected for this research. Table 2 summarizes these interpretation methods. Among them, Davisson Offset Limit is recommended in all codes listed and is the most widely used in engineering practice. This method is based on the assumption that capacity is reached at a certain small toe movement by compensating for the stiffness (length and diameter) of the pile. The Hansen's 80% criterion is also widely used. The load that gives four times the movement of the pile head as obtained for 80% of that load. The Decourt extrapolation load limit is equal to the ratio between the y-intercept and the slope of the line. The Decourt method has the advantage that a plot prepared while the static loading test is in progress will allow the user to eyeball the projected capacity.

Table 2: Summary of interpretation methods for ultimate capacity of piles

Sl. No.	**Interpretation Methods**	**Formula**	**Remarks**
01	Double Tangent Method on an Arithmetic Plot	The intersection of two straight line is referred to as the failure load	
02	Maximum Curvature Method	The point of maximum curvature is considered as failure load	
03	The Hansen 80% Criterion (1963)	In load settlement curve, define $\Delta_P = \frac{1}{4}\Delta_{Qult_t}$ $P = 80\% Q_{ult}$	Δ_Q, Settlement at interpreted ultimate bearing capacity; Δ_P ,Corresponding settlement at load P; 80% of interpreted ultimate bearing capacity Q_{ult}
04	Davisson's Method (1972)	$\Delta = \frac{PL}{AE} + \frac{D}{120} + 3.81$ Q_{ult}= intersection of the above equation with the load-settlement curve	A, Pile cross sectional area; E, Elastic modulus of pile; L, Pile length; D, Pile diameter in mm
05	Rebound Elastic Method (1956)	Rebound after the application of full test load	
06	Decourt Extrapolation (1999)	$Q_{ult} = \frac{C_2}{C_1}$ $Q = \frac{\delta C_2}{1-\delta C_1}$	C_1, Slope of the straight line C_2, y-intercept of the straight line δ, Movement Q, Applied load

4. STATISTICAL ANALYSIS

Graphical and probability approaches are engaged to verify the pile load test interpretation approaches. Cumulative probability and Log-Normal methods have been considered to compare different methods of pile capacity determination. According to cumulative probability approach the ratio of predicted value (Q_p) and measured value (Q_m) has been drawn versus cumulative probability. For a series of numerals, Q_p/Q_m has been set ascending and indexed with 1 to n, then for each of the relative amounts, the cumulative probability factor has been calculated as follows:

$$P(\%) = \frac{i}{n+1} * 100 \tag{1}$$

Where P is the cumulative probability factor, (i) is the index of considered case and n is the number of total cases. For determining the convergence or deviation tendency of the output of prediction, the following equations have been referred.

$$E_{ave} = \left(\frac{Q_P}{Q_m}\right)_{\%50} - 1 \tag{2}$$

The value of Q_p/Q_m at the cumulative probability of 50% stands as the average of this ratio, so the applied approach seems to be more precise if this average reaches to 1.

The slope of passing line from points in each approach denotes the amount of deviation; so the results from applied approach would have less deviation and more reliability if that line has low gradient. The result of cumulative probability analysis is shown in Figure 2. The average estimated errors of these 6 methods are summarized in the Table 3. The results of comparison showed that Davisson's method predict closer values for bearing capacity to actual values of cases among other methods. The error of this method is about 0 while this is 12% for Double Tangent Method on an Arithmetic Plot, 8% for Maximum Curvature Method, 4% for Hansen 80% criterion Method, 27% for Rebound Elastic Method, and 5% for Decourt Extrapolation Method.

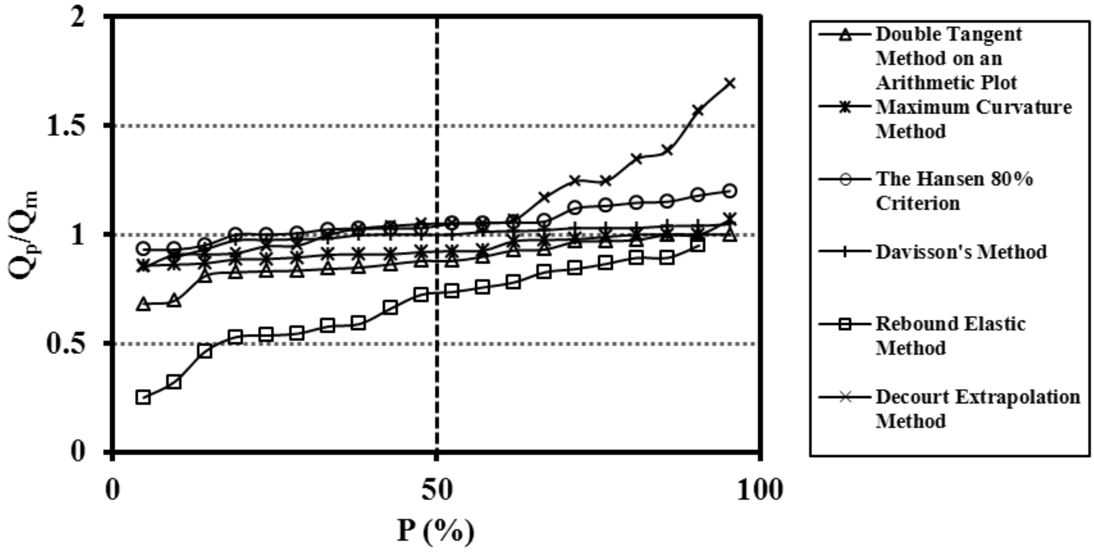

Figure 2: Cumulative probability of six pile load test interpretation methods

Table 3: Average estimated error for six common interpretation methods

Sl. No	Interpretation method	Average Error (%)	Description
01	Double Tangent Method on an Arithmetic Plot	12.1	Underestimate
02	Maximum Curvature Method	7.79	Underestimate
03	The Hansen 80% Criterion	4	Overestimate
04	Davisson's Method	0	Ok
05	Rebound Elastic Method	27	Underestimate
06	Decourt Extrapolation Method	5	Overestimate

The Log-Normal distribution can be employed to evaluate the performance of pile capacity prediction method. The Log-Normal distribution is acceptable to represent the ratio of Q_p/Q_m; however, it is not symmetric around the mean, which means that the Log-Normal distribution doesn't give an equal weight of under prediction and over prediction. In order to use Log Normal distribution, the mean ($\mu_{\ln}$) and standard deviation ($\sigma_{\ln}$) are evaluated for natural logarithm of Q_p/Q_m as follows:

$$\mu_{\ln}\left(\frac{Q_p}{Q_m}\right) = \frac{1}{n}\sum_{i=1}^{n}\ln\left(\frac{Q_p}{Q_m}\right) \tag{3}$$

$$\sigma_{\ln}\left(\frac{Q_p}{Q_m}\right) = \sqrt{\frac{1}{n-1}\sum_{i-1}^{n}\left(\ln\left(\frac{Q_P}{Q_m}\right)-\mu_{\ln}\right)^2} \tag{4}$$

The ratio Q_p/Q_m and the natural logarithm of the ratio $\ln(Q_p/Q_m)$ for each pile were calculated. Then, the mean ($\mu_{\ln}$) and standard deviation ($\sigma_{\ln}$), and coefficient of variation (COV) of $\ln(Q_p/Q_m)$ for each method determined. The Log-Normal distribution is defined as the distribution with the following density:

$$f(x) = \frac{1}{\sqrt{2\Pi} * \sigma_{\ln} * X} Exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\mu_{\ln}}{\sigma_{\ln}}\right)^2\right) \tag{5}$$

Where x= (Q_p/Q_m), μ_{ln} is the mean of ln(Q_p/Q_m) and "σ_{ln}" is the standard deviation of ln(Q_p/Q_m). The Log Normal distribution was used to evaluate the different methods based on their prediction accuracy and precision. Figure 3 shows the Log Normal distribution for different methods considered in this paper that confirms the results of cumulative probability analysis.

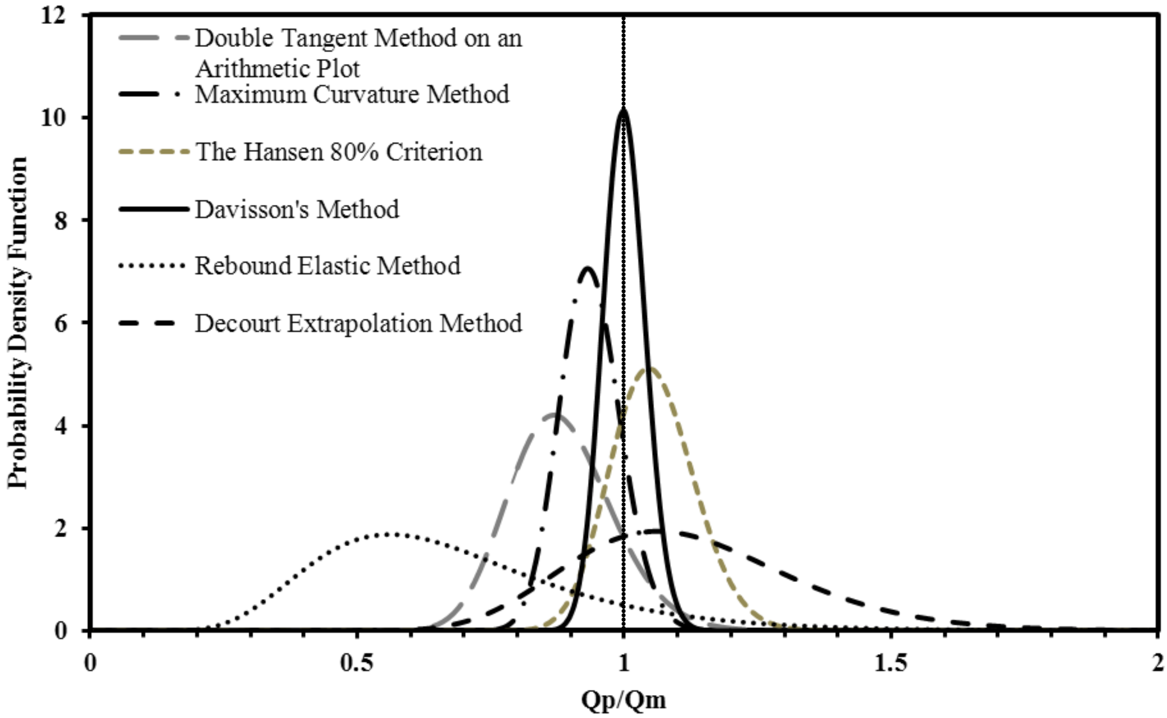

Figure 3: Log-Normal distribution of six pile load test interpretation methods

Based on the results of Log-Normal distribution analysis the probability of accurate capacity predictions using these methods can be estimated. It could be seen that, in Figure 3, Rebound Elastic method has the lowest precision and highest scatter.

Table 4 shows summary of all mean, standard deviation, coefficient of variation (COV), %Δ(Max-Min), %Δ(Min-Max), Skewness, and Kurtosis for different interpretation approaches. The results from graphical method, measured vs. predicted values are presented in Figure 4. The results clearly indicate the high precision of the Davisson's method and low precision of Elastic Rebound method.

Table 4: Summary of statistics for different interpretation approaches

Sl. No.	Interpretation Method	Mean	Standard Deviation	COV	%Δ(Max-Mix)	%Δ(Min-Max)	Skewness	Kurtosis
01	Double Tangent Method on an Arithmetic Plot	0.88	0.09	0.10	31.81	46.7	-0.64	0.15
02	Maximum Curvature Method	0.94	0.06	0.06	19.67	24.5	0.52	-0.45
03	The Hansen 80% Criterion	1.05	0.08	0.07	22.43	28.9	0.27	-0.67
04	Davisson's Method	1.00	0.04	0.04	14.62	17.1	-1.28	1.96
05	Rebound Elastic Method	0.67	0.20	0.29	73.56	278	-0.57	-0.40
06	Decourt Extrapolation Method	1.12	0.23	0.20	49.95	99.8	1.21	0.90

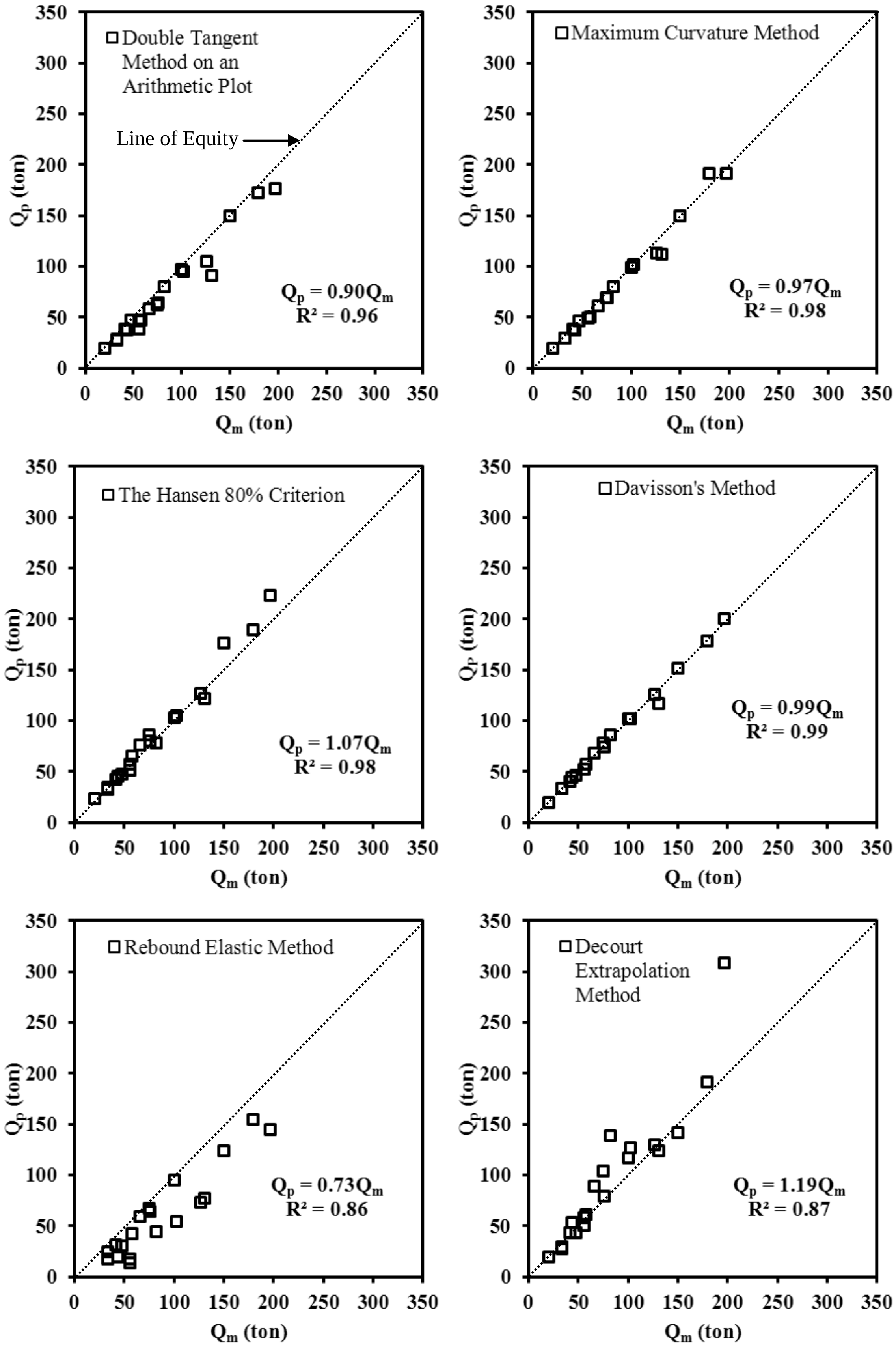

Figure 4: Scatter plots of measured capacity versus predicted capacity graphs for six common pile load test interpretation methods

5. CONCLUSIONS

It is difficult to make rational choice of the best capacity criterion to use, because the preferred criterion depends heavily on one's past experience and conception of what constitute the ultimate resistance of a pile. Actually, the interpretation of failure value from test loading is subjected to some confusion, which is understandable because load test do not provide answers only data to interpret.

Using data bank consisting of 20 case histories of full-scale pile load tests, comparison among six interpretation methods were performed. The error investigation has been made using cumulative probability, Log-Normal distribution, and measured versus predicted graph approaches.

The analyses showed that

- Rebound Elastic method, Double Tangent method on an Arithmetic Plot, and maximum Curvature method underpredict the ultimate capacity of pile. However, the scatter is low for Maximum Curvature method and high for Rebound Elastic method.
- Decourt Extrapolation method overpredicts the ultimate capacity of pile and also shows high scatter.
- Hansen's 80% criteron slightly overpredicts the ultimate capacity of pile. This method has high precision and low scatter.
- The results of comparison demonstrate that the error of Davisson's method is in acceptable range. This method could be used as pile load test interpretation.

ACKNOWLEDGEMENTS

This research endeavour was financially supported by the Housing and Building Research Institute (HBRI) under fellowship scheme. 1st author is thankful to the financial aid provided by the HBRI. The author is also grateful to Icon Engineering Services who helped in providing pile load test data.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

IMPLEMENTATION OF THE BUILDING CONSTRUCTION RULES AND REGULATIONS IN BANGLADESH : THE CASE OF DHAKA CITY AREA

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ABSTRACT

Dhaka is plagued with multifarious problems as shortage of buildable land, diminishing open space and playgrounds, acute shortage of urban facilities, traffic congestion, filling of natural water bodies, rivers, water logging and drainage congestion, environmental degradation, crisis of drinking water etc. RAJUK enacted Dhaka Metropolitan Building (Construction, Development, Preservation and Removal) Rules, 2008 as per the provision of Town Improvement Act, 1953 and Building Construction Act, 1952. The development control function of RAJUK is operated under those rules and regulations. The existing regulations introduced Floor Area Ration (FAR) index to allow permissible building coverage. It is a challenge for the development organizations to impose FAR policies in surrendering proportional land for green space. A sample of the newly constructed buildings were surveyed and it was found that about 52% of the buildings are deviated from the approved plans. As per the rule it is suggested to prepare structural design by the IEB registered structural engineer but the study shows that about 56% of the building owners did not prepare structural design from the qualified structural engineer. This study reveals the fact behind the implementation of the Building Construction Rules and Regulations specially in case of Dhaka City Area.

1. INTRODUCTION

Dhaka is the nucleous of all categorized development activities in Bangladesh. It is going to be transformed into one of the populous mega cities in the world. Growing at a very fast rate, Dhaka's urban Population is Predicted to increase from 11.3 million to about 21 million by 2015 (World Bank, 2007). Building construction in Bangladesh specially in Dhaka was brought under regulatory framework after enactment of East Bengal Building construction Act, 1952 which may be termed as 'root law'. Different subsequent rules and regulations locally named as 'bidhimala' were enacted for regulating building construction and development control activity. The Rules started in 1955 with only 5 lines of instruction on the backside of the application form. Gradually newer requirements were set- 1 page in 1970, 3 pages in 1984, and 12 pages in 1996 (Rahman, 2008). Non-professional bureaucrats mostly framed these Rules, viewing a building only within its plot boundaries, using no tool other than setbacks. Later, height limits based on road width, and few other requirements were introduced. The arbitrary requirements were not thoroughly researched in relation to developing a desired physical and climatic environment.

The subsequence rules are Building Construction Rules 1996, 'Dhaka Mohanagar Imarat (Nirman, Unnayan, Sangrakhan, Oposharan) Bidhimala 2007' which was later revised as 'Dhaka Mohanagar Imarat (Nirman, Unnayan, Sangrakhan, Oposharan Bidhimala 2008' and now in the process of revised 'Dhaka Mohanagar Imarat (Nirman, Unnayan, Sangrakhan, Oposharan Bidhimala 2012' supposed to be gezetted soon. The currently parctised rules introduced Floor Area Ratio (FAR) which is a promotional tool of planned urbanization. FAR is a index of considering built up area as per plot size and width of adjacent roads. FAR was introduced in Dhaka to achieve the objectives of land accumulation, increased green space, vertical expension, ground water recharging, environment friendly built environment.

Besides, in 1993 the Bangladesh National Building Code (BNBC) was enforced to regulate the technical details of building construction and to maintain the standard of building construction with objectives of providing safe and healthy habitat by regulating all activities related to buildings such as planning, design and construction and guidance for a uniform start of practice in planning, design, construction aspects as well as service facilities such as electrical, mechanical, sanitary and other services (Shafi, 2010). Before approval of building plans landuse clearances are issued by the Town planning department following the guidelines of the master plans known as Dhaka Metropolitan Development Plan (DMDP), 1995 which comprises of Structure plan, Urban Area Plan and Detailed Area Plan (DAP). In issuing landuse clearance and plan approval there is legislative provision of following norms and regulations of other concurrent rules such as Private Residential Land Development Rules 2004, Wet Land Conservation Act 2002, Strategic Transport plan (STP), master plan, 1959 and so on. Although planning is a continuous process and it should be followed the latest version of the master plan, due to mandatory obligation it should be followed the prevailing regulations of utility and other organizations such as rules and regulations of Directorate of Environment (DOE), Dhaka Water Supply and sewerage Authority (DWASA), Dhaka Electric Supply Authority, Fire Service and Civil Defense Authority, Civil Aviation Authority of Bangladesh (CAAB), Dhaka City Corporation (DCC), Dhaka Transport Coordination Board (DTCB), Dhaka Metropolitan Police, Titas Gas Co. Ltd, Roads and Highways Department and so on. Building Plans in the cantonment areas are approved by the Dhaka Cantonment (Building Construction) Bi-Laws, 1994 made under sub section 186 of Cantonment Act 1924 (II of 1924). In case of plan approval of buildings located surrounding the enlisted Key Points Installation (KPIs), necessary clearances should be issued by the Key Points Installation Defense Committee (KPDC) formed under Ministry of Defense, Peoples Republic of Bangladesh. In case of plan approval of buildings located surrounding enlisted heritage structures, special project permission should be taken from the special committee formed in RAJUK. Real estate sector is expending very fast contributing to 12-15% in national GDP. In 1991 Real Estate and Housing association of Bangladesh was formed with only 11 members. Now REHAB member is 1081. Now in 2011 the total number of real estate companies is more than 1500 (Seraj, 2011). Bringing the activities of private developers under regulatory framework Government enacted 'Real Estate Development And Management Act. 2010. Under this Act registration of real estate developers have been initiated in RAJUK.

Rajdhani Unnayan Kartripakkha (RAJUK) is the statutory and legitimate authority constituted by the Town Improvement Act, 1953 for planning, development and development control activity of greater Dhaka, Narayanganj and surrounding areas (TIA Act, 1953). RAJUK is an autonomous institute headed by its Chairman and five members who are the full time officials deputed by the Government who are responsible for regulating five departments as Land and Administration, Planning, Development, Estate and finance. Planning Department is entrusted with planning permission devided into three sections as Town Planning Department headed by Director (Town Planning), Director (Plan Preparation), Director (Development Control. It was found that more than 90% buildings of Dhaka city violated the rules of BCR 1996 (Mahmud, 2010). This study reveals the drawbacks of implementation of the Building Construction Rules and Regulations for sustainable urban development.

2. OBJECTIVES

- To present process of plan approval following the building construction rules and regulations imposed by the Government from time to time
- To present and analyze the drawbacks, constraints limitations in respect to technical, institutional, planning and engineering aspects in implementing the rules and regulations
- To suggest some policy guidelines in implementation of the building construction rules and regulations.

3. METHODOLOGY

The study is based on primary data collection and secondary materials collected from different sources A sample of the newly constructed buildings namely 100 building owners/representatives were interviewed for primary data collection regarding process of plan approval and construction following the building construction rules and regulations imposed by the Government from time to time . Different acts, rules and regulations were studied for enlightening the issues.

4. HISTORICAL EVOLUTIOIN OF PLANNING JURISDICTION

The first formal plan for Dhaka was prepared in 1917 by Sir Patrick Geddes, a renowned British Town Planner and proponent of what has become known as the Garden City Concept - a concept most evident in the romantic street patterns and gardens of the Ramna area. The Dhaka Master Plan, 1959 was prepared in 1959 and was approved in 1960. It covered an area of about 220 sq. miles with a population slightly exceeding one million encompassing Dhaka Municipality, Tongi Municipality and Narayanganj Municipality. The Dhaka Master Plan provided for major expansion areas at Mirpur, Tongi and Gulshan/Banani/Badda and proposed large scale reclamation at Keraniganj, Postagola and part of the DND Triangle. After Liberation in 1971 the annual population growth rate increased to 10%. A strategic plan for Dhaka was prepared in 1981 under the auspices of the Dhaka Metropolitan Area Integrated Urban Development Plan (DMA-IUDP). The DMDP Urban Area Plan (UAP) provides an interim mid-term strategy for the development of existing urban area within the RAJUK administrative boundary. The area covered by the UAP comprises DCC, Narayanganj, Jinjira, Uttara and eastern fringe, while additionally the outlying areas as Tongi, Gazipur, Savar and Dhamrai/Dhamsona are put under consideration. STP considered three Land Use Scenarios, developed within the context of the updated Structure Plan which include a strong Central Spine Scenario in which strong north-south axial characteristics of Dhaka were recognized, the Growth Pole Scenario was one that could profitably be applied to planning and investment decisions at the national level. The STP study area has a total area of 7,440 square kilometres and a population (2001) of 17.3 million of the six Districts surrounding and including Dhaka City (Dhaka, Gazipur, Manikganj, Munshiganj, Narayanganj and Narsinghdi) is expected to reach 36 million by 2024. Following the guidelines of Structure Plan and Urban Area Plan RAJUK has prepared Detailed Area Plan recently and published as a gazette in June 22, 2010. Designated landuse category of Detailed Area Plan has been shown in the following

Table 1: Designated Land Use Category in DAP

LANDUSE	AREA	PERCENTAGE
Administrative Zone	30611686.017	0.20
Agricultural Zone	3546423813.051	23.05
Commercial Zone (Business)	39163871.488	0.25
Commercial Zone (Office)	293357.212	0.00
Flood Flow Zone	3247995914.307	21.11
General Industrial Zone	172751486.097	1.12
Heavy Industrial Zone	86967970.733	0.57
Institutional Zone	254633020.170	1.66
Mixed Use (Residential-Commercial)	74525.459	0.00
Mixed Use Zone (Commercial-General Industrial)	12196058.657	0.08
Mixed Use Zone (Residential-Commercial)	351390073.049	2.28
Mixed Use Zone (Residential-Commercial-General Ind	158761290.125	1.03
Mixed Use Zone (Residential-General Industrial)	31438452.964	0.20
Non-Conforming Use	83084086.810	0.54
Open Space	142667979.055	0.93
Overlay Zone	650936111.978	4.23
Proposed Road Network	700396457.469	4.55
Rural Settlement Zone	1565397024.723	10.18
Transportation & Communication	260317586.311	1.69
Transportation Facilities	7147526.502	0.05
Urban Residential Zone	2898676468.280	18.84
Water Retention Area	240466368.253	1.56
Waterbody	901249993.202	5.86
Total		99.98

Source: DAP, RAJUK

5. PROCESS OF PLANNING PERMISSION FOR DHAKA CITY

Before constructing any building a plot owner needs to apply for a land use clearance to check its conformity with land use proposals of the master plans. If the applied land use confirms the master plan’s proposal, the plot owners gets the no objection Certification (NOC) of land use clearance and apply for the approval of the building plans for detail architectural drawing of the site and building. RAJUK is the legitimate public institution issuing approval of building plans within its jurisdiction. Before any sort of construction a land owner need to take the ‘land use clearance’ and ‘approved building plan’ from RAJUK.

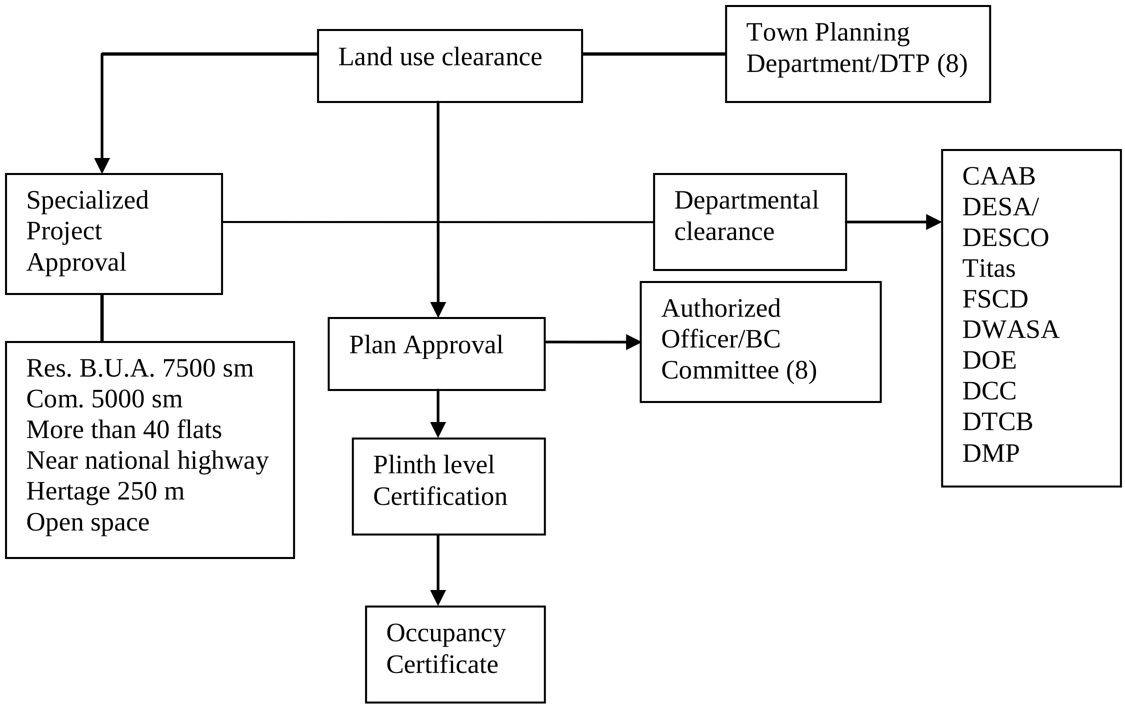


Figure1: Plan Approval Process of RAJUK

RAJUK exercises development control function as per provisions laid down in the East Bengal Building Construction Act,1952 and its subsequent Amendments and the 'Dhaka Mohanagar Imarat (Nirman, Unnayan, Sangrakkhan and Oposaron) Bidhimala, 2008' framed there under. Every construction/erection/excavation within the jurisdiction of RAJUK requires permission/approval from the Atuhorized Officer or Building Construction Committee appointed under the provision of the Town Improvement Act, 1953. Any type of building construction- housing, commercial, industrial or whatever needs to get its plan approved and they must be in conformity with the land use provision of Master Plan/Urban Area Plan/Detailed Area Plans. 'Building' as defined in the existing Building Construction Rules includes a home, hut, wall and any other structure where masonry-bricks, corrugated iron sheets, metals, wood, bamboo, mud, leaves, grass, thatch or any other material-is used. The Rules are updated, time-to- time, as and when required. At present, Eight Building Construction Committees are in force in which the Authorized Officers have been working as member-secretary of the committees. In the financial year 2010-2011 about 4796 applications submitted to the BC committee for approval and about 4171 were approved. About 204 unauthorized structures were demolished and mobile court were conducted in 76 unauthorized/deviated structures.

If the submitted plans are rejected by BC committee, he ca appeal to the ‚Appelate Sub Committee' which is chaired by Chairman, Rajuk. If Appelate Sub Committee' denies to approve the plan the aggreived can appeal to the Urban Development Committee (UDC) which is the supreme authority formed to guide city's development, provide visionary directives, and dispense appeals.. According to the Rules, UDC was supposed to be chaired by the Secretary of Housing and Public Works.

6. REASONS OF DELAY IN PLAN APPROVAL

From a survey on plan permission it was found that people seek broker's assistance to avoid unnecessary delay 53%, to avoid harassment 22%, to avoid spending time running after it 7%, to avoid the complex process of plan approval 13% and for others reasons 5% people deploy 'broker' locally named as 'dalal' in the planning permission (Mahmud, 2008). But the real fact is that such situation may happen due to a lot of reasons mentioned below:

- Most of the land owners are not well informed and awared about the rules and regulations of plan approval and institutional process, as a result they seek assistance from the brokers.

- There is a tendency among land owners and developers not paying fees to the qualified professional architects and structural engineers and saving money. They search for low paying draftsmant for designing and drawing of RAJUK plan locally named as ´DIT Sheet'. This is a reason of submitting faulty designs and the approval process is delayed.
- Plan approval ensures some sort of legitimate ownership of land and estate. In many cases the clients of plan approval submits faulty legal documents like deeds, mutation records and failed to submit updated tax receipts and other legal ducuments. They informed that they will submit them later, as a result the process of plan approval is delayed.
- False signatory practices of some professionals who are not involved with the work

7. DEVIATION IN BUILDING CONSTRUCTION

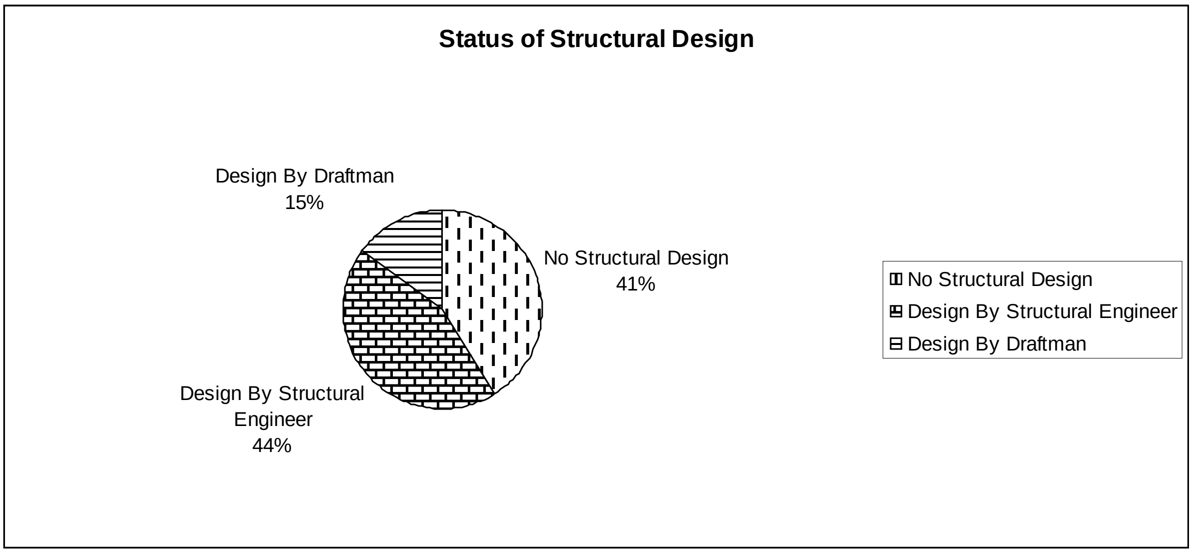
Severe problems of plan violation has found in the construction phase of residential building in Dhaka city. Land owner take the 'building approval' from RAJUK by submitting as per the requirements of the Rules. But they start to deviate from the approved plan from RAJUK while constructing the building. Condition would not be so acute if plot owners maintained the plan and rules they promised in the approved plan. The study reveals that about 52% of the buildings are deviated from the approved plans while 48% buildings were constructed as per approved plans. There are some categories of deviation mentioned in the following table 1.

Table2: Types Of Deviation And Recommended Measures

	Types of Deviation	Narrative	Recommended
1	Use Deviation	Permit as parking, open space but used as flats, shops, dispensaries	Strong imposition and eviction by the authority
2	Height Deviation	They extended floors beyond the approved plans	Demolition of the extended portion
3	Structural Deviation	They deviated columns, beams, cantilevers, balcony	Building construction with the help of skilled engineers
4	Spatial Deviation	The built structure deviates spatially from the permitted plans	Building construction with the help of skilled engineers

8. STATUS OF IMPOSING STRUCTURAL DESIGN

In Dhaka city, Structures are designed and constructed without abiding to specific building regulations (Seraj, 2010). The study reveals that about 56% of the building owners did not prepare structuctural design from qualified structural engineers. About 44% made structural design from enlisted and experienced structural engineers.



Source: Field Survey, 2011

Figure2: Status of Preparation of Structural Design

9. CONSTRUCTION SAFETY

As per BNBC in construction safety should be ensured. Recently some fatal accidents in the construction sites signifies very poor conditions regarding safety measures. The study reveals the status of safety measures presented in the following table:

Table3: Status of safety measures in construction sites

SL	Safety Measures	Yes (%)	No (%)
1	First Aid Box/Attendent in Place	15	85
2	Security Net surrounding the site	32	68
3	Shore piling/Retaining wall in place	61	39
4	Safety for utility connections	42	58
5	Use of Helmates, Gloves, safety belt	11	89
6	Safety signage in place	21	79
7	Net places in set back space	34	66
8	Ladder in right place	68	32
9	Scaffolding rightly in place	90	10
10	Crane/Lift rightly placed	82	18
11	Construction equipment rightly deposited	42	58
12	Dismantling/Demolition work done rightly	58	42

Source: Field Survey, 2011

10. RECOMMENDATIONS

- Preparation of structural design should comulsorily imposed in each and every new building contruction through compulsory imposition and legislative revision.
- Awareness generation among the citizens, developers, land owners are necessary on behalf of the positive aspects of the rules and regulations
- Professional associations like Institutes of Architects Bangladesh (IAB), Institutes of Engineers Bangladesh (IEB), Bangladesh Institutes Planners (BIP) should play key role in securing the interests of the professionals so that false signatory practices may be prohibited.
- Proper safety measures should be taken in construction sites and those should be monitored regularly to avoid any unexpected situation.
- Institutional capacity of RAJUK should be strengthened employing more engineers, architects and town planners in order to cater to specialized services.
- Decentralize and expedite approval process and introduce 1-stop service, simplify the rules, both the language and details, synchronize the provisions of BNBC, BCR, DAP, etc, eliminate discrepancies among BCR, Fire Code, Environment Law
- Revise both the FAR and Ground Cover indices, in most cases downward, strengthen and regularize Urban Development Committee (UDC), enforce and observe all provisions (e.g. supervision, occupancy) fully, make regular inspection and monitoring mandatory; consider outsourcing
- Updating of the Bangladesh National Building Code 1993 is needed to bring it at par with the present state-of-the-art paying due consideration to local needs and practices.
- In order to increase buildable land city regional master plan is necessary to be formulated and implement satellite town projects and Mass Rapid Transit (MRT) for enhancing commuting facilities.
- Stakeholders participation is necessary in formulation and adoptation building construction rules and regulations in future

11. CONCLUSION

Policies which are not feasible, realistc and pragmatic, cannot be imposed and it poses a great challenges to be imposed. In a country like Bangladesh, with huge population density, scarcity of buildable urban land, low institutional efficiency, it should be adopted rigorous process and participation of wide range of stakeholders in policy formulation and adoptation. In formulation of the future policies, participation of wide cross section of people including academics, civil society, peoples representatives, profeesional institutes of engineers, architects, planners, business groups and general mass of the people should be ensured to adopt a people centered realistic policy for sustainable built environment and urban development in Bangladesh. A substantial portion of national resource is invested in building construction in both public and private sectors. In order to ensure optimum return of this investment and to achieve satisfactory performance of the building in terms of safety, serviceability, health, sanitation and general welfare of the people, building construction needs to be controlled and regulated.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CAPACITY OF LOAD BEARING MASONRY WALL: EXPERIMENTAL, NUMERICAL AND ANALYTICAL RESULTS

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ABSTRACT

Masonry load bearing wall subjected to vertical concentric and eccentric loading may collapse through instability. The buckling behavior of masonry load bearing wall of different slenderness ratio were investigated via testing a series of scale masonry wall subjected to concentric and eccentric vertical loading. A total of thirty six masonry walls were tested in the Laboratory of Technical University of Catalonia which was the basis of numerical simulation, and results such as vertical load capacity, horizontal deflection and type of failure were investigated. In this research, a numerical finite element model was developed based on the simplified micro model approach for better understanding of buckling failure of the masonry load bearing wall and to simulate the response of walls tested in laboratory. The numerical model was calibrated by using those results found from the experimental study. However, a series of analytical studies were conducted in order to access the accuracy and performance of formulations provided by EUROCODE 6 and ACI-530 for vertical capacity of masonry wall.

Keywords: *masonry wall, strength capacity, bucking failure, eccentric vertical load, slenderness ratio.*

1. INTRODUCTION

The buckling response of masonry walls depends on parameters such as the cross-sectional area, the material properties, the slenderness (that is, the height to thickness ratio) and the effective eccentricities of loads at each end of the element. In masonry design standards, these factors are treated in different ways. For example, the north-American ACI-530 code and the European Eurocode-6 (EC-6) show significant differences in the analysis of brickwork walls under eccentric axial load. Whereas ACI-530 code analyzes the buckling failure and the cross-section material failure separately, EC-6 deals with both failure modes in a single analytical formulation. The literature on the subject shows large number of studies carried out on axially loaded walls with varying slenderness ratio. The calculation methods used in these studies range from analytical or empirical approaches to up-to-date numerical models based on the micro-modeling strategy. Among the first, Chapman and Slatford (1957) obtained closed form solutions for the load deformation behavior of brittle elastic wall by assuming that masonry material has no tensile strength and that cracking occurs whenever a tensile stress would develop. After that Yokel's (1971) results on the buckling of walls made of no-tension material are well known. While Yokel (1971) assumed a lineal stress relationship, other authors have moved to better-fitting, more sophisticate descriptions of the stress distribution in the wall cross section. De Falco's proposal (2002) on the stability of columns using an elastic-plastic material model stands among the most recently presented analytical approaches. More recently, Mura (2008) has utilized a parabolic stress-strain relationship to describe the behavior of the brickwork under compression loads. Shalin (1978) reviewed the results of analysis carried out by a number of authors and presented experimental evidence in support of the calculations. Further work was carried out by Sawko and Towler (1982) who proposed a numerical procedure for calculating the failure load of a no-tension material wall. Some analytical solutions also have been worked out for linear elastic material with or without tensile strength. An analytical solution has been carried out by Romano et al. (1993), considering no tension bearing masonry with a monomial stress–strain relationship in compression. However, all this models neglect some features of the masonry material which became significant especially in the prediction of failure load. Practically, load bearing masonry wall has tensile strength and the actual stiffness of a partially cracked wall is influenced by the tension stress field in the units which remain intact between the cracks at the unit-mortar interfaces. As a result, analyses which assume a no-tension material are imprecise for some forms of structural masonry work. Parland et al. (1982) proposed a method for determining buckling failure load of a slender wall, taking into account the effect of tension stress field which exists between the cracked joints. However, the linear elastic materials were used in this analysis.

The micro-modeling strategy, at present one of the most accurate tools available to model the behavior of masonry, and has been adopted in the present research in order to carry out the needed numerical simulations. Micro-modeling permits, in particular, an appropriate simulation of the buckling response taking into account joint tensile cracking in combination with masonry crushing in compression.
In this paper, the predictions of the ultimate capacity of walls obtained by means of micromodeling approach is compared with experimental results obtained by other authors. In turn, results obtained from current masonry standards are also considered and compared with the experimental results. Conclusions are drawn on the ability of both the numerical models and standards' formulations to accurately predict the experimental results.

2. EXPERIMENTAL STUDY

This chapter deals with the description of the testing program carried out in Structural Technology Laboratory of the Technical University of Catalonia which was the basis of numerical simulation of this research. The following is the characterization of materials used for the manufacture of the walls and a description of the manufacturing process, with emphasis on those details because it is a 1:4 scale study. Thirty six walls were tested with slenderness ratio (calculated as h / t) 6, 12, 18 and 25 and values of eccentricity of load was e = 0, e =t/6 and e = t/3. A summary of the number and type of the test specimens and test program is shown in Table 1.

Table 1: Summary of number and type of test specimens.

Wall series	Slenderness ratio	Height (cm)	Eccentricity of load	Support condition	Observation
W-0-6 W-0-12 W-0-18 W-0-25	6 12 18 25	21 42 63 87.5	0 0 0 0	Hinge-hinge	Slenderness ratio calculated as H/t
W-1/6-6 W-1/6-12 W-1/6-18 W-1/6-25	6 12 18 25	21 42 63 87.5	t/6 t/6 t/6 t/6	Hinge-hinge	Slenderness ratio calculated as H/t
W-1/3-6 W-1/3-12 W-1/3-18 W-1/3-25	6 12 18 25	21 42 63 87.5	t/3 t/3 t/3 t/3	Hinge-hinge	Slenderness ratio calculated as H/t

The walls were constructed as single leaf with scale of 1:4. The width was of 297 mm and thickness of 35 mm. In order to introduce different slenderness ratio, walls were built with heights of 210 mm, 420 mm, 630 mm and 875 mm. The thickness of the vertical and horizontal joints was approximately 2.5 mm. Figure 1 shows the layout of the tested wall.

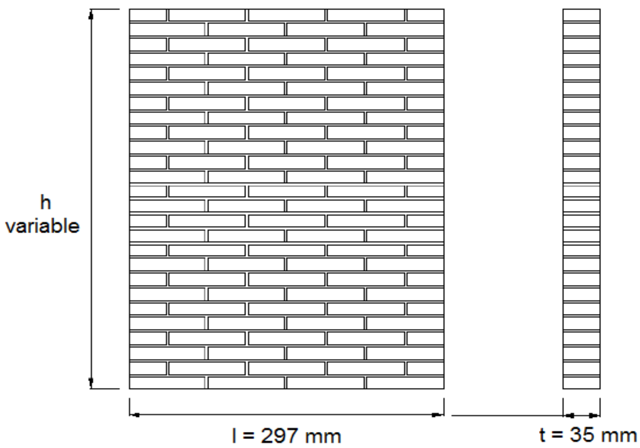


Figure 1: Layout of tested wall.

One fourth scale bricks were used for the construction of walls. The dimensions of the bricks (Length x width x thickness) are 72.5 x 35 x 12.5 mm. The average density of the brick was found, 1717.17 kg/m^3, after the drying process. The compressive strength of the brick is f_b = 32.45 MPa. The mortar used for the construction of the masonry walls was an M-8 prepared mortar. In order to adjust the fineness of the mortar for 1:4 scale sieving is made of it, removing all the percentage of material retained by the sieve1 mm aperture, since the presence of larger sizes difficult to maintain size of the joint. The average value of flexural strength is 3.05 MPa and the average compressive strength of the mortar is 7.29 MPa which was taken as the value for the further calculation. To determine the uniaxial compression and Young's modulus of elasticity five specimens of 147.5 mm x 147.5 mm (height of 10 rows and width of 2 pieces) considered. The average value of the compressive strength and Young's modulus obtained from the test are 14.2 MPa and 3458 MPa respectively.

3. MODELING OF EXPERIMENTAL DATA

The micro-modelling strategy, is considered at present as one of the most accurate tools available to model the behaviour of masonry structures, and has been adopted in the present research in order to carry out the needed numerical simulations. Micro-modelling allows, in particular, an appropriate simulation of the buckling response taking into account joint tensile cracking in combination with masonry crushing in compression. The numerical simulation presented is performed with the well-known micro-model proposed by Lourenco & Rots (1997) requires more specific software oriented to masonry analysis. For all cases, micro-models assume 2D plain-stress and a hinged-hinged configuration. The hinges are modeled by means of stiff triangular objects placed at the bottom and at top of the wall, whose end vertex is allowed to freely rotate. The units were modelled by using plain-stress continuum 8-node elements and for the mortar joints adopted 6-node zero-thickness line interface elements. Each unit was modelled with 12 x 3 elements. In addition, a minimum eccentricity of 1mm is always applied in order to account for possible irregularities of the wall geometry of the load positioning. Basically, the model assigns an elastic behavior to the units whereas masonry inelastic behavior is transferred to the joints. This analysis was performed with DIANA software. The integration schemes used are 2x2 points Gauss integration for the continuum elements and 3 points Lobato integration for the interface elements.

3.1 Material Properties

The material parameters used for the numerical simulation are shown in the Table 2. Some parameters such as G^I_f and C_S have been taken directly from the previous research The fracture energy for mode I, G^I_f have been taken from the test carried out by Van der Pluijm (1992) and for the parameter of shape of elliptical cap C_S a value of 9 has been adopted from Lourenco (1996). The different strength values f_t, c and f_m have been obtained from the experimental study carried out in UPC (2010). The compressive fracture energy G_{fc} and equivalent relative displacement K_p calculated according to Model Code 90 and Eurocode 6, respectively.

Table 2: Material parameters adopted for numerical analysis.

Components	Parameter	Symbol	Units	Values
Brick	Elastic modulus	E_b	N/mm^2	4800
	Poison ratio	v	-	0.15
	Tensile strength	f_{tb}	N/mm^2	3.95
Joint	Normal stiffness	k_n	N/mm^2	2800
	Shear stiffness	k_t	N/mm^2	1900
	Bond tensile strength	f_t	N/mm^2	0.554
	Mode – I fracture energy	G^I_f	Nmm/mm^2	0.02
	Cohesion	c	-	0.45
	Mode – II fracture energy	G^{II}_f	Nmm/mm^2	0.175
	Angle of internal friction	$tan\varphi$	-	0.812
	Angle of dilatancy	$tan\Psi$	-	0.009
	Compressive strength of masonry	f_m	N/mm^2	14.20
	Compressive fracture energy	G_{fc}	Nmm/mm^2	20.38

4. ANALYTICAL APPROACH

4.1 EUROCODE-6 (EC-6)

According to Eurocode-6, the resistance of a masonry wall subjected to vertical load depends on the geometry of the wall, the eccentricities of the load and constituent material properties. This development allowed the following assumptions:

- After each cross-section deformation remains plane and normal to the deformed axis (Bernoulli-Navier hypothesis);
- The resistance of the wall in tension perpendicular to the bed joints is zero.

4.1.1 Determination of Vertical Load Resistance

The vertical load resistance of a single leaf wall per unit length, N_{RD}, can be calculated as:

$$N_{RD} = \frac{\Phi_{i,m} t f_k}{\gamma_M} \tag{1}$$

Where,

$\Phi_{i,m}$ is the capacity reduction factor Φ_i (top or bottom of wall) or Φ_m (in the middle one fifth of the height of wall), allowing for the effects of slenderness and eccentricity of loading;

f_k is the characteristic compressive strength of masonry according to paragraph 3.6.2 of EC 6, if the cross-sectional area A is less than 1 m^2, this property is multiplied by the factor (0.7 + 3A);

γ_M is the partial safety factor for the material, under paragraph 2.3.3.2 of EC 6;

t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm.

4.1.2 Determination of Reduction Factor for Slenderness ratio and Eccentricity

- At the top or bottom of the wall.

$$\Phi_i = 1 - \frac{2e_i}{t} \tag{2}$$

Where,

e_i is the eccentricity at the top or the bottom of the wall;

$$e_i = \frac{M_i}{N_i} + e_{hi} + e_a \geq 0.05t$$

M_i is the design bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, according to 4.4.7 (Figure 4.1) of EC 6;

N_i is the design vertical load at the top or bottom of the wall;

e_{hi} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (for example, wind);

$e_a = \frac{h_{ef}}{450}$; is the accidental eccentricity and

t is the thickness of the wall.

- In the middle one fifth of the wall height.

$$\Phi_m = A_1 e^{(-\frac{u^2}{2})} \tag{3}$$

Where,

A_1 Numerical factor, $A_1 = 1 - 2\frac{e_{mk}}{t}$

u Numerical factor, $u = \dfrac{\left(\frac{h_{ef}}{t_{ef}} - 2\right)}{23 - 37\frac{e_{mk}}{t}}$

h_{ef} is the effective height, obtained from 4.4.4 of EC 6 for the appropriate restraint or stiffening condition;

t is the thickness of the wall;

t_{ef} effective thickness of the wall in accordance with paragraph 4.4.5 of the EC 6;

e base of natural logarithms, approximately, e = 2.71828;

e_{mk} is the eccentricity within the middle one fifth of the wall height;

$$e_{mk} = e_m + e_k \geq 0.05t$$

with $e_m = \dfrac{M_m}{N_m} + e_{hm} \pm e_a$

e_m is the eccentricity due to loads;

M_m is the greatest moment within the middle one fifth of the height of the wall resulting from the moments at the top and bottom of the wall, see Figure 2 (a);

N_m is the design vertical load within the middle one fifth of the height of the wall;

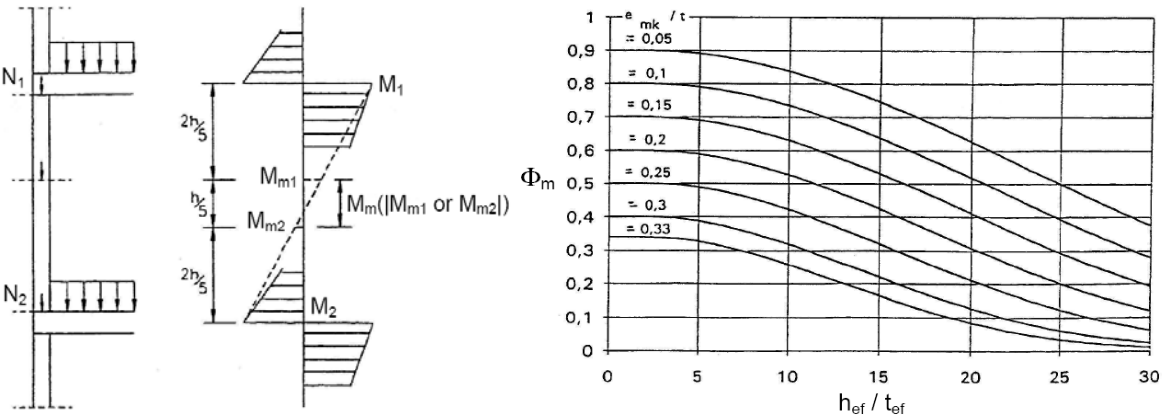
e_{hm} is the eccentricity at mid-height resulting from horizontal loads (for example, wind);

e_k is the eccentricity due to creep;

$$e_m = 0.002\Phi_\infty \frac{h_{ef}}{t_{ef}} \sqrt{te_m}$$

Φ_∞ is the final creep coefficient from Table 3.8 of EC 6.

Figure 2 (b) shows the values of Φ_m depending on the slenderness for different values of eccentricity of expression 3. This chart is entered with the value h/t and the eccentricity and extracts the value of Φ_m. This development has taken a modulus of the elasticity of masonry as a thousand times the compressive resistance property of masonry ($E = 1000 f_k$).



(a) (b)

Figure 2: (a) Moment according to EC 6 and (b) values of Φ_m against slenderness ratio for different eccentricity.

4.2 ACI-530

ACI-530-05 code was reported by Masonry Standards Joint Committee (MSJC), the design of unreinforced masonry has included a limit on the allowable axial compression force that may be applied. The limit is that the maximum allowable compressive force P is not exceed one fourth of the buckling load P_e as defined in the code. The maximum compressive force is limited to:

$$P \leq \frac{P_e}{4} \tag{4}$$

Where,

$$P_e = \pi^2 \frac{E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3 \tag{5}$$

In which, $= E_m$ modulus of elasticity; I = uncracked moment of inertia of the section; e = eccentricity of the compressive force P; r = radius of gyration of the uncracked unit section; h = unbraced height of the member under load. As the member deflects and bends under the action of eccentrically applied force, flexural tension cracking occur wherever the bending stress due to moment exceed the axial compression stress. The buckling equations for members subjected to compressive force are shown below:

$$P_e = \pi^2 \frac{E_m I}{h^2}\left(1 - 2\frac{e}{t}\right)^3 \tag{6}$$

For a solid rectangular cross-section, the radius of gyration is approximately equal to $0.289t$. For members having an slenderness ratio less than 99 and greater than 99, the allowable compression stress under axial load F_a is given from the following equation 7 and 8, respectively:

$$F_a = \left(\frac{1}{4}\right) f_m \left(\frac{h}{140rh}\right)^2 \tag{7}$$

$$F_a = \left(\frac{1}{4}\right) f_m \left(\frac{70r}{h}\right)^2 \tag{8}$$

Where, f_m specified compressive strength of masonry; t thickness of the wall.

5. COMPARISON OF COLLAPSE LOADS

A comparison between the experimental results obtained from UPC (2010), the results calculated with standards ACI-530 and EUROCODE 6 (EC 6) and the proposed numerical micro-models is presented in Figure 3 and Figure 4. As can be observed, the method proposed in EC 6 underestimates substantially the bearing capacity of the walls. The other standard considered, ACI-530, also underestimates the strength of walls in all cases.

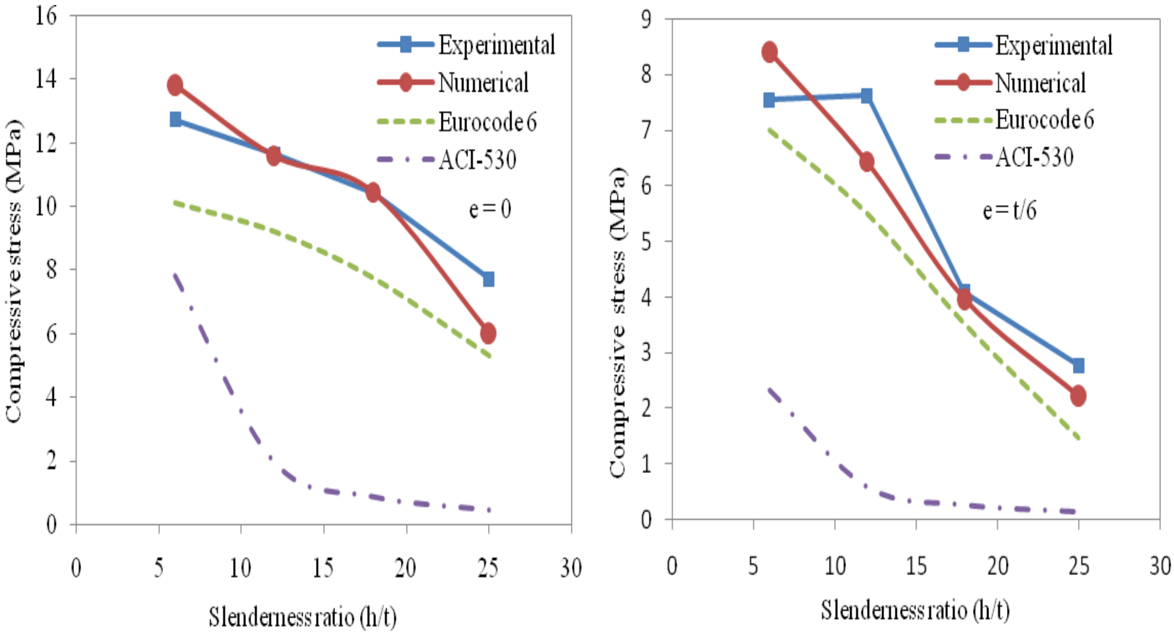


Figure 3: Comparison of compressive stress for different slenderness ratio and eccentricity e = 0 and e = t/6.

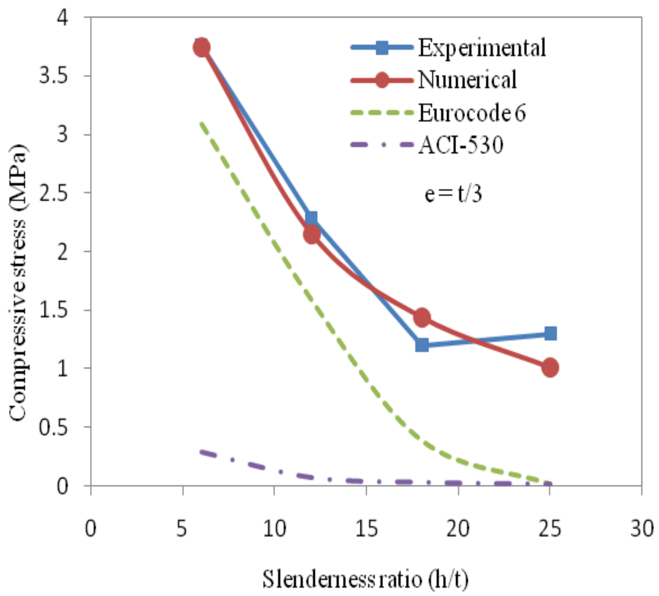


Figure 4: Comparison of compressive stress for different slenderness ratio and eccentricity e = t/3.

ACI-530 code produce average error of 76.86%, 87.62% and 96.26% compared to experimental results for the cases of load eccentricity 0, t/6 and t/3 respectively. For overall cases this code shows average error 86.92%. EC 6 underestimates the strength of the walls in all cases, although it is able to reproduce the general tendency. This code provides more satisfactory estimations of collapse loads for the lower eccentricity and lower slenderness ratio specifically, for the eccentricity t/6 which provides average error of 23.94%. Moreover, EC 6 is the most conservative method to predict the collapse load of the higher eccentrically loaded walls (e=t/3) with an average error of 53.6% while it produce overall average error of 34% when compared with experimental results.
The best fit occurs in the EC 6 (7.29%) and ACI-530 (38.47%) for the eccentricity t/6 and 0 respectively, with slenderness ratio 6 for both, while the bothe standards produce maximum error of 98.70% and 98.77% for eccentricity t/3 and slenderness ratio 25. On the other hand, both standards are underestimates the collapse load of wall when compared to numerical micro-models. ACI-530 provides most conservative results with average error of 87.19%, while EC 6 estimates collapse load by 31.30% of average error.

6. CONCLUSIONS

The diverse combinations of slenderness ratio and load eccentricity used in the experimental program which provided the means for a comprehensive numerical analysis of the masonry wall. In this research a set of experimental tests on the buckling failure of masonry walls has been numerically simulated by means of simplified micro-modeling approach. The micro-model describes the nonlinear response of masonry in compression in an indirect way by localizing it to the blocks to the joints. In all cases, the non-linear response in tension is localized to the joints. In addition, experimental and numerical results have been compared with predictions obtained from two current masonry standards.
The simplified micro-models afford a satisfactory prediction of the ultimate load of walls taking into account the buckling behavior. Simulations carried out by the micro-model provide the best fits for all load eccentricity, (with an average error of 10.79%). It must be noted that some difference with respect to the experimental results is unavoidable because of the influence of possible non-reported accidental eccentricities.
The comparison between experimental and the standards' results shows significant errors of 86.92% and 34.07% for ACI-530 and EC 6, respectively. In particular, this comparison suggests that the both method proposed by EC 6 and ACI-530 tends to conservatively underestimate the strength of walls. The micro-modeling approach has shown its ability to assess the bearing capacity of masonry walls subjected to concentric or eccentric vertical loading. It has been observed that an accurate description of tensile cracking and opening of mortar joints, by means of an appropriate interface element, is essential to obtain reliable results on the bucking failure of walls.

ACKNOWLEDGEMENTS

In completing this research, I would like to express my grateful appreciation and special thanks to Professor Paulo Lourenco, Professor Enrico Garbin and PhD student Cristian Sandoval. A financial support from European Commission through Erasmus Mundus Scholarship is gratefully acknowledged.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CORROSION EFFECT ON STRENGTH AND COVERING OF REINFORCED CONCRETE STRUCTURE

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ABSTRACT

Reinforced concrete structures have the potential to be very durable and capable of withstanding a variety of adverse environmental conditions. However, failures of the structures may occur as a result of premature reinforcement corrosion. The main aim of this paper is to evaluate the corrosion condition of reinforced concrete structures and to assess the structural behavior. Accelerated constant current corrosion tests were performed to investigate the influence of reinforced bar corrosion on the bond performance. To carry out these tests, sixty concrete cylinders of 100 mm and 150 mm diameter with two different compressive strengths of 16 MPa and 38 MPa were made. Direct current (DC) supply was applied in a corrosion cell containing 5% NaCl solution to expedite the corrosion propagation on reinforced concrete samples. Pull out test was performed to find out the bond stress of each specimen. From the experiment it was observed that the corrosive resistance increases with the increase of strength and also covering of concrete.

Keywords: *corrosion, bond strength, accelerated current supply, covering*

1. INTRODUCTION

Reinforced concrete is a versatile, economical and successful construction material. It can be formed to a variety of shapes and finishes. Usually it is durable and strong, performing well throughout its service life. However, sometimes it does not perform adequately as a result of poor design, poor construction, inadequate materials selection, a more severe environment than anticipated or a combination of these factors (Broomfield 1997).

Corrosion of reinforcing steel in concrete is the predominant causal factor in the premature degradation of reinforced concrete structures (Broomfield 1997). Practical experience and observations suggest that, although many RC structures are seen as “badly” deteriorated, characterized by mass concrete cracking and spalling, they are still structurally sound (Dhir et.al. 1999). The reason for this is attributed to the nature of the problem; the corrosion products exert an expansive stress on concrete the tensile strength of which is usually low. It is also partially due to the fact that the safety factors used in structural design for strength are usually larger than those for serviceability since the paramount importance of structural safety.

Enormous losses occur due to corrosion every year in all countries of the world. These losses are made up of direct losses (which include price of corroded metal and labor cost of replacement) and indirect losses (which incorporate losses in efficiency, contamination of products, explosions and possible loss of life, wastage of material due to over-designing provision as well as the cost associated with the prevention of corrosion). One American estimate is that $150 billion worth of corrosion damage on their interstate highway bridges is due to deicing and sea salt induced corrosion. In a recent, Transportation Research Board Report on the costs of deicing (Transportation Research Board 1991), the annual cost of bridge deck repairs was estimated to be $50 to $200 million, with substructures and other components requiring $100 million a year and a further $50 to 150 million a year on multistory car parks.

The physicochemical interaction between a metal and its environment which results in changes in the properties of the metal and which may often lead to destruction of the function of the metal, the environment (Demadis et al. 2006; Javaherdashti 2000). It has been established as the predominant factor causing widespread premature deterioration of concrete construction worldwide, especially of the structures located in the coastal marine environment (Song et.al. 2007). The most important causes of corrosion initiation of reinforcing steel are the ingress of chloride ions and carbon dioxide to the steel surface. After initiation of the corrosion process, the corrosion products recognized as red-brown dust (hydrous ferric oxide Fe_2O_3. $3H_2O$) are usually deposited in the restricted space in the concrete around the steel. Their formation within this restricted space sets up expansive stresses, (the corrosion products resulting from the corrosion of steel reinforcing bar occupy a volume equal to three to six times that of the original steel) which crack and spall the concrete cover. This in turn results in progressive deterioration of the concrete. The corrosion induced pressure caused by this expansion at the rebar-concrete interface causes splitting of the concrete cover. When assessing structural integrity, the reduction in bond strength by splitting can be a bigger factor than the loss of reinforced bar tensile load capacity by cross section reduction (Almusallam et. al. 1996). Thus two different mechanisms might thus be responsible for loss of bond of plain bars as a result of corrosion [Cairns et al. 2006].

a) a mechanically weak layer of corrosion products at the steel-concrete interface.
b) a reduction in the confinement as cover crack develop along the bar owing to volumetric expansion of the products of corrosion.

As a result, corrosion affected RC structures are more prone to cracking [than, e.g., loss of strength], incurring considerable costs of repairs and inconvenience to the public due to interruptions. This gives rise to the need for thorough investigation on corrosion induced cracking process in order to achieve cost-effectiveness in maintaining the serviceability of the RC structures. Potentially corrosion rehabilitation is a very large market for those who develop the expertise to deal with the problem. It is also a major headache for those who are responsible for dealing with structures suffering from corrosion. The physical barriers such as sealers and membranes, admixtures, overlays, and coating on steel reinforced bar can prevent or delay the ingress of chlorides, oxygen, moisture through the concrete cover to the reinforced steel. Corrosion resistant material includes austenitic stainless steel and fiber reinforced polymer (FRP) can also be used (U.S Department of Transportation 2000).

Considerable research has been conducted to investigate corrosion behavior of concrete material and prevention methods. In experimental investigations on corrosion induced cracking in concrete, the corrosion process is usually by various means so that the concrete cracking can the achieved in a relatively short time (Alonso et al. 1998; Andrade et al. 1993; Liu et.al. 1998; Francois et.al. 1998). The composition, pore structure and size of the bulk concrete cover play a significant role in corrosion initiation period as well as rate of corrosion.

In this research accelerated constant current corrosion tests are performed on rebars embedded in concrete cylinders. The influence of concrete strength and cylinder size, on the extent of corrosion and on the bond between steel-concrete is explored. The effect of the above parameters on the rate of bond degradation is observed and subsequently, the corrosion resistance on concrete strength and covering was investigated.

2. EXPERIMENTALS

2.1 Materials

2.1.1 Cement

The Portland composite cement was used as binding material for preparing the specimens. The different mechanical properties of cement are represented in Table 1 below.

Table 1: Properties of Cement

Type	Compressive Strength (MPa)			Normal Consistency (%)	Initial Setting Time (minutes)	Final Setting Time (minutes)	Fineness (%)
	3 days	7 days	28 days				
PCC	12	19	28	28.3	90	195	2.36

2.1.2 Aggregate

2.1.2.1 Coarse aggregate

Locally available stone chips of 12.5 mm downgrade was used as coarse aggregate. Some Physical properties are shown in Table 2 and the gradation curve is shown in Fig. 1.

Table 2: Properties of stone chips

Properties	
Type	Stone chips
Maximum Size	12.5 mm passing
Specific Gravity	2.60
Absorption Capacity (%)	0.52
Unit Weight (Kg/m^3)	1448

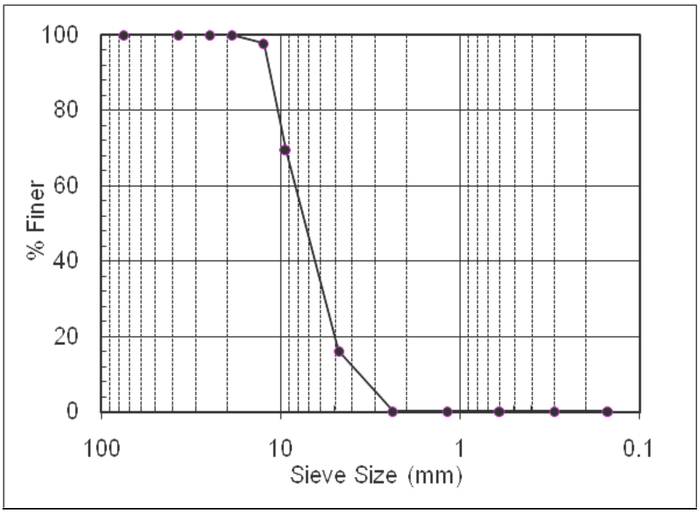


Figure1: Gradation curve for stone chips

2.1.2.2 *Fine aggregate*

Available river sand with F.M. value 2.8 was used as fine aggregate. The different physical properties are shown in Table 3 & 4.

Table 3: Properties of River bed sand

Properties	
Type	River bed Sand
Specific Gravity	2.75
Absorption Capacity (%)	2.35
Unit Weight (Kg/m^3)	1560

Table 4: Fineness modulus (F.M.) of sand

Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative Weight Retained (%)	% Finer	F.M. Value
No. 4	4.75	0.00	0.00	0.00	100.00	2.80 (Two Point Eight Zero)
No. 8	2.36	30.00	30.00	6.00	94.00	
No. 16	1.18	150.00	180.00	36.00	64.00	
No. 30	0.60	134.60	314.60	62.92	37.08	
No. 50	0.30	100.60	415.20	83.04	16.96	
No. 100	0.15	42.70	457.90	91.58	8.42	
Summation of Cumulative Weight Retained (%) =				279.54		

2.2 Specimen Design

The light weight aggregate concrete batch was mixed with cement (PCC), sand (FM = 2.8), stone chips (d_{max} =12.5 mm) and water in ratio of 1:3.5:3.0:0.65 and 1:2.3:2.1:0.60 by weight to achieve 28 days compressive strength of 16 MPa and 38 MPa respectively representing in Table 5. The reinforced bars were positioned in the mould's centre after placing 25 mm of concrete from the bottom of the moulds and the rest concrete was cast. Then the specimens were cured for 28 days in a standard curing room (temperature of about 23^0C). The specimen geometry was shown in Fig. 2.

Table 5: Material compositions and strength in concrete mix.

<table>
<tr><td rowspan="2">Concrete Ingredients</td><td colspan="2">Mixing Proportions</td><td colspan="6">Strength (Mpa)</td></tr>
<tr><td>Concrete Type I</td><td>Concrete Type II</td><td colspan="3">Concrete Type I</td><td colspan="3">Concrete Type II</td></tr>
<tr><td rowspan="2">Cement</td><td rowspan="2">1.0</td><td rowspan="2">1.0</td><td rowspan="2">Target</td><td colspan="2">Tested</td><td rowspan="2">Target</td><td colspan="2">Tested</td></tr>
<tr><td>Individual</td><td>Average</td><td>Individual</td><td>Average</td></tr>
<tr><td>Fine Aggregate</td><td>3.5</td><td>2.3</td><td rowspan="4">16</td><td>16.2</td><td rowspan="4">16.1</td><td rowspan="4">38</td><td>37.3</td><td rowspan="4">38.3</td></tr>
<tr><td>Coarse Aggregate</td><td>3.0</td><td>2.1</td><td>15.8</td><td>39.4</td></tr>
<tr><td>Water</td><td>0.65</td><td>0.60</td><td>16.1</td><td>38.2</td></tr>
<tr><td>Admixture</td><td>--</td><td>--</td><td>16.3</td><td>38.4</td></tr>
</table>

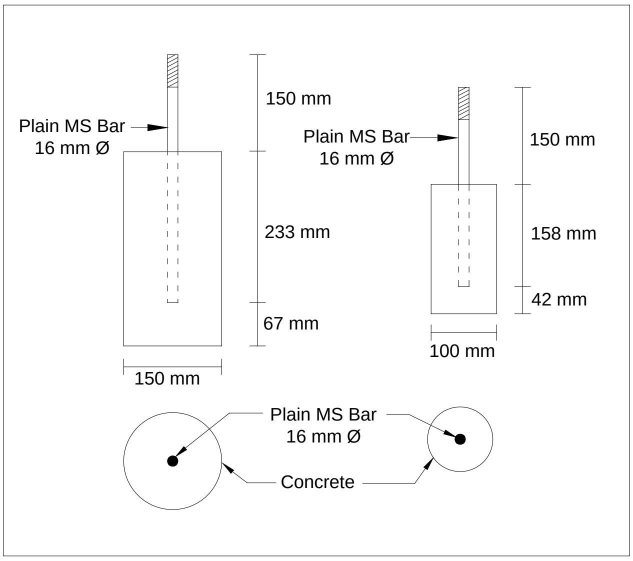


Figure 2: Detail Drawing of Specimen

A total of four different categories of specimens were casted with two concrete protective layers (67 mm and 42 mm) and two different strengths (16 MPa and 38 MPa). In our experiment the embedded reinforce plane bar diameter was constant of about 16 mm. For corrosion protecting during curing period a coating of paint was applied in the exposed rebar zone.

2.3 Corrosion Cell

The corrosion cell (Fig. 3) consisted of a constant current power (DC supply), a shunt- resistor (2.2 ohms), a copper plate, and the light weight concrete specimen submerged in a 5% wt. NaCl solution.

The corrosion cell consist of two dissimilar types of metals in contact and shearing a common electrolyte (sodium cloride solution), is called a galvanic cell. Of the two special types of metal, the more negative standard electro potential value uses as anode, whereas the less negative standard electro potential value serves as cathode.

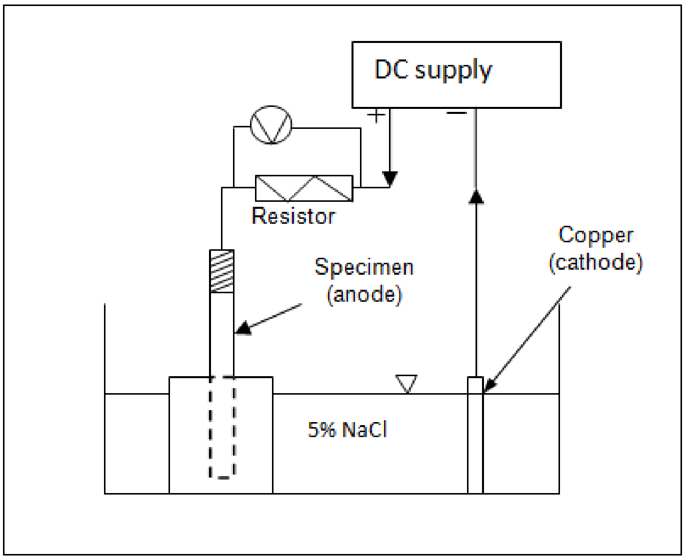


Figure 3: Corrosion Cell

In the research, mild steel (E = -0.61 Volts) and copper (E = -0.36 Volts) are used as anode and cathode respectively. A setup with parallel electric connection is shown in Fig. 4.

Figure 4: Parallel connection in corrosion cell

2.4 Bond Strength Measurement Method

Three samples of particular strength and covering were disconnected from the electricity supply and tested on every week. Fine sand was used at the top of the concrete cylinder to keep the surface near about smooth. Then a steel plate was placed on the concrete. All these arrangements were done for distributing the load over the concrete uniformly coming from the jack, thus to avoid side splitting. After that the jack was placed over the plate and locked by using nuts. A dial gauge was also connected to measure the slip of steel during test.

When all the arrangements (Fig. 5) were completed, the lever arm was manually operated giving pressure by the jack to pullout the steel from the concrete. A load gauge connected with the jack give the amount of required load to pullout in kN directly. This test was done for every sample. Thereafter, the bond stresses were calculated by dividing the forces obtained from the test with the contact surface area of the steel surrounding the concrete.

Figure 5: Arrangement of Pullout Test

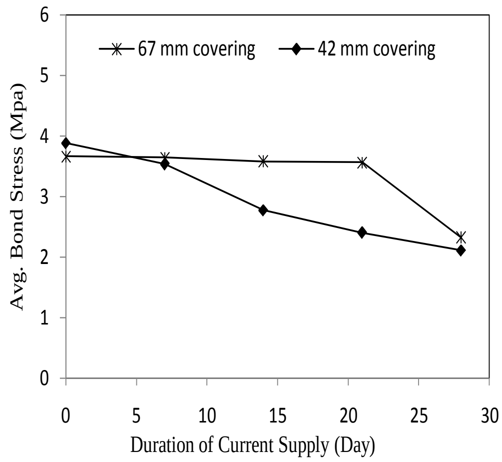
3. RESULTS AND DISCUSSIONS

In our research two different types of strength and two different types of covering of concrete were used and the corresponding bond stresses are shown in Table 1 below. The plain reinforced bar embedded into each cylinder was performed as anode and copper plate as cathode. The 5% wt. NaCl solution was used as electrode to transfer electron so the corrosion occurred only when the solution reached to the anode. That's why the distance and permeability of concrete have a great influence on the electron movement. Furthermore a higher strength offers a lower permeable concrete.

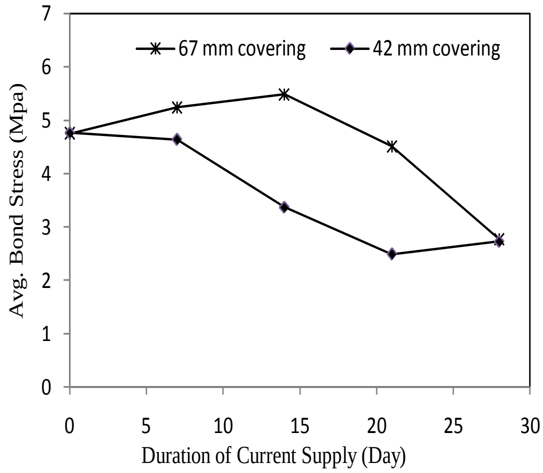
Table 1: Average bond stress according to concrete strength and covering

Concrete Strength (Mpa)	Concrete Covering (mm)	Duration of Current Supply (days)	Avg. Bond Stress (Mpa)
16	67	0	3.667
		7	3.645
		14	3.580
		21	3.568
		28	2.325
	42	0	3.885
		7	3.534
		14	2.773
		21	2.401
		28	2.109
38	67	0	4.752
		7	5.241
		14	5.484
		21	4.515
		28	2.773
	42	0	4.767
		7	4.638
		14	3.375
		21	2.494
		28	2.728

In Fig. 4 (a) it was shown that the bond stress gradually decreased with the duration of current supply. It was the reason of that at lower concrete strength, permeability of concrete was higher. So the electrode (Nacl solution) can easily enter into the concrete to propagate the concrete corrosion. Due to corrosion, a passive layer of rust was produced around the steel making a slip favour environment at the steel surface and thus reduces the bond stress. On the contrary in case of Fig. 4 (b), it can be revealed that there is a slight increase of bond stress with respect to initial value and then progressively decreased. The increase at the initial stage is due to the higher strength and low permeability of concrete. After the supply of current initially bond is gained by concrete until NaCl touches the steel.



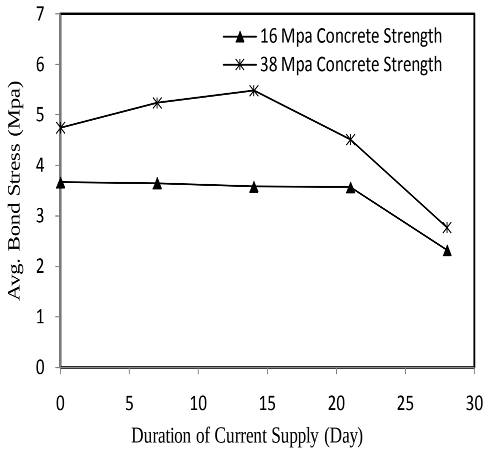
(a) For 16 Mpa Concrete



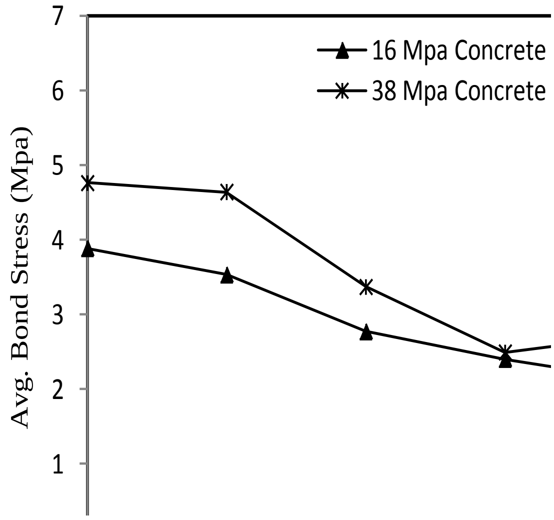
(b) For 38 Mpa Concrete

Figure 4: Graph of bond stress vs. time for different concrete strength

Fig. 5 (a) and 5 (b) show the results of bond stress, where the bond stress of the corrosive reinforced bar is normalized with respect to concrete covering. It can be detected that the influence of bond stress is higher for lower strength of concrete.



(a) For 67 mm concrete covering



(b) For 42 mm concrete covering

Figure 5: Graphs of bond stresses vs. time for different concrete covering

4. CONCLUSIONS

Based on the experimental work and argument presented, the following conclusions may be drawn.

- The accelerated constant current technique has been confirmed to be an effective and quick method of accelerating chloride-induced corrosion.

- The average bond stress increased for both types of strength when the concrete covering was increased from 42 mm to 67 mm
- High strength concrete showed lower degradation of bond stress.
- Higher strength or adequate covering of concrete may considerably reduce the occurrence of corrosion of reinforcing bar at adverse environment for the most part in reinforced concrete structure.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECTS OF AGGREGATE TYPE ON SPECIFIC HEAT OF CONCRETE

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ABSTRACT

Specific heat of concrete is an important parameter on which thermal diffusivity, conductivity and thermal expansion of concrete depend on. This paper represents the specific heat of concrete by using two different categories of coarse aggregates. For this purpose forty eight concrete cylinder of 150mm and 100 mm diameter were casted using burnt clay brick chips and stone chips as coarse aggregate with two different mixing ratios of 1:2:3 and 1:1.5:2. The specific heat was measured by using a semi adiabatic calorimeter. From the experiment it was observed that the specific heat of concrete having brick chips is greater than the concrete having stone chips. Again the specific heat is greater at lower mixing ratio for both types of aggregates.

Keywords: *specific heat, burnt clay brick chips, stone chips, calorimeter*

1. INTRODUCTION

Now-a-days temperature change is becoming a burning issue across the world. Due to global warming, temperature increases quickly. As a result of urbanization, huge amount of carbon di-oxide (CO_2) and carbon monoxide (CO) are produced and causing global warming. For this reason various civil engineering structures go through temperature changes. The development of high concrete temperatures could cause a number of effects that have been shown to be detrimental to long-term concrete performance. High concrete temperatures increase the rate of hydration, thermal stresses, the tendency for drying shrinkage cracking, permeability, and decrease long-term concrete strengths, and durability as a result of cracking (Schindler et.al. 2002). One of the challenges in the design of mass concrete structures is to avoid the initiation of cracks regardless of the concrete element size, concreting procedure, weather conditions and material properties. (Milovanovic et. al. 2011)

From the point of view of thermal behavior, concrete is one of the most problematic materials. The main problems are presented by the chemical changes that take place in the Portland cement paste. In the case of structural concretes the aggregates rarely present particular problems, but as the aggregates used in normal weight masonry units may also lack chemical stability at higher temperatures (Harmathy et. al 1973). However, thermal properties of concrete are depended by the concrete's mixture proportions, the thermo physical properties of the aggregates that it contains, and those of its hydrating cement (binder) paste component, (Bentz 2007; Bentz 2011). While the densities are generally well known and easily measurable, less information is available on the heat capacity and thermal conductivity, particularly those of the hydrating cement paste. The cement paste undergoes an uninterrupted series of dehydration reactions as its temperature rises from 100 ^{0}C (212 F) to about 850 ^{0}C (1562 F). These reactions affect the thermal properties of the cement paste in three ways; (1) by the absorption of latent heats, (2) by the changes brought about in the physicochemical characteristics of the solid matrix and, (3) by the changes in the porosity of the bulk material. The thermal conductivity depends on all three of these effects, although its dependence on the first is not satisfactorily understood (Zhuze et. al. 1969). The bulk density depends only on the second and third effects, and the specific heat only on the first and second.

The analysis for the temperature response of the structures can be subdivided into two major sequences, a nonlinear temperature analysis and a successive structural analysis (Kang et.al 2001; Pho et.al 1995). It should be noted that the material properties are also subdivided into two categories following the analysis type in this study. The density, the specific heat, and the thermal conductivity are used in the temperature analysis and called as thermal properties in this study. Whilst, the thermal expansion coefficient, the initial elastic modulus, the strength, and etc. are included in the mechanical properties for the successive structural analysis (Suk-Wang et.al. 2002).

Thermal gradients can cause internal stresses which can lead to cracking on a microscopic or macroscopic scale (Mehta 2006). Hence, the temperature of concrete during hardening is a major design consideration, especially since the time temperature history affects both strength and durability (De Schutter 2001). In some construction projects, thermal insulating blankets were used to minimize the interior and exterior concrete temperature gradient (Whittier et al. 2004). Generally, for crack control, maximum temperature difference within the concrete mass should not exceed 20^0C (Neville 1995), but with limestone aggregate, the difference can be allowed up to 31^0C (Portland Cement Association 2003). The prediction of temperature gradient of mass concrete is essentially useful for crack control analysis and design.

Specific heat measures the index of the facility with which concrete can undergo temperature changes. Specific heat is also related to thermal conductivity and thermal diffusivity. For this reason measurement of specific heat of concrete is very essential. It is defined as the amount of heat that a unit mass of a material must gain or lose to change its temperature by a given amount. The specific heat of a material is related to heat capacity and is a property of the materials. In MKS unit it would be expressed in term of Joules per kilogram per kelvin (J/kg-K). The common range of values for ordinary concrete is between 840 and 1170 J/kg-K. It is highly influenced by moisture content, aggregate type, and density of concrete (Phan 1996; Kodur and Sultan 1998; Harmathy 1970).

Initial studies (Lerch 1955) and duplicated research (Wilson 2005; Kosmatka 2011) has shown that to lower the temperature of normal concrete by 1°C, the temperature of the cement must be reduced by 8.2 °C, the temperature of the water reduced by 4.9 °C, and the temperature of the aggregates lowered only 1.5 °C. Thus the influence of cement temperature on overall concrete batch temperature is less than that of the other concrete ingredients. The reason for this effect can be traced to the mix proportions and the intrinsic thermal properties of the concrete ingredients. Of the materials added to normal concrete, cement occupies only 7-15% of the concrete volume. Water and aggregates generally constitute approximately 70-90% of the concrete volume. When compared to the weight of the other constituents, cement's potential influence on temperature is diminished. In addition, cement has a very low average specific heat capacity (0.92 kJ/kg-K) – meaning that it gains and loses heat very readily when exposed to high temperatures or when supplied with energy (Kosmatka et.al. 2011). Stone aggregates, on average, have an equivalent specific heat capacity to that of cement, but in case of the artificial aggregates say as; burnt clay brick chips may represent different thermal properties in concrete. For this purpose two different types of coarse aggregates are used to show the effect of aggregate and the mixing proportions on the specific heat of concrete.

2. MATERIALS AND METHOD

2.1 Materials

Forty eight concrete cylinders of two different sizes (4"× 8" and 6"× 12") were studied in the test series report here. The concrete batch was mixed in proportions of 1:2:3 and 1:1.5:2 parts by weight of cement (PCC), fine aggregate (FM=2.72) and coarse aggregates with water cement ratio 0.5. To identify the variation of specific heat with coarse aggregates, two different types (burnt clay brick chips and stone chips) of coarse aggregates are used.

Various properties (Table 1) such as, unit weight, percent voids, moisture content, specific gravity, gradations of coarse aggregates were determined following ASTM standard. The size of coarse aggregates for both types was kept same which are shown in gradation curve (Fig. 1), as we know there is a great effect on the size of the coarse aggregate in concrete. The FM of fine aggregate (sylhet sand) was also determined having value of 2.72.

Table 1: Properties of coarse aggregate

Properties	Coarse Aggregate Types	
	Bricks chips	Stone chips
Moisture content (%)	9.4	2.4
Voids (%)	50.56	56.8
Unit weight (kg/m^3)	898	1138
Specific Gravity	1.82	2.64

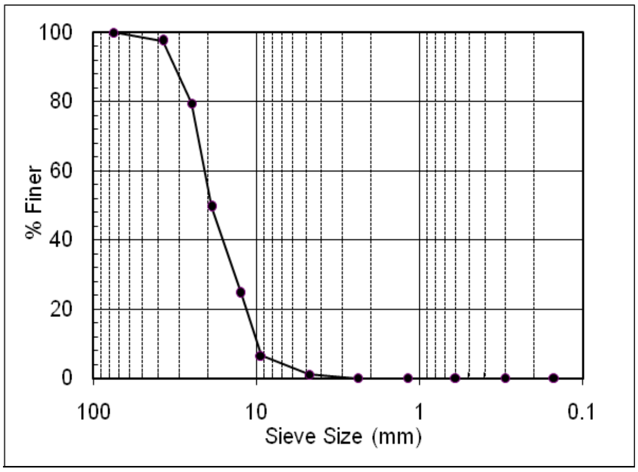

Figure 1: Gradation Curve for coarse aggregates

2.2 Experimental Procedures

The test specimens of 4"× 8" and 6"× 12" cylindrical shape were prepared and cured at room temperature for 28 days. After curing two holes were made at the middle third point by drilling with a drill bid of 3/8" (9.5 mm) in diameter. Immersion heater (2000 watt & 220-250 volt) into one hole and a thermometer into another hole were inserted of the cylinders and put them into the calorimeter.

In this experiment a semi adiabatic calorimeter (Fig.2) was made with a wooden box of size 1'8"×1'3"×2'6". The inside wall of the box is surrounded by 1" thick coksheet layer. Another small box of size 9"×9"×1'9" was made and inserted into the large box. The outside wall of the small box was covered with 1" thick coksheet layer and 1/2" thick glass wool layer. The inside wall is covered with mild steel plate.

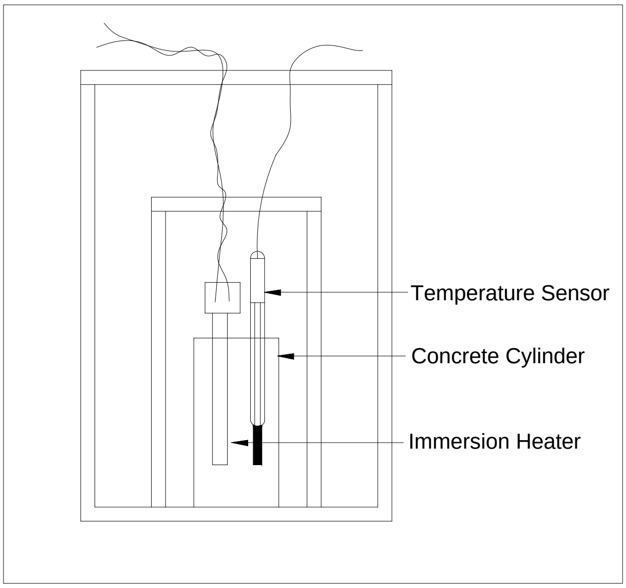

Figure 2: Line diagram of a calorimeter with a concrete sample

The concrete sample with heater and thermometer was put in the calorimeter and the cables of temperature sensor and heater was drawn out through the hole which was made on the top of the box. The electricity was supplied by multi-meter and different parameters such as current (I), voltage (V), time (t), initial temperature (θ_0), final temperature (θ_1) etc. etc were measured. Also the input energy (Q_1, Q_2) and mass of the samples (m_1, m_2) were measured. By putting those data in equation (1) the specific heat of the specimens were measured.

$$C = \frac{Q_2 - Q_1}{(m_2 - m_1)(\theta_1 - \theta_0)} \tag{1}$$

3. RESULTS AND DISCUSSIONS

The specific heat of forty eight concrete cylinders of different categories (mixing ratio, coarse aggregate type, size) were measured. From those, twenty four samples were sorted according to value so that we can determine the maximum and minimum average specific heat of concrete and to compare them how the values change with respect to the variation of coarse aggregates and the mixing ratio. In the sorting process the values are sorted in such a way that the variation among the values of each category does not exceed ten percent and averaging them which are shown in Table 2 below.

For the expediency, the samples are identified with some ID marks such as BR2 means the concrete using brick chips with mixing ratio 1:2:3; BR1.5 means the concrete using brick chips with mixing ratio 1:1.5:2; SR2 means the concrete using stone chips with mixing ratio 1:2:3; SR1.5 means the concrete using stone chips with mixing ratio 1:1.5:2

Table 2: Specific heat of different samples

Sample ID	Mix Proportion By Volume	W/C Ratio	Specific Heat (J/Kg-K)	Average Value (J/Kg-K)	Remarks
BR2	1:2:3	0.5	1430	1426	Maximum
BR2			1451		
BR2			1396		
SR2			1231	1204	
SR2			1189		
SR2			1193		
BR2			1139	1098	Minimum
BR2			1097		
BR2			1058		
SR2			978	978	
SR2			956		
SR2			974		
BR1.5	1:1.5:2		1386	1356	Maximum
BR1.5			1378		
BR1.5			1305		
SR1.5			1163	1158	
SR1.5			1181		
SR1.5			1130		
BR1.5			1025	1039	Minimum
BR1.5			1021		
BR1.5			1070		
SR1.5			906	938	
SR1.5			976		
SR1.5			932		

Fig. 3.1 compares the specific heat of concrete with mixing ratio 1:1.5:2. It can be clearly seen that the specific heat of concrete using the burnt clay brick chips is greater than the concrete having stone chips as coarse aggregate. The highest value of specific heat is 1356 J/Kg-K and the lowest value is 1039 in case of brick chips, but in case of stone chips the value varies from 1158 to 938 J/Kg-K.

Fig. 3.2 which reveals the specific heat of concrete with mixing ratio 1:2:3 showing the same characteristics with respect to coarse aggregate as in figure 3.1. The range of specific heat is 1426-1098 J/Kg-K in case of brick chips and in case of stone chips it varies from 1204 to 978 J/Kg-K. These values show some deviation from other researchers. According to Phan 1996; Kodur and Sultan 1998; Harmathy 1970, the common range of values for ordinary concrete is between 840 and 1170 J/kg-K. The variation of their result from this result may be due to the environmental conditions which have a great effect on concrete.

If we compare between two figure then we can find that the values (maximum and minimum) of specific heat of concrete having brick khoa as coarse aggregate is greater than the concrete having stone chips as coarse aggregate. On the otherhand, if we compare these values with respect to mixing ratio then it is clear that the specific heat of concrete at lower mixing ratio (1:2:3) shows the greater value of specific heat than the concrete of rich mixing ratio (1:1.5:2).

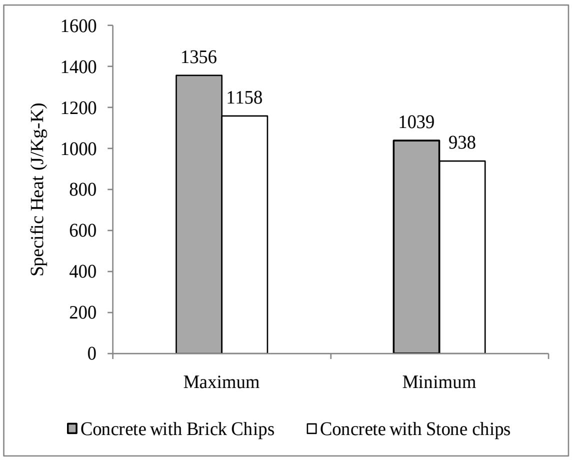

Figure 3.1: Specific heat of concrete of mixing ratio 1:1.5:2

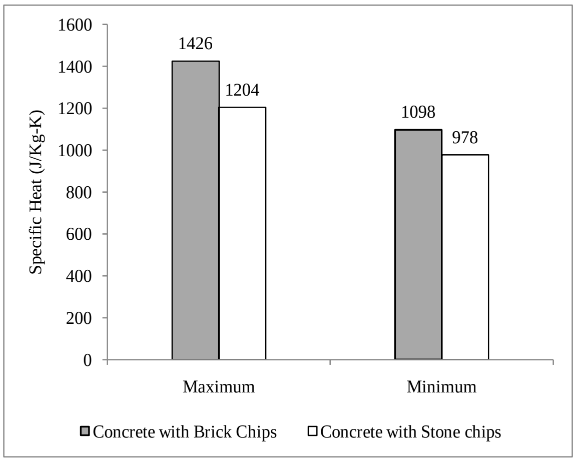

Figure3.2 : Specific heat of concrete of mixing ratio 1:2:3

As we know that the specific heat depends on the density of a material. The lower density shows the higher specific heat. If we use burnt clay brick chips than stone chips then the density of concrete is lower and the rich mixing ratio gives denser concrete. So the value of the experiment is reasonable in all conditions.

4. CONCLUSIONS

The following conclusions can be drawn from the experiment

- The burnt clay brick chips used as coarse aggregate shows near about 15% greater specific heat than the stone chips used as coarse aggregate in concrete.
- Specific heat is about 5% less in rich mixing ratio when compared with lower mixing ratio.
- The aggregate types and mixing ratios have a great influence on the specific heat of concrete.

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STRENGTHENING AND PERFORMANCE EVALUATION OF MASONRY SHEAR WALL USING DIFFERENT FRPs

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ABSTRACT

This paper investigates strengthening masonry shear walls using two types fiber reinforced polymer (FRP) sheets. An experimental research program is undertaken. Masonry wall made from wire-cut clay brick specimens are tested strengthening by PET (Polyethylene terephthalate) and CFRP (Carbon fiber reinforced polymer)sheets . Strengthening is considered on both side of the wall to ensure uniformity and symmetricity of stiffness of the wall. Static tests are carried out on eight masonry panels, under a combination of vertical precompresion, and in-plane horizontal shear loading. The mechanisms by which load was carried were observed, varying from the initial, uncracked state, to the final, fully cracked state. The results demonstrate that a significant increase of the in-plane shear capacity of masonry can be achieved by bonding with PET sheets to the surface of masonry walls. The experimental data were used to assess the effectiveness of the strengthening, and suggestions are made to allow the test results to be used in the design of strengthening for masonry structures. Test of walls strengthening by CFRP are still undeway and the test results are expected to come soon.

Keywords: *Brick; Masonry; Shear wall; FRP sheets; Shear tests.*

1. INTRODUCTION

Recent earthquakes in China, Indonesia, Iran, India and Pakistan have caused an extensive damage in a large number of existing masonry buildings and RC frame with masonry infill. Damage was mainly due to the in-plane shear failure and the out-of-plane bending or slip during the tremor of a moderately strong earthquake. Those huge losses on both human lives and properties demonstrate a burning need for retrofitting existing masonry structures in a viable mean that fits with the socio-economic condition of the people in those affected areas. For strengthening of these kinds of structures, different techniques have been developed and applied in many countries around the world. Among the available techniques, externally bonded Polyethylene Terephthalate (PET) fiber with a large fracture strain is one of the retrofitting technique that has drawn a significant attention as an unique alternative to CFRP or GFRP due to its pronounced ductile behavior and relatively low material cost (Fig. 1), but not compromising the other advantages of FRP like low weight-strength ratio, short installation period and a minimum intervention during the fabrication process. The main objective of using FRPs is to enhance performance of structure at normal loading condition and to offer greater resistance at the time of severe loading. Neither too much stiffness nor the very high strength will be coherent with the overall performance of the structures. Material with high stiffness will produce very little deformation before complete failure and the failure if any will be brittle and explosive in nature without any prior warning, which is not the expected failure mode from a well performed structure. On the other hand too soft material with nominal strength will not be well fitted with the purpose of strengthening. In this study, PET and CFRP have been used as strengthening materials. CFRP has a high stiffness among the other widely used FRPs, where as PET posses a relatively low stiffness but with a higher fracturing strain than CFRP. The purpose of this study is to show the difference in behavior of masonry shear wall for two distinct FRPs.

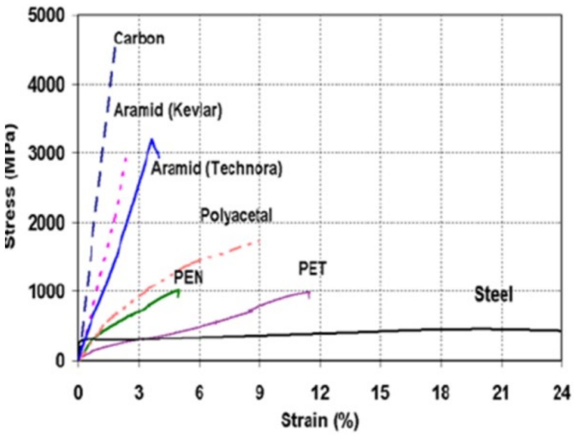


Figure 1: Stress-strain relationship of different FRPs

Ultimate load bearing capacity, deformation at peak load, mode of failures, are observed in this study. A comparative economic analysis will be done at the end of the study.

2. METHODOLOGY

In this experimental study 8 single leaf brick masonry walls with nominal dimension of 1270×1016×127 mm have been fabricated with wire cut brick having a compressive strength of 17 MPa. 10 mm thick mortar with a strength of 12 MPa has been used for all of the wall specimens. A pre-compression of 100 kN was applied on the top of the wall ahead of the shear loading to simulate the load of structural component coming on the wall from the above in the actual masonry wall. The wall top and bottom were attached with the steel plate by stiff putty and 30mm dowels. Finally the steel plates were bolted with very rigid top and bottom steel H-beams. Sufficient wire strain gages (KYOWA-5,10&20 mm) and LVDTs were used to record all the necessary information during the course of testing. Loads were applied on roller and ball-bearing system to avoid any excessive friction and deviation of loading points. The experimental set-up for the masonry shear wall can be seen in Fig 2. In the testing procedure, the relative stiffness of PET and CFRP was kept as same by using an equivalent amount of FRPs, an equivalent stiffness was ensured (Eq. 1).

$$A_{CFRP} = A_{PET}\frac{E_{PET}}{E_{CFRP}} \tag{1}$$

The FRP was applied by three different techniques, 100% , 40% and 20% coverage of the gross surface area of the wall to be tested. The different wrapping techniques and method of anchorage are shown in Fig. 3. Lateral load was applied on a lever arm to counter balance the over turning moment. Load was applied untill failure and a diagonal cracks were clearly visible with naked eyes. Figures 4(a) & 4(b) show the instrumentaion of two walls that were tested by the author (Rahman & Ueda 2011). They were 100% and 40% wrapped by PET sheet respectively. Table 1 gives the detail of shear walls and Table 2 shows the properties of PET and CFRP sheets

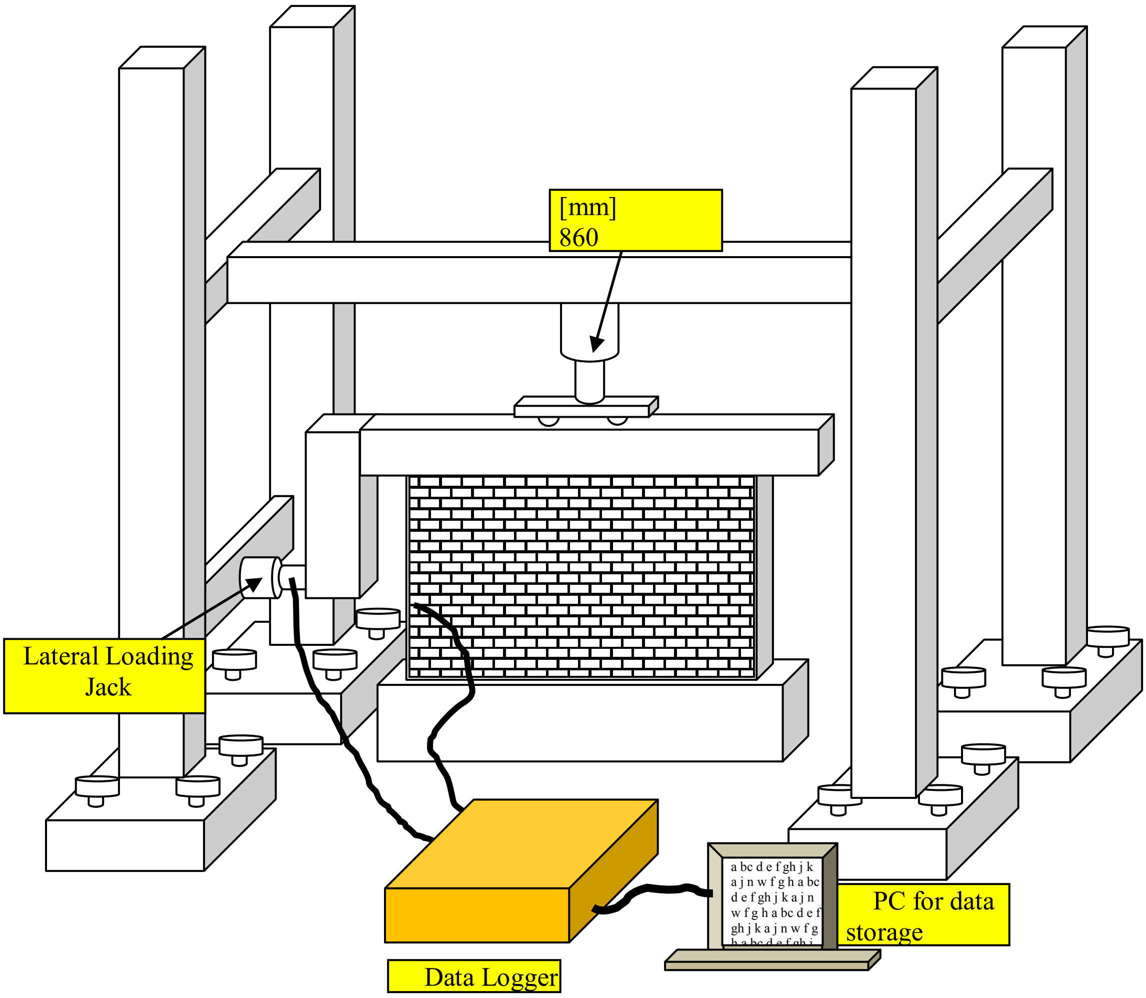
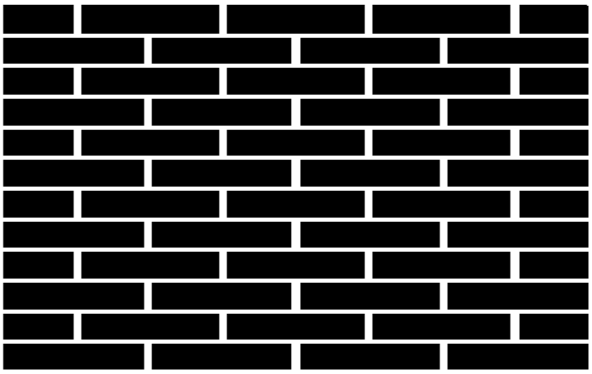
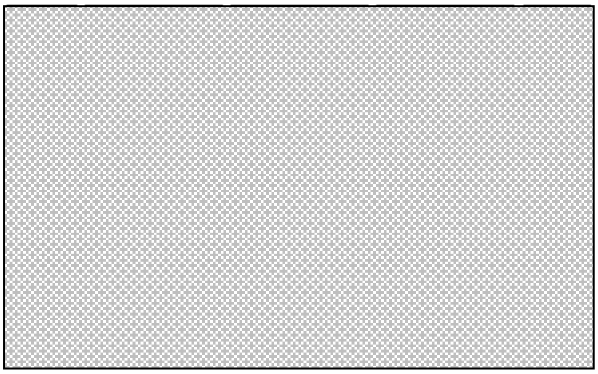
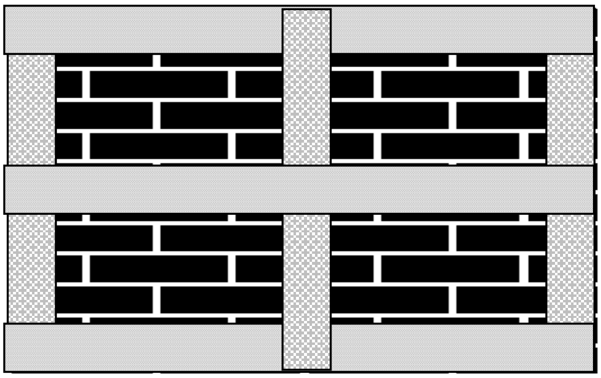


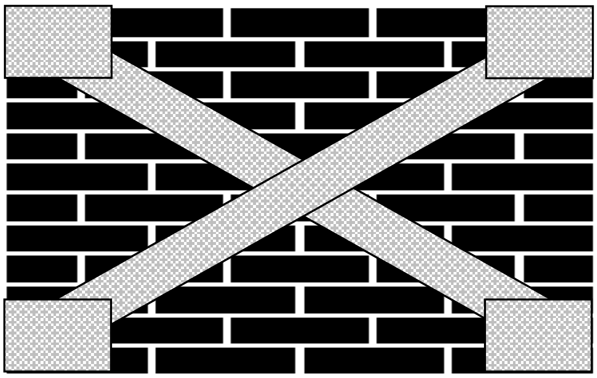
Figure 2: Schematic view of existing facilities in our laboratory for testing shear walls.





(a) Control wall (RW1 & RW2) (b) 100% coverage (PW1 & CW1)





(c) 40% coverage (PW2 & CW2) (d) 20% coverage (PW3 & CW3)

Figure 3: Different Wrapping technique of masonry wall by FRP

with epoxy resin.

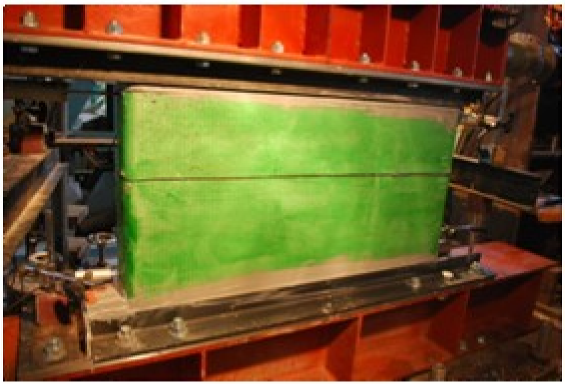


Figure 4(a). Anchorage system for shear wall PW1

Figure 4(b). Anchorage system for shear wall PW2

Table 1: Properties of different shear wall specimens.

Wall Designations	**RW1**	**RW2**	**PW1**	**PW2**	**PW3**	**CW1**	**CW2**	**CW3**
Vertical compression, V_c (Tons)	10	10	10	10	10	10	10	10
Brcik strength, f_c^b (MPa)	17	17	17	17	17	17	17	17
Mortar strength, f_c^m (MPa)	12	12	12	12	12	12	12	12
PET(%)	-	-	100	40	20	-	-	-
CFRP (%)	-	-	-	-	-	100	40	20
L/H	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25

Table 2: properties of FRPs and resign

FRP properties	PET 600	CFRP FTS-C1-20	***Properties***	Resin (D-90 R)	Putty (T-30)
Fracture Strength (N/mm^2)	740	3400	Compressive strength, MPa	85	-
Elastic Modulus (kN/mm^2)	10±1	245	Flexural strength, MPa	65	-
Elongation (%)	10±1	1.5	Tensile strength, MPa	45	20.0
Thickness (mm)	0.841	0.111	Compressive modulus, MPa	2500	-
*Width (mm)	300	250	Tensile modulus, MPa	1560	-
			Elongation after fracture, %	28%	-

3. TEST RESULTS

The whole experimental work is underway and the results are expected to come very soon. Here the author likes to present the test results previously done in Japan for wall with PET sheet only. For control masonry wall (RW1) in shear , flexural failure is very much detrimental as the initiation and propagation of flexural crack at the wall heel happens all of a sudden and the load reduces abruptly to almost zero. But it was interesting to observe in the experimental process that the flexural crack was arrested in the halfway of the wall length and no further propagation of the crack was noticed (see Figure 5). The wall internally adjusted itself to carry out more load until its failure in crushing at the wall toe. During the secondary course of loading, another crack at the mid-height of wall heel started to appear and advanced towards to center of the wall and went down in stepped fashion. Throughout the loading process no other visible damage elsewhere on the wall could be seen except the two long horizontal cracks that only widened in their opening with the increase of load. An out-of-plane movement of the wall was noticed at the crack plane just a little before of the total collapse of the wall by the toe crushing. Figure 6 shows a typical lateral load-deflection curve of both of the control (RW1, RW2) shear wall specimens without FRP. They had the aspect ratio of 1.2 and 1.67 respectively. The graph shows that high aspect ratio offer higher shear resistance and vice versa.

Figure 7 shows load deflection characteristics of shear walls (PW1 and PW2) strengthened with PET sheet by 100% and 40% respectively with the plot of control walls as well. andThe failure pattern of PW1 and PW2 are shown in Figure 8a & 8b respectively. As PW1 was wrapped by the FRP in both vertical and horizontal straps, damage on the wall if any was difficult to observe during the loading process. Only a visible shear crack was

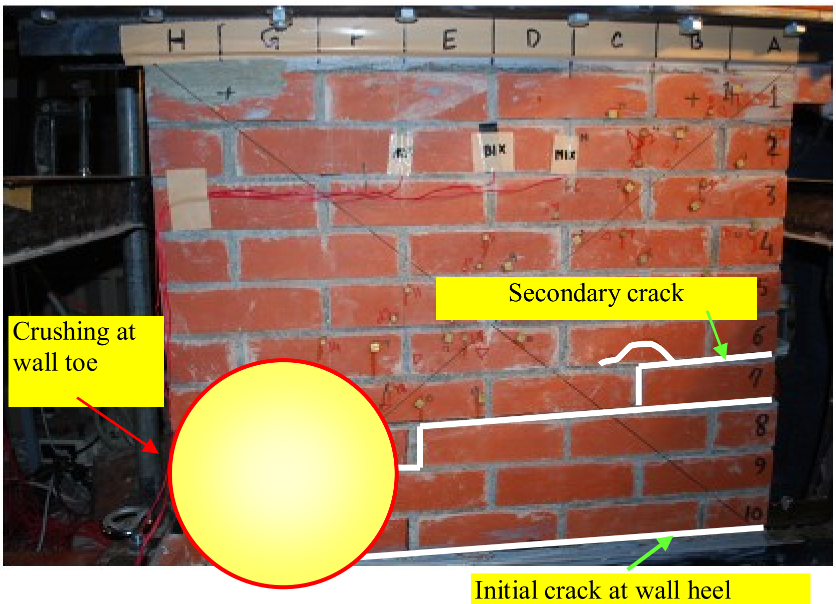


Figure 5. Failure of wall-1 due to flexural crack and toe crushing

Figure 6. Load deflection characteristic of control wall RW1 and RW2

found at the bottom of the wall on the loading side as the wall separated from the steel angle on the both sides of the wall. Even after the experiment the wall specimen was cut into small pieces, still no sign of damage or debonding of the FRP was observed. This can be said as premature failure. If the wall could be anchored with the steel beam more tightly, a higher load would be resulted. On the other hand PW2 was strengthened with FRP in both vertical and horizontal strips of 80mm width that cover about 40% of the wall gross area. Failure was mainly due to the diagonal shear crack ended with a toe crushing over a large area (see Figure 8b). No rupture of the FRP was observed even after a large deformation (drift =1/42) after the peak load.

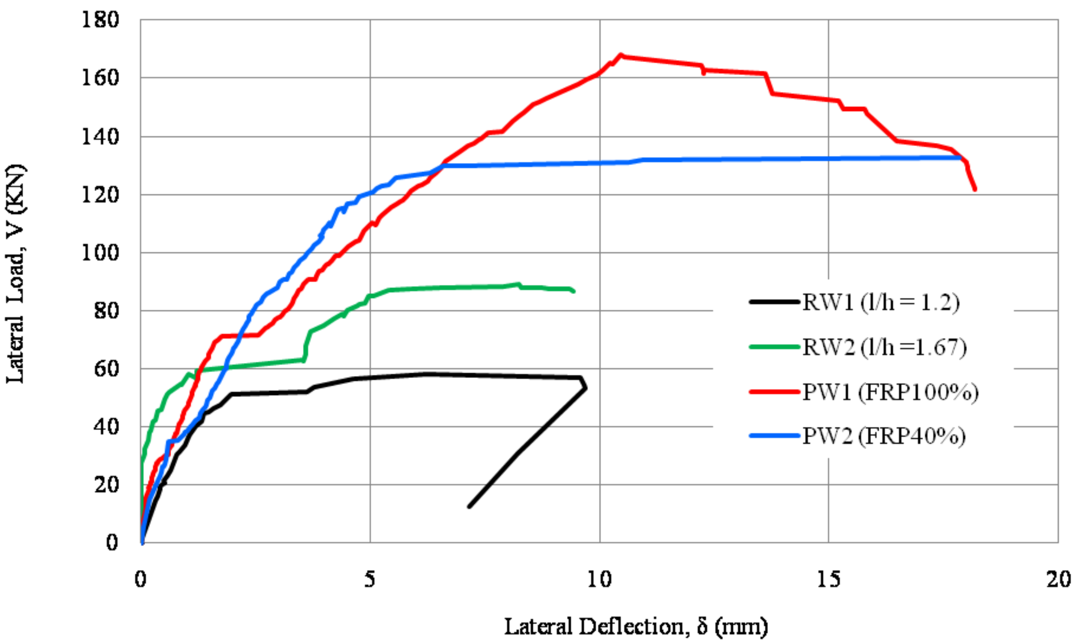


Figure 7. Load deflection characteristic of shear wall PW1 and PW2

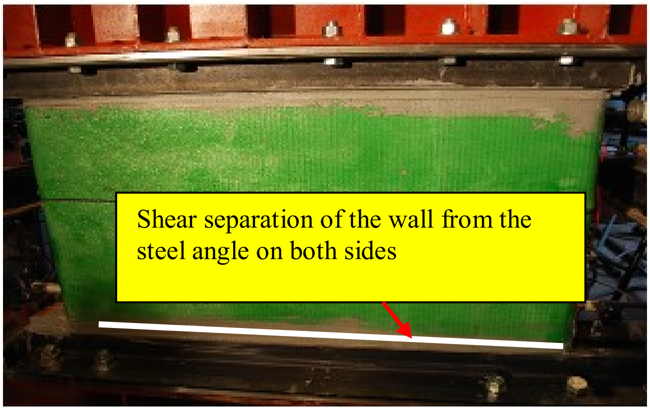


Figure 8a. Failure of PW1 due to slip

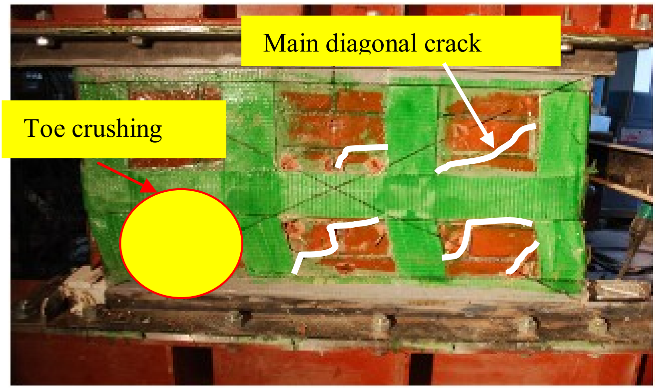


Figure 8b. Failure of PW2 due to crack along the diagonal and crushing at wall toe.

Figure 9 and 10 show the FRP strain at different location on the wall. Deformation of FRP in PW2 is higher than that in the PW1.This in turn proofs that the efficient use of FRP material will maximize the benefit, both structurally and economically. Another interesting phenomenon can be noticed from this result. It was mentioned earlier that PET fiber FRP sheet has an elongation over 10% at fracture. For the case of shear PW2 the PET fiber was strained about 25% of its capacity where as for PW1 only 6%(as in Figure 9 and 10)

4. CONCLUSION

From the experimental study we found that if the wall is fully wrapped with the FRP the shear load capacity can be increased up to 90% and if the FRP is laid over an area of 40% of the wall gross area, capacity can be enhanced as much as 46%. From numerical results they are 57% and 10% respectively. Another interesting finding from both the experiment is that the efficient use of FRP produces better structural performance in terms of ductility. This is because for brittle structure like masonry, the requirement for ductility is more crucial than the enhancement of strength alone. On this aspect, it can be concluded that minimum use of FRP material will definitely maximize the structural overall performance. But this cannot be said for CFRP or GFRP where the elongation of these materials is quite lower than the PET fiber. On this ground our selection of PET fiber as strengthening material of masonry shear wall was quite wise and proved as unique. Because PET fiber with which also have lower strength. The last but obviously not the least benefit of using PET fiber is the cost. Masonry itself is not as much costly structure as RC, so selection of a retrofitting material in such a way that goes with the masonry, is an another important parameter. In that sense, PET fiber is comparatively cheaper than the popular CFRP or GFRP. A lot of research still pending ahead on some our aspect of FRP strengthening such as anchorage technique, diagonal FRP overlay, out-of plane behavior of masonry under UDL or thrust.

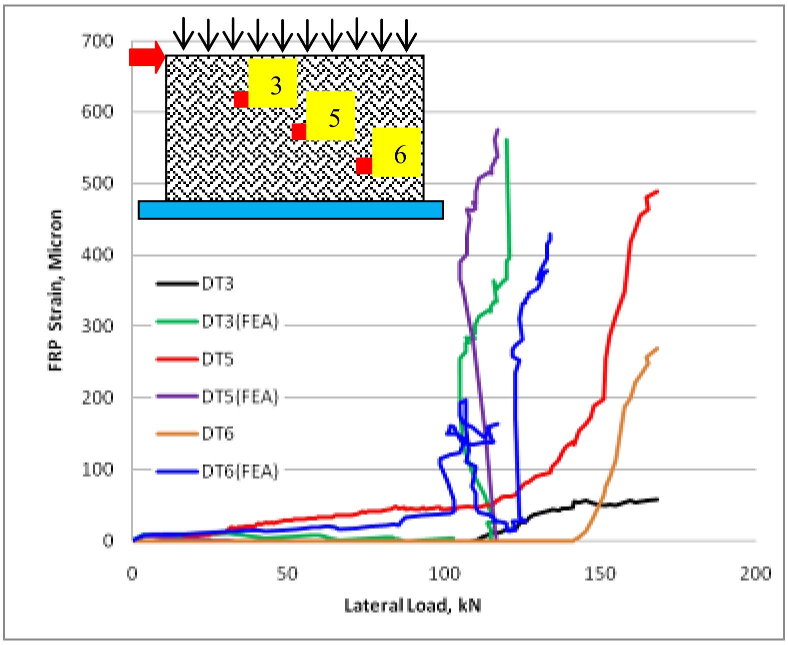


Figure 9. Comparison of numerical and experimental FRP strain for shear PW1

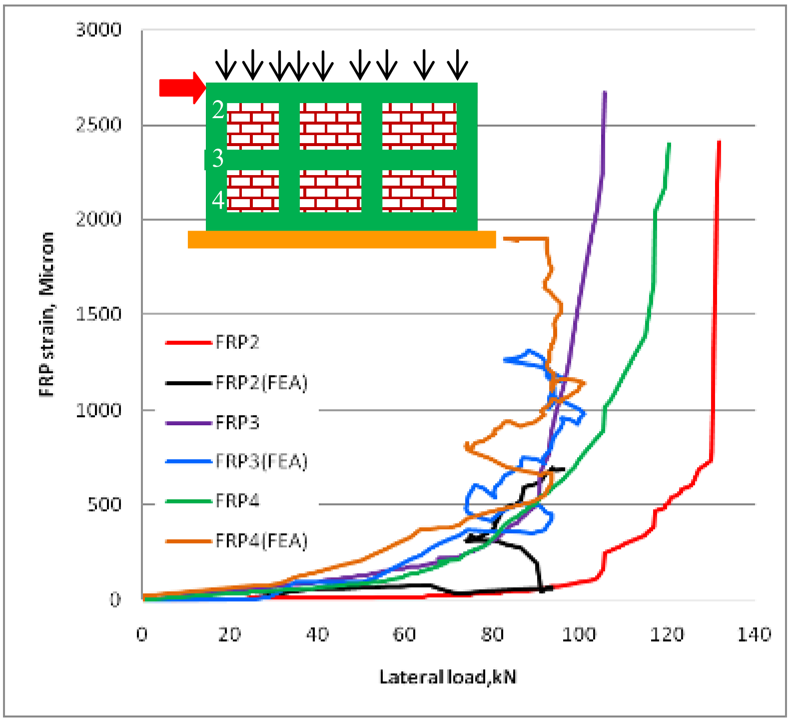


Figure 10.Comparison of numerical and experimental FRP strain for shear PW2

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PROPERTIES OF FIBER REINFORCED CONCRETE USING MILD STEEL NAIL

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1. INTRODUCTION

Fibre reinforced concrete (FRC) is defined as a composite materials made with Portland cement, aggregate, and incorporating discrete discontinuous fibres. Steel fiber reinforced concrete (SFRC) was introduced commercially into the European market in the second half of the 1970's. No standards or recommendations were available at that time which was a major obstacle for the acceptance of this new technology. Plain unreinforced concrete is a brittle material, with a low tensile strength. The role of randomly distributes discontinuous fibers is to bridge across the cracks that develop some post cracking "ductility". If the fibers are sufficiently strong, sufficiently bonded to material, it permits the FRC to carry significant stress over a relatively large strain capacity in the post cracking stage. Steel fibers can reliably inhibit cracking and improve resistance to material deterioration as a result of fatigue, impact, and shrinkage, or thermal stresses.. Initially steel fibres were mostly used as a substitute for secondary reinforcement or for crack control in less critical parts of the construction. Today steel fibres are widely used as the main and unique reinforcing for industrial floor slabs, shotcrete and prefabricated concrete products. They are also considered for structural purposes in reinforcement of slabs on piles, full replacement of the standard reinforcing cage for tunnel segments, concrete cellars, foundation slabs and shear reinforcement in prestressed elements.

Steel fibres, macro synthetic fibres, micro synthetic fibres, cellulous fibres are widespread within construction now a days. The fibers may take many shapes. Their cross sections include circular, rectangular, half-round, and irregular or varying cross sections. They may be straight or bent, and come in various lengths. A convenient numerical parameter called the aspect ratio is used to describe the geometry. This ratio is the fiber length divided by the diameter. The designer may best view fiber reinforced concrete as a concrete with increased strain capacity, impact resistance, energy absorption, and tensile strength. However, the increase in these properties will vary from substantial to nil depending on the quantity and type of fibers used; in addition, the properties will not increase at the same rate as fibers are added. Steel fibers are normally high strength steel with a tensile strength varies from 1000 to 2000 Mpa and are relatively expensice than mild steel (MS). In this study a low cost MS nail has been used as non standard steel fiber. The objective is two fold; firstly, to keep the cost of the fiber resonably low and secondly, to compare some properties of concrete mixed with mild steel nail with conventional SFRC.

2. METHODOLOGY

An understanding of the mechanical properties of SFRC and their variation with fiber type and amount is an important aspect of successful design. These are compressive strength test, tensile strength test, and flexural strength test. For these tests several number of cylinders and beams are prepared. The cylinders of 6 inch ∅ and 12 inch high is used for both compressive and tensile strength test. A 24 inch long, 6 inch wide and 6 inch high beam is prepared for flexural strength test. There are a number of different factors that can influence the variability of results, namely:

- Fibre type and dosage
- Ratio of fibre length to max aggregate size
- Batching and mixing of concrete

2.1 Compressive Strength Test

The effect of steel fibers on the compressive strength of concrete is a variable. Experimental results show that the increase of compressive strength is zero to 23% for different l/d ratio and % of fiber. In this study, MS nails of three different length and diameter are used. They are 14.3mm, 23.1mm, 31.1mm nails. Their diameters are 1.28mm, 1.52mm, and 2.06mm respectively. And *l/d* ratios are 11.17, 15.20, and 15.10 respectively. Each nail is used in four different proportions. They are 0.5%, 1.0%, 1.5%, 2.0%.

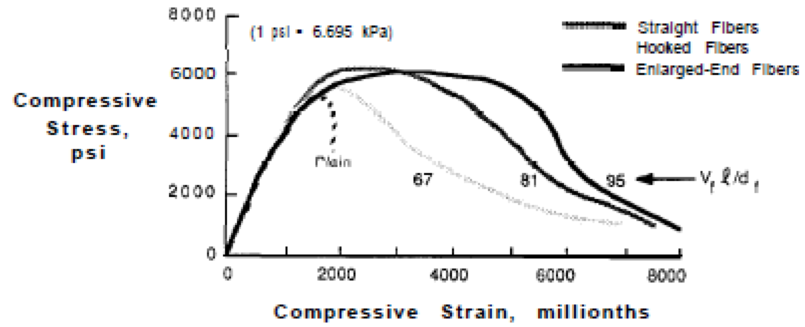

Figure 1. Stress-strain curves for steel fiber reinforced concrete in compression, $^3/_8$ -in. (9.5-mm) aggregate mixtures (Shah 1978)

2.2 Tensile Strength Test

No standard test exists to determine the stress-strain curve of fiber reinforced concrete in direct tension. The observed curve depends on the size of the specimen, method of testing, stiffness of the testing machine, gage length, and whether single or multiple cracking occurs within the gage length used. typical examples of stress-strain curves (with strains measured from strain gages)
for steel fiber reinforced mortar are shown in Fig. 2 (Shah et al. 1978). The ascending part of the curve up to first cracking is similar to that of unreinforced mortar. The descending part depends on the fiber reinforcing parameters, notably fiber shape, fiber amount and aspect ratio. Diameter, length and % of fiber are used in tension test are same as that in compression test.

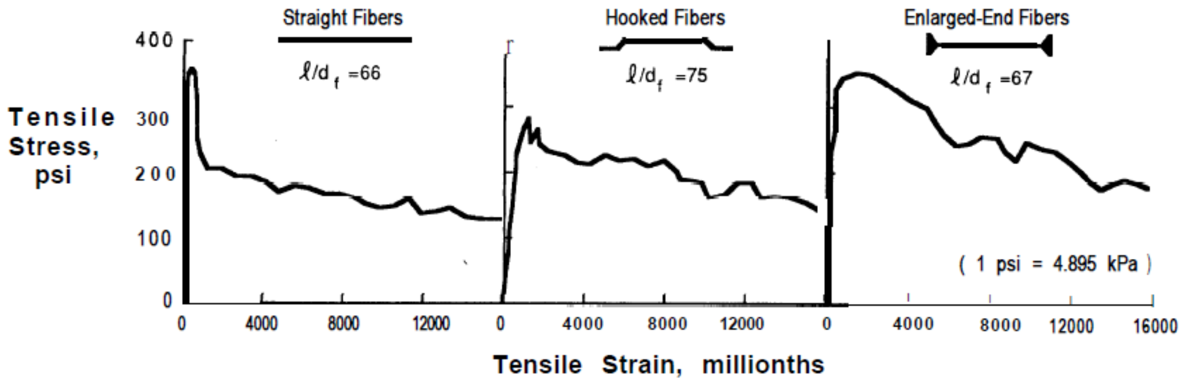

Figure 2. Stress-strain curves for steel fiber reinforced mortars in tension (1.73 percent fibers by volume) (Shah 1978)

2.3 Flexural Strength Test

The influence of steel fibers on flexural strength of concrete and mortar is much greater than for direct tension and compression. Two flexural strength values are commonly reported. One, termed the first-crack flexural strength, corresponds to the load at which the load-deformation curve departs from linearity and the other corresponds to the maximum load achieved, commonly called the ultimate flexural strength or modulus of rupture.

3. RESULTS

Table: Compressive strength of concrete prepared by using nails of different length and percentages

	Strength(KN)			
Percentage (%) / Length of nail(mm)	0.5	1.0	1.5	2.0
14.3	470	480	512	538
23.1	446	480	506	512
31.1	556	542	538	502

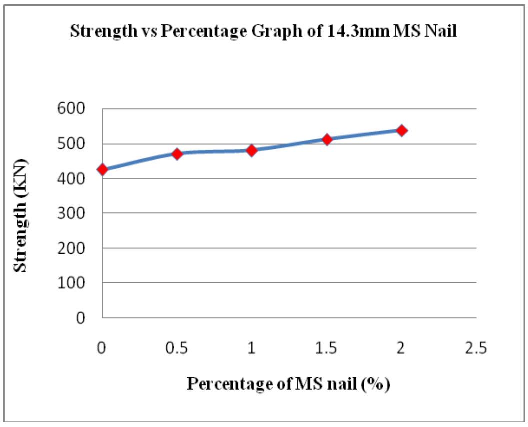

Figure 3:Strength vs Percentage Graph of 14.3mm Nail

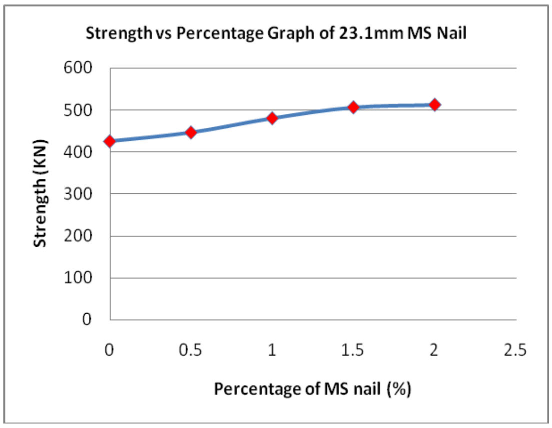

Figure 4:Strength vs Percentage Graph of 23.1mm Nail

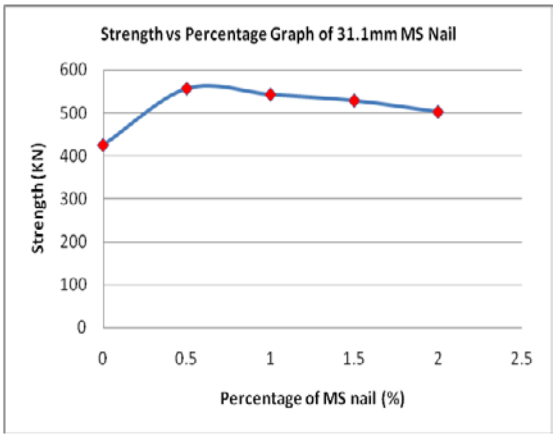

Figure 5:Strength vs Percentage Graph of 31.1 mm Nail

4. CONCLUSIONS

This test shows that using nail is suitable for increasing concrete strength.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

OPTIMUM DESIGN CRITERIA OF TALL BUILDING FOR DIFFERENT SUBSOIL CONDITIONS CONSIDERING BASEMENTS

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ABSTRACT

Now a day's earthquake is the major concern of disaster of the world. Bangladesh is also in the risk zone. As a developing country like Bangladesh, for industrial and commercial growth it is necessary to build high rise building structures. For the survival of the structures it is necessary to withstand the maximum stress that may induce due to earthquake. Normally high rise building is constructed with basement. As building is fixed at the ground level the basement of the building may not be included in the analysis. However, the basement may introduce flexibility to the structure resulting in larger lateral displacements and longer vibration periods. The seismic loads applied to a building structure will affect the member forces in the basement. In this study , different typical types of framed structures with or without basement was subjected to static lateral loads by using commercial software and displacement of the roof was measured for different structure.

1. INTRODUCTION

Recently, most of the high-rise buildings may have basement used as parking lots or shopping malls etc. In general, it is commonly assumed that the building is fixed at the ground level in the analysis. In general, only gravity loads are considered in designing the basement structure without the effect of lateral forces as earthquake loads applied to the super structure such. But the seismic loads applied to the super structure will affect the member forces in the basement structure. Recent earthquakes with low to moderate magnitude very close to Dhaka are certainly indications of its earthquake source and vulnerability. The time history analysis procedure cannot be applied by using composite, envelope motions, as can be done for the response spectrum procedure. Rather, multiple time histories that together provide a response that envelops the expected motion must be used. A response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, that are forced into motion by the same base vibration or shock. Seismometers are instruments that measure motions of the ground, including those of seismic waves generated by earthquakes, volcanic eruptions, and other seismic sources. Records of seismic waves allow seismologists to map the interior of the Earth, and locate and measure the size of these different sources.

In engineering, the damping ratio is a dimensionless measure describing how oscillations in a system decay after a disturbance. A mass suspended from a spring, for example, might, if pulled and released, bounce up and down. On each bounce, the system is "trying" to return to its equilibrium position, but overshoots it. The damping ratio provides a mathematical means of expressing the level of damping in a system relative to critical damping.

Response spectra are very useful tools of earthquake engineering for analyzing the performance of structures and equipment in earthquakes, since many behave principally as simple oscillators (also known as single degree of freedom systems). A degree of freedom is an independent physical parameter, often called a dimension, in the formal description of the state of a physical system. The set of all dimensions of a system is known as a phase space. The peak response of the building can be estimated by reading the value from the ground response spectrum for the appropriate frequency. In most building codes in seismic regions, this value forms the basis for calculating the forces that a structure must be designed to resist.

A full time history will give the response of a structure over time during and after the application of a load. To find the full time history of a structure's response.

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building (either calculated or defined by the building code).

A dynamic load can have a significantly larger effect than a static load of the same magnitude due to the structure's inability to respond quickly to the loading (by deflecting). The increase in the effect of a dynamic load is given by the dynamic amplification factor (DAF):

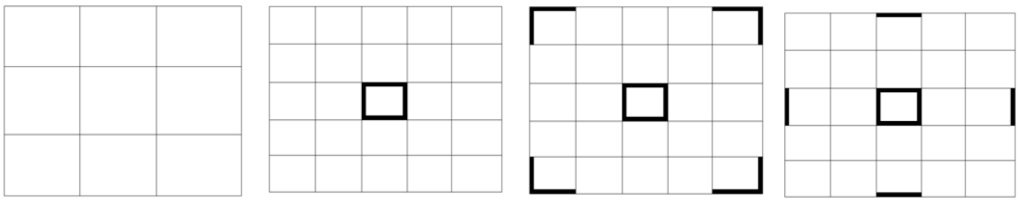
$$DAF = \frac{u_{max}}{u_{static}}$$

Where, u is the deflection of the structure due to the applied load.
Selection of an appropriate foundation system is dependent upon many factors. These factors may include:

- soil conditions
- groundwater conditions
- surface conditions
- structural loads
- structural function (i.e. basement, cold structure, etc)
- economy

2. METHODOLOGY

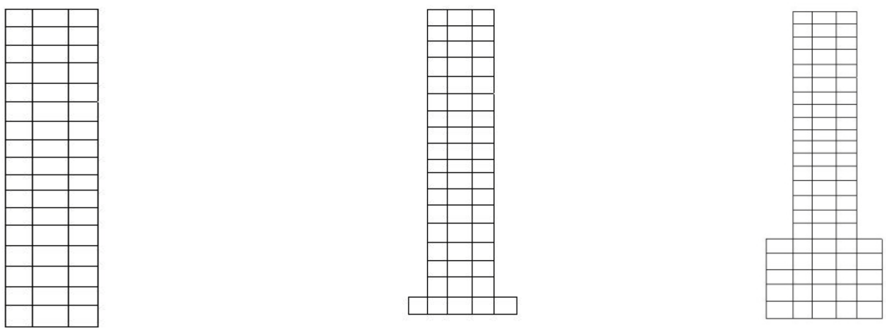
A typical framed structure (structure type A) and a framed structure with a reinforced concrete core (structure type B) were used as example structures to investigate the seismic response of high-rise buildings with basement. The structure type A was modeled as the structure A0 which is fixed on the ground level as shown in Fig.1 and the structure A5 which has five stories in the basement as shown in Fig.2. The structures A1, A2, A3 and A4 have 1, 2, 3 and 4 stories in the basement, respectively. Similarly, the structure type B was also modeled as B0, B1, B2, B3, B4 and B5 as shown in Figs. 1 and 2 according to the number of stories in the basement. All example structures have 20 stories above ground and the structural behavior was investigated by varying the number of stories in the basement from 1 to 5.



A **B** **C** **D**

Figure 1: Plan view of typically framed structures.

Above plan is designated as A, B, C & D which would be implying in software to get result. Below The structures A1, A2, A3 and A4 have 1, 2, 3 and 4 stories in the basement.



A0 A1 A5

Figure 2: Elevation of typically framed structures.

The fundamental building period is simply the inverse of the building frequency at the lowest harmonic - easy right? Basically, every system has a set of frequencies in which it "wants" to vibrate when set in motion by some sort of disturbance (in building design, typically a seismic or wind event) based on the system's mass and

stiffness characteristics. The shortest frequency is known as the natural frequency. The inverse of frequency is the period of the system, and more specifically, the inverse of the natural frequency is the fundamental period.
In seismic design, the closer the frequency of an earthquake is to the natural frequency of a building, the more energy is introduced into the building structure.
Time period for maximum spectra, maximum spectral ordinate coefficients a, b of the equation $S=aT^b$ which has been used to modify response spectra for simulated earthquake. It is observed that the coefficient b which represents the rate of displacement is always greater than 1.

2.1 Seismic record collection:

Time history analysis is performed by collected seismic record which occurred in 2009 natore. In the Natore earthquake record the peak ground acceleration is 2.43 cm per Second Square. Seismic zone coefficient for Khulna (Zone I) is 0.075 (BNBC, 1993). So, the maximum ground acceleration for Khulna comes accordingly following the code is 0.075 g ≈73.6 cm per Second Square which is 32times greater than that of Natore data. So, all values are multiplied by scaling factor 31. Seismometers are instruments that measure motions of the ground, including those of seismic waves generated by earthquakes, volcanic eruptions, and other seismic sources. Records of seismic waves allow seismologists to map the interior of the Earth, and locate and measure the size of these different sources. According to building codes, earthquake-resistant structures are meant to withstand the largest earthquake of a certain probability that is likely to occur at their location. This means the loss of life should be minimized by preventing collapse of the buildings for rare earthquakes while the loss of functionality should be limited for more frequent ones.

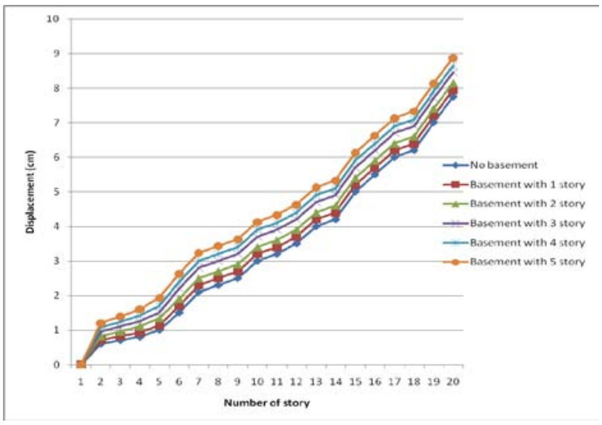


Figure 3: Variation of story displacement

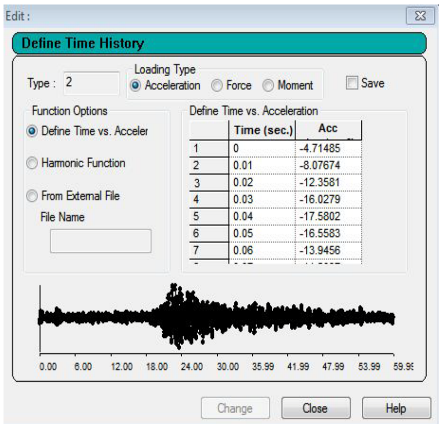


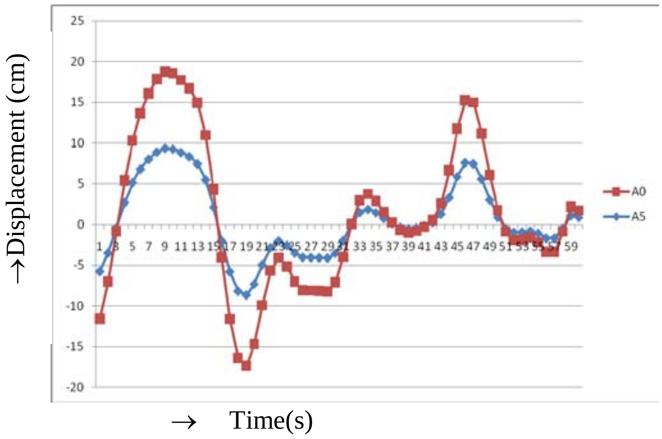
Figure 4: Seismic record data

2.2 Equivalent Static Analysis Result:

The roof top displacements from the equivalent static analysis are shown in figure3. As the number of stories in the basement increases, the rotation at the bottom of columns in the first story Increases because of the flexibility introduced by the basement structure. Due to this phenomenon, the lateral stiffness decreases resulting in the increase of the lateral displacements. The variation in story displacement indicates that it is 5 story basement produce maximum displacement in all story which can effect in the effective stiffness analysis of the structure.

2.3 Time History Analysis Result:

This result is for floor plan "type A0" by using STAAD Pro. Here shows that variation of displacement with respect to basement. The difference in the displacement time histories of the structure type A was less significant than that of the structure type B as illustrated in Fig. 5 as could be expected from the differences in the natural periods of vibration of both structures. In this figure shows that value of maximum basement (5 story) produces maximum displacement. Which obiviously makes disturbance in the earhquake analysis.



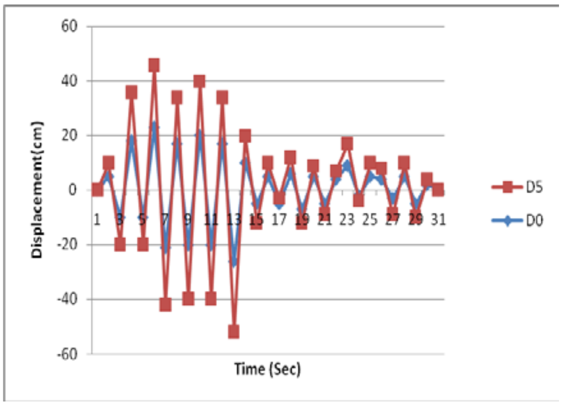


Figure: 2 Displacement with respect to time. **Figure 6:** Variation in roof top displacement

2.4 Analysis scheme

For spectral analysis purpose, using different acceleration spectra, only one direction, that is, short direction of the building have been performed by locking the global degree of freedom of other direction. As spectral acceleration was applied towards the short dimension of the building, the mode shapes transverse to the direction and torsional mode shapes, should have no mass participation factor to the final analysis. To validate this, two analyses have been performed taking 16 and 10 storey special moment resisting conrete frames factors for both the analyses. Taking this fact into account to minimise the time of computer run, short direction analysis scheme have been adopted for further analyses, which is expected to produce equally good accuracy of the analytical analysis.

Seismic coefficients are intended to define the minimum spectral ordinates to be used in design. The terms CA and CV correspond to constant-acceleration and constant-velocity regions of the response spectrum respectively. The seismic coefficient value (UBC, 1994) of CA is illustrated in Table 1 and CV at Table 2 as well. The value of CA (from Table 1 for Z = 0.15 and soil profile type S3) is found as 0.22 and the value of CV (from Table 2 for Z =0.15 and soil Profile Type S3) is found as 0.32. So, Ta = 1/33 = 0.03 s, Tc = 0.58 and Tb = 0.11 s as per Taking the values for these points from the uneven curve and free hand smoothing, acceleration response spectrum has been found which is shown in Figure 8.
Now for soil type III, the values of a and b can be obtained 1.48 and -0.98 respectively (Ansary and Noor, 2000). From the Equation 34, for Tc = Tsm = 0.58, Sa = 2.503. This value resembles to the generated spectral value corresponding to same time period 0.58. As for Tb up to Tsm, spectral ordinate is constant so Sa = 2.503 for also Tb = 0.11 s. Selected periods for changing spectral values also mostly resemble to the period selection described as Ta = 1/33, Tb = 1/8, Tc = 1/2 s (Lee and Kanamori, 2002) and Tb = 0.10 and Tc = 0.60 s (Kappos, 2002) and accordingly maximum value of response spectrum is well similar to 2.5* üg. The initial value of spectral acceleration has been taken as at T0. For T>Tc, Sa has been derived from.

3. RESULTS

This result is for floor plan "type A0" by using STAAD Pro. Here shows that variation of displacement with respect to basement. The effect of the basement on the seismic response of high-rise buildings and the effect of the lateral forces applied to the superstructure on the member forces in the basement were investigated in this study. Lateral loads affect not only the response of the super structure but also that of the basement structure.
Therefore, seismic loads as well as gravity loads should be considered in the analysis of a high-rise building structure for the design of the basement structure. While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system which will vibrate perceptibly under frequently occurring loads such as wind loads or braking loads.
Although great progress has been made regarding since seismic design was made mandatory by various building codes, it is still not completely understood. So, structural engineers have been giving more and more attention to the design of structures for earthquake resistance.
In this study response spectra for real and simulated earthquakes have been developed. Further these spectra modified and have been found quite consistent. Efforts have been here to 5 percent damping. Value of maximum spectral ordinate coefficient a, b for Modified simulated Response Spectra for different site Category:

Soil Category	Time period (s)	Maximum value of ordinate	Coefficient a	Coefficient b
I	0.16	3.18	0.74	-0.90
II	0.21	3.30	0.95	-0.93
III	0.35	3.37	1.48	-0.98
IV	1.10	2.83	3.74	-1.07

The difference in the natural periods will result in the difference in the seismic response. In general, the natural periods of a structure considering the effects of the basement are longer than that of the structure without basement structure. The accelerations of the structure with a basement tend to be smaller, because the natural periods of the structure types A5 and B5 are longer than those of the structures A0 and B0, respectively. Therefore, if the basement structure is included in the analytical model, the seismic loads in the response spectrum analysis become relatively small. Even though the difference in the periods is small, the difference in the spectral acceleration becomes larger when the period is relatively short because the slope of the response spectrum becomes steeper. Equivalent static analyses of structures A5 and B5 were performed using three different models.
Seismic response of high-rise building structures with basement may be significantly different from those of the structures without basement. Thus, seismic response of high-rise building structures in the basement can be predicted by including the basement in the analytical model. An accurate prediction of the shear force in the basement is very important in the seismic analysis of high-rise building structures with basement.

4. CONCLUSIONS

In fact Bangladesh lacks heavily on seismic instruments and enough data of earthquake records is not available especially of Dhaka. The effect of the basement on the seismic response of high-rise buildings and the effect of the lateral forces applied to the superstructure on the member forces in the basement were investigated in this study .So nearby natural earthquake record has been considered to develop acceleration time history for this region. The generated response spectra and time history may be used in dynamic analysis of structures in Dhaka. For other regions of Bangladesh, the generated response spectrum may be used adjusting the zoning condition. It is imperative to install suitable number of seismic stations so that in future, spectra based on site specific real earthquake records for allover Bangladesh can be developed. Due to the fact that Bangladesh lacks heavily simulated earthquakes had to be generated to arrive at site specific response spectra suitable for dynamic analysis. Whereas the spectra generated here may be used in dynamic analysis of structures, it is imperative to install suitable. It is clear from above discussion that maximum amplitude of the acceleration spectra decreased as the soil type changed from soft to rock. For larger periods, it is evident that soft soil spectral acceleration is greater than rock spectral acceleration. It is also observed that largest amplification occur near the natural time period of the soil. It has also been understand that stiffness of rock was faster than stiff soil and so was the case as the soil became softer. Rate of softer is faster for the earthquakes taken in this study. When designing the basement structures, it is assumed that the seismic loads do not affect the basement structure and only gravity loads are considered in the analysis while the member forces in the basement structure may be significant.

ACKNOWLEDGEMENT

The author gratefully acknowledge the effort by Khulna University of Engineering & Technology as well as the ICCESD (International Conference on Civil Engineering for Sustainable Development) and also very much thankful to my supervisor teacher Md. Atatur Rahman.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EXPERIMENTAL INVESTIGATION OF SHEAR STRENGTH FOR BRICK-MORTAR INTERFACE FOR DIFFERENT BRICK STRENGTH

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ABSTRACT

This paper presents a study on the shear load-displacement behavior of horizontal joints in unreinforced brick masonry subjected to constant compression. In general, under static shear loading, masonry joints show a peak shear stress followed by a residual shear strength. To investigate these aspects in more detail, laboratory tests were conducted on masonry triplet specimens using different types of mortar and brick. The investigation between the authors and previous tests results show that normal compressive stresses acting on the interface, as well as the interface mortar strength, affect the peak shear stress and the residual strength in a rather similar way. The cohesion and the internal friction angle, i.e. the two parameters required by the Mohr-Coulomb criterion, are then derived from a linear regression of the test results. The effect of brick strength on masonry shear is still under investigation.

Keywords: *masonry, triplet test, interface constitutive law, peak and residual shear stresses.*

1. INTRODUCTION

Shear failure is the dominant mode of failure observed in many masonry buildings subjected to lateral loading due to earthquakes, wind (in tall and slender structures), support settlements or unsymmetrical vertical loading. Lateral loading can produce both diagonal cracking failures, and shear failures of the horizontal joints. The joint resistance is of particular concern in the analysis of load-bearing unreinforced masonry structures, that are rather common among older buildings in many countries in the world. The shear generally acts in combination with compression, which is caused by the self-weight and floor loads. Confinement by, for instance, structural frames to in-fill walls may also lead to shear compression. The present state of knowledge concerning shear strength and shear load-displacement behavior of masonry is far less advanced than that concerning masonry behavior in compression, even though shear failure is an important, often governing mode of failure in many masonry buildings (Van Zijl 2004). This lack of understanding is reflected by the low values of shear resistance allowed by the present U.S. building codes (ASCE 31-02). Information on the post-peak behavior, and on the deformations associated with pre-peak and post-peak responses are also lacking. The previous studies on joint shear behavior, while providing insight into some of the parameters influencing the shear strength, do not, in general, provide the detailed information related to the constitutive behavior which would be required to set-up analytical models for simulating the structural response under different loading conditions. Such a model will require definitions of: (1) shear stiffness for initial loading states; (2) peak and residual stresses; and (3) the effect of materials properties and normal loads on shear strength/stiffness and dilatancy.

This paper examines the shear failure mode occurring in horizontal joints, and the shear stress-slip behavior of unreinforced brick masonry under static loading. Nonstandard tests were conducted on four series of masonry samples; triplet shear specimens (Fig. 1d) were used, in order to study the effect of brick and mortar strength on joint shear behavior. Four types of brick were used for the specimen preparation. 18 MPa mortar was used for all the specimens.

2. METHODOLOGY

In this study, laboratory tests were conducted on four series of masonry samples using triplet shear specimens (Fig. 1(d)) in order to study the effect of brick and mortar strength and other properties on bed joint shear behavior.

2.1 Experimental program

The materials used in the construction of the triplet shear test specimens included four types mortar (E, M, S, N) and four types of brick with different strengths, namely as AFIL,J.C.,KBI, JBP. All bricks used were inundated in water the day before the fabrication of the tested specimen assembly. They were then dried under normal laboratory conditions for at least 1 day prior to use to ensure 80% saturation according to ASTM. Table 1 summarizes the results for the four types of brick used in the investigation and table 2 gives the properties of four types of mortar used for this test specimens.

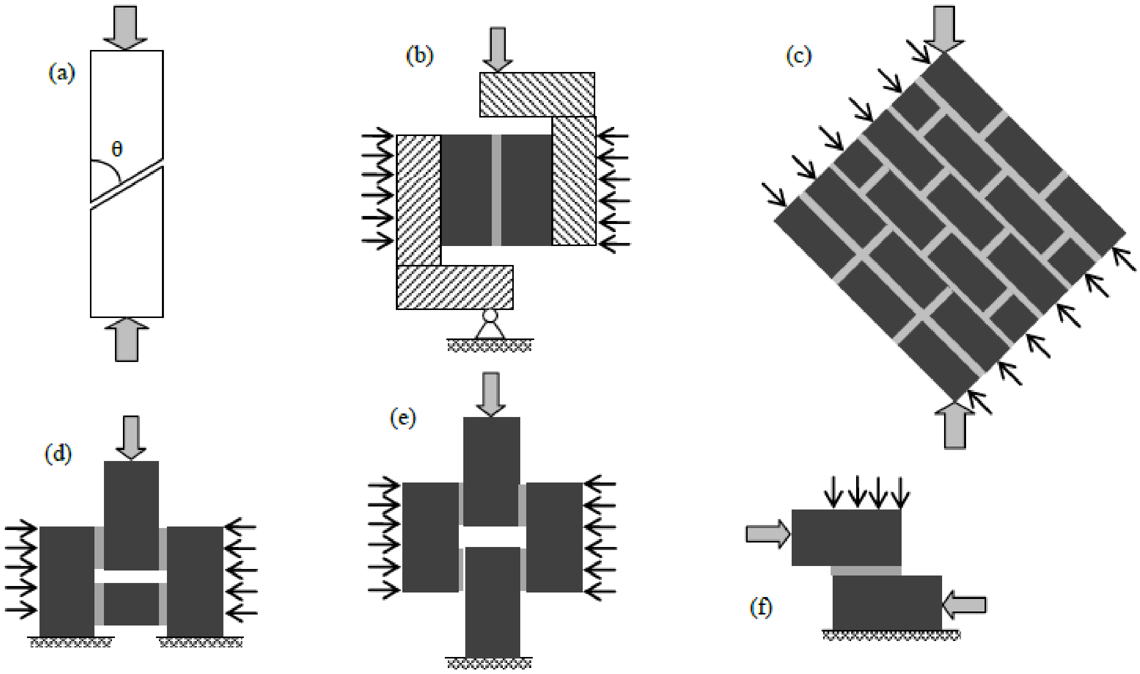


Figure 1: Different type of shear test specimens: (a) Nuss shear test(1978); (b) van der Pluijm test(1993); (c) Diagonal tension test; (d) Triplet test; (e) Meli test(1973); and (f) Direct shear test

Table1. Properties of bricks

Brick Types	Average Compressive strength (MPa)	Average Absorption capacity (%)
AFIL	34	14.3
J.C.	32	20.5
J.B.P.	16	17.6
K.B.I.	10	27.3

2.2 Specimen Preparation

The specimens were constructed with two full bricks and two half bricks bonded together by a 10 mm rectangular mortar joint as shown in Fig. 2. The first brick was placed on the ground and a timber block that was thicker than the brick by 10 mm was placed at head on position. More than the needed amount of mortar was placed on the top face of the brick with a trowel. The one half brick was then placed in such a way that some part (15mm) of it is extruded from the bottom brick and rested on timber block. Another half brick was then placed 15 mm apart from the half brick. The second brick was then placed on the two half bricks and tapped with a wooden mallet and leveled in two directions with a sprite level to create a 10- mm-thick mortar joint.
The excess mortar squeezed to the sides was removed with a trowel, and the sides of the mortar joint were flattened level with the bricks from all sides. The timber block was then removed and the specimen was left in place to allow the mortar to gain in strength. After the 5 days of initial curing, the specimens were then left for

an additional 23 days to cure under ambient conditions in the laboratory before testing at 28 days from construction.

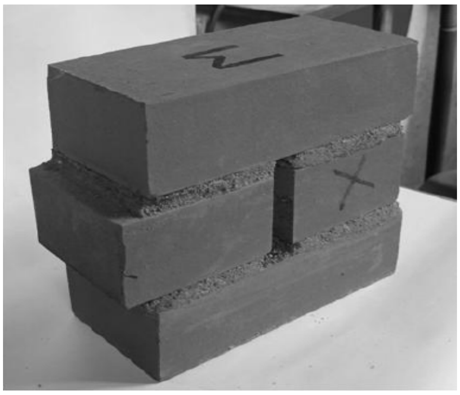


Figure 2: Triplet prism specimen for shear

Table 2. Specification of mortar used in triplet shear test specimen

Mortar types	Cement : Sand (by volume)	Water/Cement (by weight)	Compressive strength (MPa)	Splitting tensile strength (MPa)	Young's modulus (GPa)	Poisson's ratio ν	Flow (mm)
E	1 : 5.75	0.55	28.5	3.0	26.0	0.186	154
M	1 : 6.75	0.70	20.0	1.7	19.3	0.156	140
S	1 : 8.50	0.87	12.5	1.5	15.7	0.200	186
N	1 : 4.50	1.00	10.0	0.9	14.5	0.188	138

2.3 Instrumentation and Test Setup

Six specimens of each brick type were constructed and cured for 28 days. Before testing, the length of the mortar joint was recorded. Two steel plates were attached on both sides of the specimen and kept in position with four bolts. Uniform compressive pressure was exerted on the specimen using these bolts. Two mechanical strain gages were attached with each of the bolts to record the level of pressure that is being applied by the tightening of these bolts and calculated from the stress-strain diagram of the steel bolt, which was calibrated before the test.

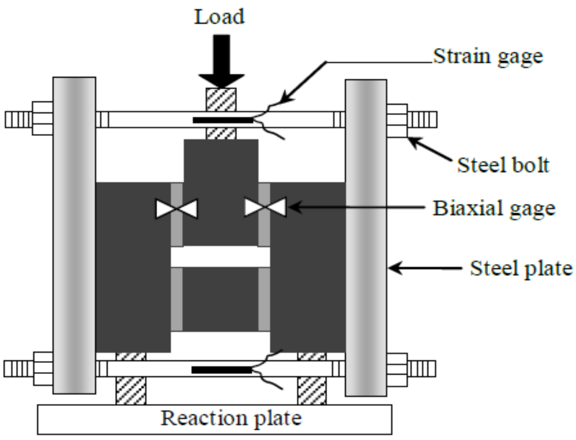


Figure 3: Instrumentation of the shear specimen

When the expected level of pressure was reached, the specimen was ready for the shear test. Two biaxial strain gages were attached on the opposite sides of the specimen along the bed joints to record the shear displacement.

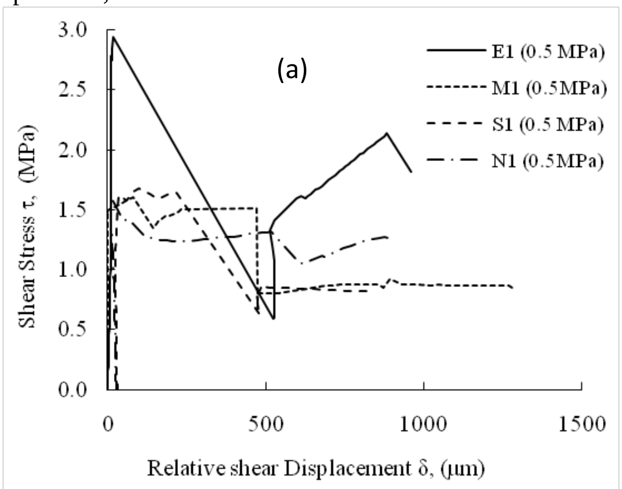
The specimen was designed in such a way that the applied load will be transferred through the upper half brick as shear and the pre-compression will be carried out by both the top and the bottom half bricks. The area of shear and compression was calculated accordingly. Figure 3 shows the loading and support arrangements used for testing of the shear specimens.

2.4 Testing and Measurements

For each type of brick, six specimens were tested with a constant pre-compression of 0.5 MPa and 1.0 MPa respectively, resulting in a total of twenty four specimens. All four bolts were tightened simultaneously and the corresponding strain was recorded through a data logger. Once the level of strain meets the required stress level, the specimen was transferred under the actuator of a Servo-control testing machine to apply shear load as compression as shown in Figure 6. The maximum loading capacity of the vertical actuator is 50 kN. The shear load was applied with a rate of 0.05 mm/sec and the corresponding shear displacement was captured with the help of biaxial gages attached to the opposite side of the specimen and was recorded through the data logger. Throughout the loading process the pre-compression was kept almost constant. A little fluctuation of the compressive pressure may be occurred due to the fact that when two rough surfaces of brick and mortar slide over each other, a phenomenon of dilatancy occurs, which causes an increase in volume and exerts pressure on steel plate. This excess pressure is somehow contributes to the overestimation of the shearing strength of the interface, but for the simplicity of the analysis the dilatancy effect was neglected in numerical model.

3. TEST RESULTS AND DISCUSSION

It is quite obvious that the ultimate shear strength increases with increasing confining pressure normal to the shearing surface. However, this is not the only governing factor that influences the shear strength of the brick-mortar interface. The other factors are 1) characteristics of bond between mortar and brick; 2) characteristics of brick and mortar; 3) coefficient of friction between the two sliding surfaces; and 4) the overall quality of the joint. Since, during the fabrication of the specimens, an overall uniformity was difficult to be attained, some inconsistencies are inevitable. Fig. 4a and Fig. 4b show the nominal shear stress as a function of the shear displacement for all the tested specimens. According to the figures, it is plain to see that just before the peak shear stress the stiffness is very high, with very little shear deformation. The interlocking between the grains of the brick and the mortar under confining pressure, is the main reason for the high stiffness of the shear load-displacement relationship. There is a barely detectable hardening phase, but just for a short range before the peak load. As the imposed shear displacement overcomes the interlock between brick and mortar, a phenomenon of volume increase (dilatancy) takes place, and give rise to a much higher strength than expected. In the present study, the effect of dilatancy was not considered, due to the fact that at a confining pressure higher than 0. 5 MPa, this dilatancy becomes marginally small, the effect of internal friction dominates over dilatancy (Armaanidis 1998), and therefore makes it possible to evaluate the shear strength by mean of the Mohr-Coulomb criterion. Moreover, the high confining pressure restricts the upward dilatant deformation of the specimen, and turns it into deformation of brick and mortar by squeezing them laterally, at constant volume.



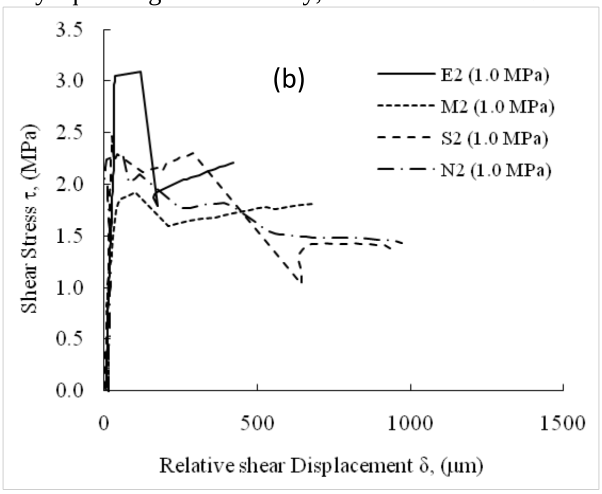


Figure 4. Experimental results of shear stress vs. shear deformation: (a) for 0.5 MPa; (b) for 1.0 MPa of confining pressure.

As previously mentioned, the interface behaves like a quasi-brittle material, and exhibits a very small hardening branch, that appears between the elastic limit and the peak stress. The post-peak damage and release of strain energy is quite evident, as the stress drops gradually. After the initial damage, the shearing surface readjusts and

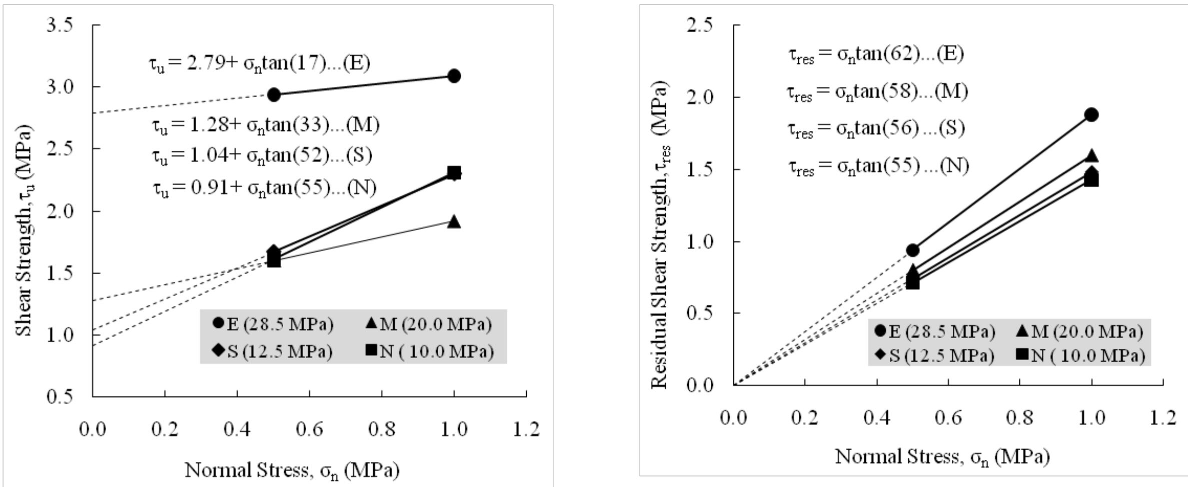


Figure 5. Effect of normal stress: (a) variation of shear strength with normal stress; and (b) variation of residual shear strength with normal stress

relocates its position for new sliding resistance, after losing the cohesive bond at the brick-mortar interface. This stage is called residual stress, and depends mainly on the interface static friction and confining pressure; in the following, it will be indicated as residual shear strength. After reaching the residual shear strength, the relative movement between the two sliding surfaces turns into a rigid body movement, with very little (or no) relative shear deformation. This stage can be considered as a complete failure stage, and the whole phenomenon can be indicated as dynamic friction, something that is beyond static equilibrium and static analysis.

In this study, two important parameters, confining pressure and mortar strength were noticed as major factors contributing to the shear capacity at the brick-mortar interface. The increase in shear strength with increasing confining pressure, for different mortar strengths (N, S, M and E) can be seen in Fig. 5a. This figure indicates that the shear strength for the highest mortar strength (E) is much higher than that mortar with lower strength (such as N, M and S). The increase in shear strength with increasing confining pressure (or initial friction coefficient) for the high strength mortar (E) is lower than that of lower strength mortar. The relationship between confining pressure and residual shear strength is shown in Fig. 5b. It is quite interesting to notice that once the interface cohesion is lost, the ratio of residual shear strength to confining pressure (or residual friction coefficient) increases to an almost constant rate, that is independent of the mortar strength. The trend of shear strength and residual shear stress will be discussed in more detail in the next section, considering other experimental results from different literatures.

The failure modes of the shear test specimens were predominantly interface failures. In all cases the mortar separated from either the inner brick or the outer, or both. No substantial damages was seen on the brick surfaces; rather, some small mortar pieces appeared to remain attached to the brick surfaces, something that indicates that if the brick strength is higher than the mortar strength, damage takes place within the mortar. During the increase of the shear load, a minor crack was observed, propagating almost halfway into the depth of the outer bricks at the peak shear load. This crack can be regarded as a flexural crack. Since the crack does not reach the interface, its effect on the average shear strength at the interface is not very significant.

Shear Strength and Shear Stress-Slip Relationship

The shear capacity of masonry joints with moderate confining pressure can be predicted by the Mohr-Coulomb criterion (Lourenço et al. 2004), which establishes a linear relationship between the shear strength τ_u and the normal compressive stress σ_n (see Fig. 4a), by $\tau_u = c + \sigma_n \tan\phi$. Here, c represents the cohesion between the brick-mortar interface and $\tan\phi$ is the tangent of the friction angle of the interface, which shows that the shear strength increases linearly with the increase of confining pressure. It can be observed in Fig. 4 that the shear strength increases almost linearly with both confining pressure and mortar strength. It is also interesting to note that the increment ratios for the increase in confining pressure are rather close to each other among different mortar strengths and brick types. This reveals that for ordinary clay bricks and normal strength mortar, the coefficient of internal friction is a constant, whose average value is close to 40°. The internal friction angle for mortar-limestone interface is experimentally found to be 37.5° by Armaanidis (1998). Gabor et al. (2006) also found a friction angle of 40° for hollow clay brick masonry. The value of interface cohesion c represents the quality of bond between the brick-mortar interfaces. The increase in mortar strength will increase the interface cohesion but this is not the only factor that controls the interface cohesion. There are other influential factors like the type of brick, the surface roughness of the brick, the absorption of the brick, as well as the brick strength, that also affects the quality of the interface cohesion. Since, both this and the previous studies did not deal specially with those parameters explicitly; a clear idea about their influence on the interface cohesion is still open to investigate.

The internal frictional angle ϕ at the interface is a function of material properties, as well as surface properties of the two sliding surfaces. In static conditions it should, by definition, be equal to a constant that is called the coefficient of static friction. During the evolution of sliding under compression, however, both the two sliding surfaces undergo substantial deformation, and after the loss of interface cohesion, the value of the frictional angle is different from the static one. Until (and unless) an accurate analysis of the brick surface texture, abrasion characteristics of both brick and mortar, and pore structure of the brick are explicitly carried out, conclusive remarks cannot be drawn on the relationship between the static and dynamic coefficient of internal friction as a function of brick and mortar characteristics. Both the initial and the residual frictional angles are somehow independent of mortar strength. The later lies around 40° except in the case of the Authors' experiments (see Fig. 5b) while the former exhibits a greater scattering. The pre-peak stiffness does not change noticeably with the increase of confining pressure; rather, a change in stiffness is observed with the increase of mortar strength for both solid and hollow bricks. High strength mortar offers strong interface cohesion, which in turn increases the interface stiffness. On the other hand, higher confining pressure levels merely increase the shear capacity of the interface, with the stiffness almost constant. Nevertheless, it cannot be concluded that the confining pressure has no effect on interface stiffness, unless more tests are carried out.

The effect of brick strength on the interface shear strength is another topic of interest. But at this stage the authors regret for not making the results available. The experiment is underway and is expecting to come soon.

4. CONCLUSIONS

In this study, some results obtained on mortar joints in brick masonry under shear and compression, are compared. At first, various experimental results are compared, on the basis of two major affecting parameters, namely confining pressure and mortar strength. From these comparisons the following conclusions can be drawn:

1) The shear capacity of the joints, for both solid and hollow bricks, will definitely rise with the increase of confining pressure acting normal to the joint. The resulting relationship turns to be linear, something that is consistent with the Mohr-Coulomb criterion. Once cohesion is lost (or the shear stress reduces to the residual shear strength), the confining pressure plays a significant role also for the residual shear capacity.

2) The shear strength does increase with the increase of mortar strength, as the interface cohesion increases with the increase of mortar strength, but a definitive relationship cannot be established at this current stage of knowledge. Strong mortar with weak brick and weak mortar with strong brick will behave differently, and as a result, the shear strength will be very much scattering in nature.

3) The shear stress-slip relationship can best be described as pre-peak and post-peak regime. In pre-peak stage the stiffness is somehow constant throughout the loading process, and shows little hardening phase near the peak shear stress. So, the pre-peak behavior can be said as elastoplastic. In post-peak regime, the damage is rather gradual and shear stress reach to a constant value after the loss of interface cohesion, this stage is called residual shear strength.

4) Brick types, its surface roughness and mortar texture definitely have an effect on interface friction. Until and unless some vigorous investigation is carried out to know these effects, an explicit correlation between friction coefficient and mortar strength cannot be established at this present study. Merely a variation of friction coefficient can be shown with respect to mortar strengths and the strength of bricks.

5) The so called dilatancy which causes an upward displacement of the brick units upon sliding has some marginal effect on the overall deformation of brick-mortar assemblages but this deformation can be neglected on the grounds that the deformation of the brick and mortar themselves are large enough in comparison with the dilatant deformation.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECT OF SULFATE RICH ENVIRONMENT ON THE PERFORMANCE OF CEMENT MORTAR

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ABSTRACT

Mortar is a workable paste used to bind construction blocks together and fills the gaps between them. The soils which content sulfate ions have a detrimental effect on structures are called Sulfate Rich Environment or Soil. The durability of structures is affected by many environmental factors the sulfate corrosion being one of the most frequent and detrimental processes. The sulfate ions contained in soils or ground water may diffuse through the capillary pores of concrete due to concentration gradient and react with unhydrated components of the hardened cement paste. For this reason an investigation was carried out on the effect sodium sulfate concentration on cement mortar. Experiments were carried out on the ordinary Portland cement standard mortars. Sulfate exposure of mortar was initiated after 7 days distilled water curing various changes (i.e. mass, volume, strength) in cement mortar due to curing sodium sulfate solution was observed.

Keywords: *Curing, Acidic Environment, Strength*

1. INTRODUCTION

Concrete durability is one of the most important considerations to ensure the performance of structures. Sulfate attack is possibly the most common and widespread form of chemical attack on concrete. The durability of concrete structures exposed to sulfate reach environment has been facing a problem of long standing. The role of Sulfate ions in causing deterioration of concrete has been investigated intensively. Sulfate attack in concrete has been known to occur when sulfate solutions, derived either from a constituent in the concrete such as aggregate or from external sources such as groundwater, react with the cement hydrates present in the hardened cement paste to form the products which can occupy greater volume than the reactants (Cohen and Mater, 1991; Skalny et al. 2002; stark 2002). The major sources of these salts are soils (particularly Arid Region), groundwater, seawater, industrial chemicals & wastes, and fertilizers. Sulfates may occur naturally in soil and groundwater, in industrial effluents and wastes from chemical and mining industries, as well as in sea water. Sulfate attack has been recognized as responsible for concrete deteriorations in a wide variety of structures. The ready availability of these sulfates causes damage to concrete, depending on its concentration, solubility and the ambient environmental conditions (Miller et al., 1945). Sulfate induced problems are catastrophic in the case of light weight structures, pavements and underground transportation and drainage facilities and also pose problems in disciplines of chemistry, geology, hydrology, environmental science and engineering (Burkart et al., 1999; Puppala et al. 1999). Oxidation and weathering of sulfur and sulfide bearing earth materials such pyrites, shale and gypsum leads to volumetric changes, inferior engineering properties, acid production, failure of stabilization methods and stability problems in rock and soil masses (Dubble et al., 1999). Sulfate attack was first noticed in Portland cement concrete and recognized early in the 19^{th} century. It was proved that the reaction of Portland cement mortar in sulfate rich environment yields delicious end products namely ettringite or thaumasite (ACI, 1982). With the advent of chemical stabilizations methods of soils i.e. stabilization of soils with foreign materials such as cement and lime researchers came to know about the possibility of sulfate problem in soils (Shenwood, 1962; Mitchell, 1986).

From last few decades, sulfate induced problems became more predominant and attracted attention of many researchers (Mitchell, 1986; Hunter, 1988; Little et al., 1988). According to Hunter (1988), sulfate heaving or chemical swelling is a complex mode of heaving resulting from the reaction of ground sulfates with calcium, alumina available in cement and rock/soil in the present of moisture at a specific temperature and pH conditions. According to Hunter, sulfate heaving scenario which worsens the situation with elapsed time (Hunter, 1988). Due to conversion of anhydrite into gypsum, volume was changed. Sulfate is a common mineral, exists in three phases namely solid, liquid and vapor. Soils and rocks are the examples of sounds for sulfates in solid state; similarly water and air in their respective phases. Sulfates, sulfides and organic sulfurs are the main sources of sulfate induced heave in soils and rocks and mortars and pose a great danger to infrastructure facilities built to them (Hawkins et at., 1987).

Many studies have shown that the ingress of sulfates into a concrete structure by ground water containing soluble sulfates or soils laden with sulfates leads to several expansive reactions and the alteration of the micro structure. When the sodium sulfate is brought into contact with an hydrous particles of the hardened cement paste ettringite and gypsum may be produced which are responsible for concrete expansion and micro cracking.(Crammond and halliwell, 1995) This process may result in a gradual loss of concrete strength accompanied by surface spalling and exfoliation. Besides ettringite, gypsum is also formed in the system, which is dependent upon the sulfate concentration. Formation of gypsum softens the cement paste and therefore leads to the deterioration of the paste microstructure (Brown and Taylor, 1998).The formation of ettringite in cement concrete is well known for decades. Formation of ettringite in presence of cement is attributed to the ability of cement to produce additional calcium and alumina required in the process. Tri calcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminoferrite(C_4AF) are the compounds present in Portland cement in its polished form (Wang, 2003). The reactions that take place between cement and water i. e. hydration of cement are as follows:

$$2C_3H + 6H = C\text{-}S\text{-}H + 3Ca\,(OH)_2$$
$$2C_3S + 4H = C\text{-}S\text{-}H + Ca\,(OH)_2$$
$$2C_3A + 27H = C_4AH_{19} + C_2AH_8$$
$$C_3A + 3CSH_2 + 26H = \underset{\text{(ettringite)}}{C_6A\hat{S}H_{32}}$$

Once ettringite is formed it may move upward by capillarity on can be carried downward by infiltration on it can remain its position as an evaporate (Natarajon, 2004). Due to the complexity of the problem and the increasing numbers of cases of ettringite formation and consequent damages to the structure, it is vital to elucidate formation of ettringite in sulfate bearing soils and rocks in presence of cement, lime on alumina source. According to Hunter (1998), for ettringite to form four ingredients namely alumina (from soil/rock/cement/lime), additional calcium (cement/lime), sulfate source (soil/rock/ground water) water are needed at specific temperature (72^0F) and (>10.5) environments (ACI, 1982; Depuy, 1994).

Solid Sulfate does not attack the concrete severely but when these are in solution, they find the entry into porous concrete and react with the hydrated cement products. The deterioration of concrete exposed to sulfate is the result of the penetration of sulfate ions into the concrete and their chemical reaction with the cement matrix (Euangelou et al., 1995). The sulfate ions of soils or ground water may diffuse through the capillary pores of concrete due to concentration gradient and react with unhydrated components of the hardened cement paste. For this reason an investigation was carried out on the effect sodium sulfate concentration on cement mortar. Numerous research works has been conducted to find out the effect of sulfate ions on concrete. When the sulfate ions are brought into contact with a hydrous particles of the hardened cement paste, ettringite and or gypsum may be produced which are responsible for concrete expansion and micro cracking. This process may result in a gradual loss of strength (Hawtins et al., 1998; Minesota Pollution Control Agency, 1999).

2. EXPERIMENTAL DETAILS

Materials: In this experiment river bed sand was used as a fine aggregate. It was prepared according to ASTM C778 standard and given below.

Table 1: Fine Aggregate Properties

Sieve no.	% Cumulative Retained (ASTM C778)	% Used
#30	0-4	3
#40	25-35	25
#50	70-80	70
#100	96-100	98

Cement: Portland composite cement was used in this experiment. The normal consistency, initial setting time and final setting time were 138.5cc, 75 minutes and 218 minutes respectively. The clinker, gypsum and fly ash were 65-79%, 0-5% and 21-35% respectively.

Water: In this experiment tap water was used for preparation of samples. The characteristics of the used water are given in table 2.

Table 2: Tap water properties

Characteristics	Tap water
pH	7.63
Salinity (mg/L)	1.40
Conductivity (mS/cm)	2.80

Working Procedure:

Mortar cubes (50 × 50 × 50 mm) were prepared from cement, sand and water,. The mixing ratio of cement to sand was 1.0, 2.75 and W/C ratio 0.49. The fine aggregate was river bed sand prepared according to ASTM C778. All samples were cured for 7 days in curing chamber and then one third were kept in the chamber. Fifty percent of the remaining samples were kept in distilled water and rests were in 5% Na_2SO_4 solution up to 90 days. After every 7 days the samples were taken out for compressive strength and dimensional test. At the same time the pH values of the curing solution and water were also measured.

Figure 1: Preparation of cement mortar

Strength Measurement: The failure loads were taken by Universal Testing Machine of cement mortar cube. From these load compressive strength was calculated.

Figure 2: Failure load taken by Universal Testing Machine

Volume Measurement: The length of cement mortar of each direction was taken by digital slide calipers. From these lengths volume was calculated and later % volume change was determined.

Figure 3: Length measurement

Mass Measurement: The mass of cement mortar cube was taken by digital balance. Then percentage of mass change was determined.

Figure 4: Mass measurement

3. RESULTS AND DISCUSSIONS

In order to find out the effect of sulfate environment on cement mortar, the curing type was changed in this study after 7 days of pre curing in normal water. In laboratory it was found that the mass of mortar samples gradually increased when it was cured in sulfate environment. And the samples which were cured in normal and distilled water exhibits higher rate of mass changes than sulfate solution for maximum 28 days and at later ages these samples showed different nature. This may be due to the formation of ettringite in cement mortar when they were faced sulfate ions. The nature of mass changes shown in Fig 5.

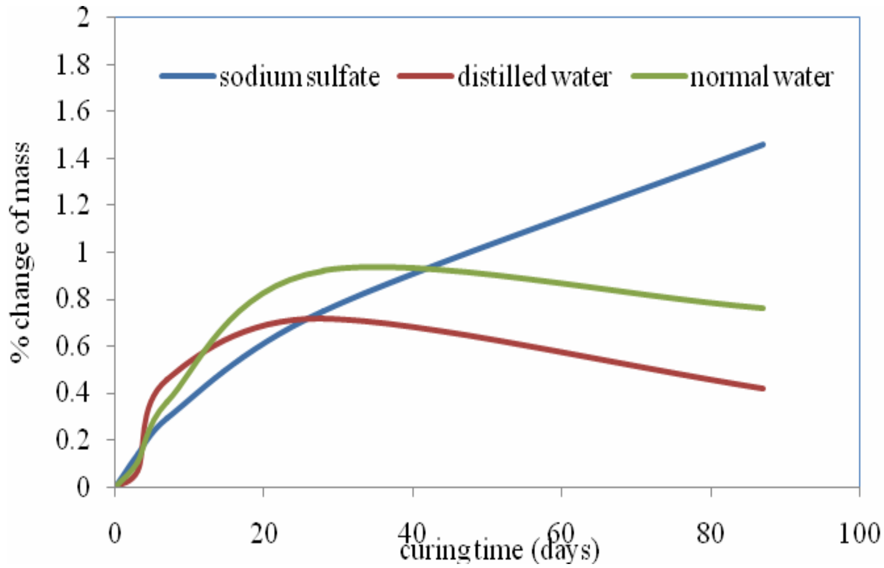

Figure 5: changes in mass due to curing condition and time

The compressive strength increased with the period of exposure in sulfate solution and this is observed at 90 days curing time. Prior 90 days the compressive strength was observed in increasing nature and at 28 days cured in sulfate solution it showed highest strength among all the samples. This may be due to the formation stage of gypsum and from Fig 5 it is observed that at this stage the mass is minimum. The total results were shown in Fig. 5.

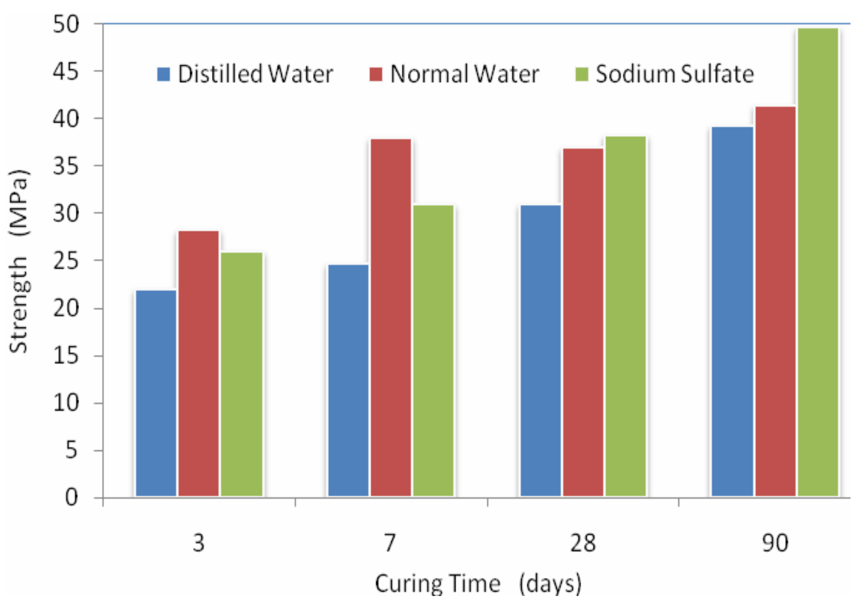

Figure 5: Compressive strength of mortar with curing time and condition

The volume was fluctuated in sulfate solution. Initially the volume was increased sharply in normal water and sodium sulfate solution. Then the volume decreased in normal water but the volume decreased in sulfate solution up to 30 days at a greater rate than normal water, even it's 30 days volume was less than initial volume.

Then the volume increased and crossed over it's initial volume. But in case of distilled water it showed a different nature. It's volume decreased sharply up to 7 days, then increased up to 40 days and decreased at a slow rate up to 90 days. This may be due to the formation of thaumasite. The nature of volume change shown in Fig. 6.

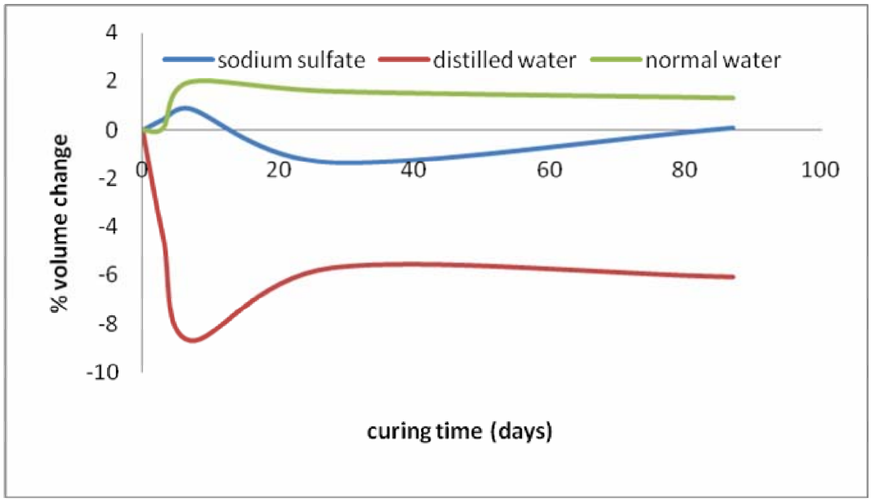

Figure 6: Volume change of mortar with curing time and condition

4. CONCLUSION

Based on the experimental results obtained from this study, the following conclusions can be drawn:

Cementitious materials in sulfate rich environment is shows increasing in strength up to 28 days and at later age it exhibits reverse result.
The rate of gain of mass is slower up to 28 days and after this the rate is faster when the cement mortar samples are in contact with the sulfate ions.
The rate of change of volume is slower in contact with the sulfate ions then normal and distilled water.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 2~3 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PERFORMANCE OF CEMENT MORTAR IN ACIDIC ENVIRONMENT

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ABSTRACT

The effect of sulfuric acid on cement mortar has been widely paid consideration to research for various purposes. The durability of concrete structures is affected by many environmental factors, the acid corrosion being one of the most detrimental processes. The experimental research on mortar subjected to acid corrosion is conducted in this study. This paper presents the findings of a laboratory study under accelerated conditions on cement mortar. In the laboratory investigations, 50 mm cube mortar specimens were prepared using Portland composite cement, ASTM graded sand and tap water. In the experiment before curing in sulfuric acid, the samples were cured in distilled water for 7 days. The pH of acidic solution was controlled to 1 and 2 during the period of curing. Samples were also exposed in normal water and distilled water. The laboratory specimens were tested for weight reduction and compressive strength. The results indicates that the acid environment has influenced on mortar performances.

Keywords: *Acidic, Cement mortar, Strength, Mass*

1. INTRODUCTION

Sewer systems endure severe corrosion mainly to sulfuric acid attack which causes a serious threat to the concrete integrity of the system. In dry environmental conditions, concrete deterioration in sewer systems due to sulfuric acid formation is enhanced due to the elevated temperature conditions and scarcity of flush water. Under anaerobic conditions in sewage environments, the sulfate-reducing bacteria reduce organic sulfur compounds to hydrogen sulfide (H_2S). As H_2S is a weak acid, it dissolves in the sewage under neutral or alkaline conditions. If the pH value decreases, e.g., due to acidification, it is emitted into the sewage atmosphere(T. Kawahigashi, 2005.10.). Turbulence in the sewage flow is another cause for H_2S emission. Once H_2S reaches the atmosphere, it is absorbed by the moisture on metallic and concrete surfaces of the structure. H_2S then reacts with oxygen, in the presence of moisture and aerobic bacteria it transformed to sulfuric acid, which leads to the corrosion of metallic and concrete surfaces. Besides this acid rain, sulfur-rich soils, animal husbandry and industrial processes can produce acidic conditions that degrade concrete (Saricimen et al. 2003, KFUPM, 1999). The spectrum of aggressive acidic media is wide. Acidic attack usually originates from industrial processes, but it can even be due to urban activity. Even natural exposure conditions may cause acid attack. Free acids in natural water are rare.

Corrosion is a natural phenomenon which can be considered either chemical or electrochemical in nature. Sulfuric acid is very damaging to concrete as it cause both acid and sulfate attack (Mindess et al. 1981). Acid rain, sulfur-rich soils, animal husbandry and industrial processes can also produce acidic conditions that degrade concrete. The degree of aggressivity of an acid is dependent on the chemical character of an ions present(Zivica V, Bajza A ,2002). The strength of acid, its dissociation degree in solutions and, mainly, the solubility of the calcium salts formed are dependent on the chemical character of an ion(Raju et al. 1984).

Cement concrete continues to be the pre-eminent construction materials for use in any type of civil engineering structure. Performance of these structures in terms of their strength and stability has withstood the test of time but the life span of the structures has become a matter of concern(T. Kawahigashi. 2007.2) .Cement concrete is the most widely used structural material due to its satisfying performance in strength requirements (Jaarsveld et al. 1998). When one thinks about the performance of concrete the chemical attack is the most important one among others. Durability is an important engineering property of concrete, which determines the service life of concrete structures significantly. In this study the effect of acidic environment on cementitious materials is discussed.

2. EXPERIMENTAL DETAILS

Materials: In this experiment graded sand was used. It was prepared by ASTM specification C778 and given below-

Sieve no.	% Cumulative Retained	% Used
#30	0-4	3
#40	25-35	25
#50	70-80	70
#100	96-100	98

Cement: Portland composite cement was used in this experiment. The normal consistency, initial setting time and final setting time were 138.5cc, 75 minutes and 218 minutes respectively. The clinker, gypsum and fly ash were 65-79%, 0-5% and 21-35% respectively.

Water: In this experiment tap water was used. The characteristics of this water are given below-

Characteristics	Tap water
pH	7.63
Salinity (mg/L)	1.40
Conductivity (mS/cm)	2.80

Working Procedure:

The laboratory samples were prepared from Portland composite cement, ASTM graded sand and tap water. A sand to-cementitious materials ratio (s/c) of 2.75 and an effective water-to-cementitious materials ratio (w/c) of 0.49 were used in this experiment. The mixing water had a pH of 7.63 and salinity 1.40 mg/L. Using these materials the samples were prepared according to ASTM standard. After initial curing of 7 days in curing chamber the specimens were immersed in an acidic solution chamber having pH of 1 and 2, at this time some samples were also kept in curing chamber and distilled water chamber. The pH of the curing media were monitored on a weekly basis, and appropriate adjustment of the acid solution was made, as required, by adding acid and distilled water to the acid bath. The specimens were then taken out of the curing chamber and air-dried before testing. After photographic recording, the specimens were weighed and tested for compressive strength.

Figure 1: Preparation of cement mortar

Strength Measurement: The failure loads were taken by Universal Testing Machine of cement mortar cube. From these load compressive strength was calculated.

Figure 2: Failure load taken by Universal Testing Machine

Volume Measurement: The length of cement mortar of each direction was taken by digital slide calipers. From these lengths volume was calculated and later % volume change was determined.

Figure 3: Length measurement

Mass Measurement: The mass of cement mortar cube was taken by digital balance. Then percentage of mass change was determined.

Figure 4: Mass measurement

3. RESULTS AND DISCUSSIONS

During the accelerated laboratory test where the mortar specimens were exposed to normal water, distilled water, and pH of 1 and 2 which indicates an acidic solution, deterioration of the mortar specimens was quite severe for those samples which were in pH of 1 solution. The effect of sulfate on the specimens was photographically recorded during the exposure period before testing of the specimens. Fig. 2 shows the mortar specimens after 28 days of exposure to different solution.

(a)

(b)


(c)

Figure 2. Sample condition after 28 days of curing (a) Distilled Water (b) pH2 and (c) pH1

Fig 3. shows the compressive strength for the mortar samples cured at different curing media. Samples cured in solution with pH1 shows no strength after 28 days curing and at the same time samples in pH2 exhibits lower strength than the 7 days strength. At the same time samples cured in other two medias showed increasing nature in strength depending on time of curing. The internal bond of the cementitious material is decreasing in nature when contact with acidic environment.

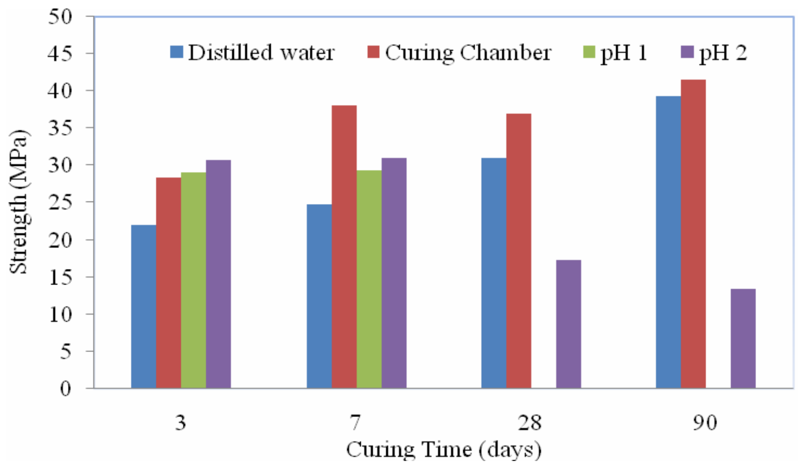

Figure 3. Compressive strength with curing time

From fig 4. It shown that the mass increased at faster rate up to 10 days, then the samples were failed in contact with pH=1 solution. In case of Ph=2 solution, the mass increased at a faster rate up to 7 days, after then the mass increased linearly up to 90 days. But normal water and distilled water showed a different nature. Initially the mass increased randomly up to 30 days, then the mass decreased approximately linearly up to 90 days.

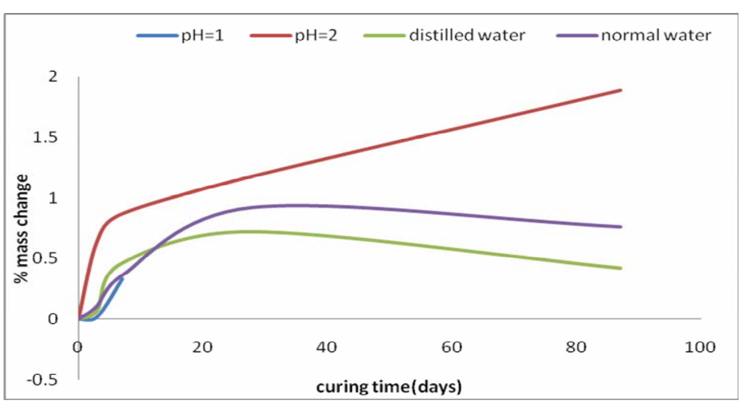

Figure 4: Mass change with curing time and conditions

In acidic conditions, the volume changed approximately linear. The volume increased at a faster rate up to 53 days and remained constant up to 90 days in contact in pH=2 solution. The volume decreased at a slower rate up to 28 days and samples were failed in pH=1 solution. But normal water and distilled water shown a different nature. In case of normal water th volume increased sharply up to 7 days, then remained almost constant. In distilled water the volume decreased fastly up to 7 days , then increased up to 40 days and decreased up to 90 days at a slow rate. This may be due to the formation of calcium sulphoaluminate. This nature shown in Fig. 5.

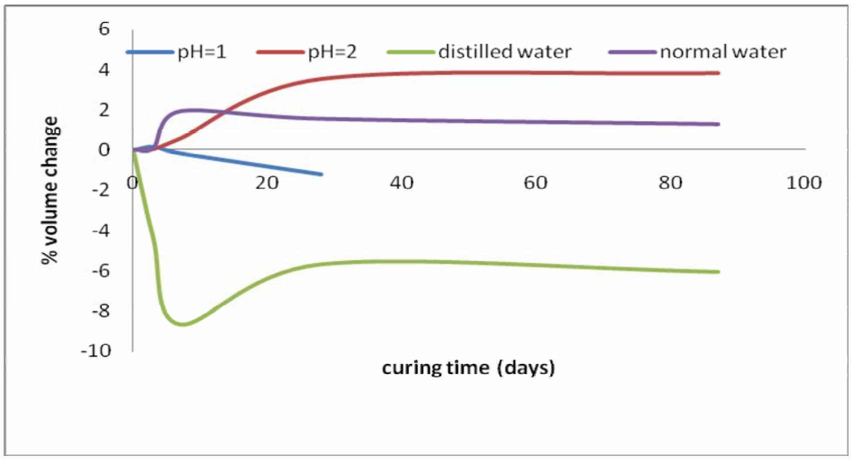

Figure 5. Volume change with curing time and conditions

4. CONCLUSIONS

In order to assess the corrosion of water-cement system, estimation was performed for long term. Environmental operation of sulphuric acid was estimated by the pH and ion measurement. And the corrosion was also performed simultaneously by the mass change and length measurement.The mortar specimens had very significant deterioration during exposure to pH1 and pH2 in the laboratory. Acidic environment reduce the durability of cementitious materials. A lower pH value of curing media degrades the performance of cement mortar rapidly. Mass of the sample increased due to cured at acidic environment and indicates to produce gypsum which is in expansive nature and causes of deterioration of the samples at early age.

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Geotechnical Engineering

Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

LIQUEFACTION CHARACTERISTICS OF IN-FILLED SOIL AT BANASREE IN DHAKA CITY

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ABSTRACT

Soil goes under a significant amount of deformation or liquefaction during earthquake. A moderate to major earthquake can cause liquefaction, which may have catastrophic effect on structure. During liquefaction, soil behaves more like fluid and loose cohesionless soil is more susceptible to liquefaction. A soil type which may be subjected to liquefaction can be identified by Chinese Criteria described by Wang (1979) and reported by Seed and Idriss (1982). Dhaka is the largest city in Bangladesh, had a population over 15 million. To meet the need of increasing population, Dhaka city is extending continuously. As a result most part of modern Dhaka city is being constructed over a low laying land filled with primarily cohesionless soil or soil with low plasticity. This uppermost filling layer may be very much hazardous against liquefaction and to identify the prone ness to liquefaction of this soil type is the main objective of this study. SPT based semi-empirical procedure has been used for evaluating cyclic resistance ratio afterwards Liquefaction Potential Index (LPI) has been used to determine its probability.

Keywords: *Liquefaction, Cohesionless soil, Earthquake, SPT, LPI*

1. INTRODUCTION

The response of soil due to seismic hazards, produce a significant amount of deformation or liquefaction, has been one of the major concerns for geotechnical engineers working in the seismically active regions. Liquefaction can occur due to moderate to major earthquakes, which can cause severe damage to structures. Marcuson (1978) defines this phenomenon as "transformation of a granular material from solid state to liquid state due to increased pore water pressure and reduced effective stress". When this happens the sand grains loose its effective shear strength and will behaves more like fluid. The grain size distribution of soil, duration of earthquake, amplitude and frequency of shaking, distance form epicenter, location of water table, and cohesion of the soil and permeability of the layer affects liquefaction potential of soil (Seed and Idriss 1971). The liquefaction hazards are associated with saturated sandy and silty soils of low/non plasticity and density. The liquefaction potential of soil is generally estimated from laboratory tests or field tests. Among the field tests, the SPT test data has been widely used for this purpose.

2. METHODOLOGY

Estimation of two variables, required for evaluation of liquefaction resistance of soil, are: i) the seismic demand on soil layer, expressed in terms of CSR (Cyclic Stress Ratio) and ii) the capacity of the soil to resist liquefaction, expressed in terms of CRR (Cyclic Resistance Ratio). In this calculation Modified Chinese Criteria has been considered. As per the modified Chinese criteria, if the liquid limit of the soil is greater than 33.5, the deposits are classified as non-liquefiable.

2.1 Evaluation of CSR

Seed and Idriss (1971) formulated the following equation for calculating the cyclic stress ratio:

$$CSR = 0.65 * ({a_{\max}}/{g}) * ({\sigma_{vo}}/{\sigma'_{vo}}) * r_d \tag{1}$$

Where a_{max} = peak horizontal acceleration at the ground surface generated by earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses respectively (unit weights have been estimated according to the SPT value and soil type for the corresponding soil layer) and r_d = stress reduction coefficient. For ease of computation, T. F. Blake (1996) approximated r_d by the following equation:

$$r_d = \frac{(1.0 - 0.4113 * z^{0.5} + 0.04052 * z + 0.001573 * z^{1.5})}{(1.0 - 0.4177 * z^{0.5} + 0.05729 * z - 0.006205 * z^{1.5} + 0.00121 * z^2)} \quad (2)$$

Where z = depth beneath ground surface in meters.

2.2 Evaluation of CRR

Several field tests data are generally used for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear wave velocity measurements (Vs), the Becker penetration test (BPT). In this study only SPT data has been used to calculate the cyclic resistance ratio. A clean sand based equation, given by A. F. Rauch (1998), has been used in this study.

The equation is valid for $(N_1)_{60cs} < 30$. For $(N_1)_{60cs} \geq 30$, clean granular soils are too dense to liquefy and considered as non-liquefiable. Seed et al. (1985) noted an apparent increase of CRR with increased fine content. The following equation (Youd et al. 2001) were developed to normalized the effect of fine content by I.M. Idriss with the assistance of R.B. Seed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$.

$$(N_1)_{60cs} = \alpha + \beta \quad (N_1)_{60} \quad (3)$$

2.3 Correction for SPT Value

Standard penetration resistance (N) value used in this study has been corrected to normalize the effects of over burden pressure, energy ratio, borehole diameter, rod length and the presence liner. The recommended correction according to T. L. Youd et al. (2001) is shown as follows.

$$(N)' = N_m C_N C_E C_B C_R C_S \quad (4)$$

Where, N_m = Measured standard penetration resistance; C_N = Factor to normalize N_m to common reference effective overburden pressure (approximately 100 kPa) as per Kayen et al. (1992); C_E = Correction for hammer energy ratio; C_B = Correction factor for borehole diameter; C_R = Correction for rod length and C_S = Correction for samplers with or without liners. All other correction factors except C_N have been used according to Robertson and Wride (1998).

2.4 Magnitude Scaling Factor

The equation of CRR given by A. F. Rauch (1998) is valid only for a magnitude of 7.5 earthquakes. To adjust the clean sand based equation to magnitude smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed as "Magnitude Scaling Factors (MSF)". To illustrate the influence of MSFs on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR, CSR and MSFs as follows:

$$F.S. = (CRR_{7.5}/ CSR)*MSF*K_s \quad (5)$$

Where, the MSF can be evaluated as $MSF = (10^{2.24}/M_w^{2.56})$. The Modified Chinese Criteria have been applied by considering the Atterberg limits of soil. As per the modified Chinese criteria, if liquid limit of the soil is greater than 33.5, the deposits are classified as non liquefiable. In case of non plastic sand or silty sandy soils, a factor of safety of 1.2 or above has been considered as non liquefiable soil. It has been previously mentioned that due to plate movement, sufficient energy has been accumulated to produce a probable earthquake of magnitude 6.8. So a magnitude of 7.5 has been used in this study after considering factor of safety.

3. SEISMISITY OF THE REGION

Bangladesh is located in a seismically active zone. Previous records show that, Bangladesh and the surrounding areas experienced at least 1000 earthquakes having $M \geq 4$ in the last 100 years (M. A. Ansary and M. A. Rashid, 2000). Recent research, jointly conducted by Lamont-Doherty earth observatory, Columbia University, USA and the department of Geology and Dhaka University, shows that the Dhaka is moving 30.6 cm/year to the direction of northeast. There is an active subduction zone across Bangladesh which lies very close to Dhaka city. It may cause an earthquake of magnitude 6.8 (Khan and Hussain 2005). According to BNBC (1993), Dhaka city together with its surroundings is situated in the seismic zone-2, which has a basic seismic coefficient, Z = 0.20. The maximum ground surface acceleration has been estimated from the seismic map based on 2 % probability of exceedance within a period of 50 years. The peak ground surface acceleration has been estimated around 0.20g (according to BNBC proposed edition). So the safety of lives and infrastructures against seismic hazards is now a major issue.

4. LIQUEFACTION POTENTIAL INDEX

Assessment of liquefaction potential is an important issue for determining seismic hazard in geotechnical engineering. The factor of safety (F.S) determined from conventional method is not a sufficient alone for the evaluation of the liquefaction potential. The severity caused by soil liquefaction can not be assessed directly by the FS. Iwasaki et al. (1982) introduced the method of Liquefaction Potential Index (LPI) in order to quantify the severity of liquefaction. They assumed that the severity of liquefaction should be proportional to the:

i) thickness of the layer
ii) proximity of the liquefied layer to the surface , and
iii) the factor of safety to the liquefied layer is far less than 1.0

According to Toprak and Holzer (2003), the simplified procedure predicts what will happen to the soil element whereas the index predicts the whole soil column and the consequence of the liquefaction at the ground surface.
Sonmez (2003) modified the method proposed by Iwasaki et al. (1982) by considering the threshold value of 1.2 of factor of safety as the limiting value between the categories of marginally liquefiable to non-liquefiable. Thus LPI is defined as:

$$LPI = \int_{0}^{z} F(z)W(z)dz \tag{6}$$

Where z is the depth below the ground surface in meters; F(z) is a function of the factor of safety against liquefaction (FS), where F(z) = (1- FS) when FS < 0.95, F(z) = $2*10^6*e^{-18.427F.S}$ if 0.95<FS<1.2 and if FS>1.2, F(z) = 0, W(z) is a function of depth (Z), when Z is greater or equal twenty (20) meters, W(z) = 0, when Z is smaller than 20 meters W(z) = 10 – 0.5*Z.
A chart has been given in Table 1 to assess the liquefaction potential based on liquefaction potential index (LPI).

Table 1: Liquefaction potential classification based on LPI

Liquefaction Index (LPI)	Liquefaction potential
0	Non-liquefiable
0 < LPI ≤ 2	Low
2 < LPI ≤ 5	Moderate
5 < LPI ≤ 15	High
15 > LPI	Very High

5. GEOTECHNICAL DATA AND RESULTS

Geotechnical data of the study area as shown in Figure 1 have been collected from different soil testing companies. Wash boring and sampling with the help of split spoon at a constant interval of 5ft has been used to advance the bore hole. Data have been collected from Sixty six bore holes over twenty two locations all over the Banasree area in Dhaka. Essential physical and engineering properties of soil such as grain size distribution, atterberg limits and soil type along with SPT (N) values, have been sorted and tabulated according to location and depth of soil specimen. Geotechnical data represents that the investigated soil consists of both cohesive and non cohesive in nature, but the upper most layers varying up to the depth of twenty feet comprises of sand filling with very loose to medium dense state. These types of in-filled saturated sand are most vulnerable to liquefaction, as ground water table will play an important role during liquefaction. Due to unavailability of sufficient data regarding the variation of ground water table with season, all the calculations have been performed considering the ground water table at EGL (Existing Ground level). Due to lack of sufficient grain size analysis data and unit weight of sandy soil, the fine content and unit weight of soil have been estimated, from the classification of the soil layers according to ASTM D 2488-00 and Soils and Foundations Workshop Manual, FHWA HI-88-009 respectively. Using the collected field data, liquefaction potential index of each soil layer have been evaluated according to the above mentioned procedures which have been found to vary from thirteen to sixty.

Figure 1: Approximate Location of the study area from Google Map

6. CONCLUSIONS

The main objective of this study was to access the liquefaction susceptibility of in-filled soil at Banasree, the northeast part of Dhaka city. It has been found that the estimated liquefaction potential index value varies from thirteen to sixty where there is a layer of in-filled soil. Most part of the study areas are occupied by this type of filling soil. According to Sonmez (2003), those filling type soil have a high to very high susceptibility to liquefaction during earthquake, when the soil comes in to saturation. It can be concluded that the evaluation of liquefaction potential can not be the only measure of determining seismic hazard but it should be properly and carefully evaluated before construction and proper remedial measure should be taken to minimize its effect.

ACKNOWLEDGEMENT

The authors would like to express their gratitude to "Icon Engineering Services" and "Pile Stone Engineering & Construction Firm" for their cooperation. Without their kind help it would be very much difficult to continue this study.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

STUDY ON THE LOCALLY AVAILABLE CLAYEY SOILS FOR THE PREPARATION OF LINER

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ABSTRACT

The main purposes of this study were to identify the soil suitability for clay liner, to improve the performance of soil samples by introducing cement admixture and to estimate the level of performance of liner. To conduct the experiment, three soil samples were collected at 5′ below of ground level from three different location of KUET campus, Khulna and the routine laboratory tests were conducted to measure the physical properties of soil samples. Standard Proctor Compaction tests were performed on collected soil samples at different molding water content. The hydraulic conductivities of soil samples at optimum water contents without cement were found as 0.230x10^{-7} cm/s, 13.20 x 10^{-7}cm/s and 15.30x10^{-7} cm/s respectively. When 5% cement was added with samples, the hydraulic conductivities were found as 0.228x10^{-7} cm/s, 8.30x10^{-7}cm/s and 9.55x10^{-7} cm/s at 39%, 16.5% and 19.4% water content, respectively. Simple laboratory test setups were established and the necessary tests were run to observe the changes of leachate properties to justify the performance of soil samples as liner materials. The test results of leachate and infiltrated leachate indicate that the selected clayey soils can be used as liner to prepare a hydraulic barrier for preventing the flow of fluids.

Keywords: *Landfill, Clay liner, Bottom liner, Bentonite, Municipal Solid Waste (MSW)*

1. INTRODUCTION

From the emergence of urban civilization, large amount of waste materials are being produced every year from urban and industrial areas. With the increasing numbers of population, large amount of waste is generated, which results in environmental degradation. One of the preferred methods of dealing with this kind of environmental problem is to dispose of the waste in sanitary landfills (Arasan and Yetimoglu, 2008). Because of the pollution concern that arises from infiltrating leachate, landfill site should be on low permeable soil or has to be lined with some material to reduce permeability. Several types of materials such as clay, plastic, composite liner, asphalt concrete liner etc. are available for lining. Among the materials available, clay is locally and readily available material and may be as double liners for very hazardous waste landfills. The double liners include a top liner designed to prevent the migration of hazardous materials through it during the active life and postclosure period and bottom liner consisting of one or more layers of clay.

The low permeability of clayey soil is to use as liner in sanitary landfill to prepare a hydraulic barrier for preventing the flow of fluids. Soils classified as inorganic clay with high plasticity is considered are the suitable material for landfill liner (Oweis and Khera, 1998). If naturally available clay or clayey soil is not suitable for liner, kaolinite or commercially available high swelling clay (Bentonite) can be mixed with local soils or sand. Especially, bentonite clays are preferred, because of its fine particle size and consequent micro pores and high surface charges; it possesses low hydraulic conductivity and a high adsorption capacity (Sivapullaiah *et. al.*, 2000). When exposed to water, the bentonite in the geosynthetic clay liner (GCL) hydrates and swells to form a thin layer having low hydraulic conductivity (Kraus *et. al.*, 1997). In Bangladesh, this materials are not locally available and would have to be imported from elsewhere and could significantly increase the cost of construction (Alamgir *et. al.*, 2005).

This study is aiming to evaluate the properties of locally available clay as top and bottom liners and assess its performance as a landfill material. For this study, the characteristics of clay soil samples collected from KUET Campus were observed from several routine laboratory tests such as Atterberg limits, hydraulic conductivity at different water content, standard proctor compaction test and specific gravity test. The test results revealed that the subsoil properties, could be considered as a potentially suitable material for the clay liner of landfill.

After observation of conductivity of different samples, the actual performance of liner against Leachate collected from an active ultimate disposal site of MSW, namely, Rajbandh at Khulna city was observed by permeability test.

2. METHODOLOGY

The soil samples used as liner in this study were collected at 5 feet below ground level from three different locations of KUET campus, Khulna. After collection, different properties of soil were determined through laboratory tests. The basic tests such as water content, specific gravity, particle size distribution and Atterberg limits of the soil samples were performed according to British Standard (BS 1377:1990). Standard proctor compaction test and Hydraulic conductivity at different water content were determined in the geotechnical laboratory, KUET, Khulna.

Cement is very available and comparetivly low cost than the geosynthetic material that can bind other materials together. Cement admixtures reduce the soil permeability. So, 5% cement was mixed with collected soil samples to improve the lining properties. Again Hydraulic conductivity of soil samples with 5% cement at different water content were determined. The collected data were analyzed and compared with the recommended values.

The following recommended values for clay liner are usually apply:

- Coefficient of permeability < 1 x 10^{-7} m/s.
- Minimum layer thickness of 1 m.
- Minimum clay content of 10%.
- Minimum Fines (clay and silt) content > 30%.
- Plasticity index >10% and <65%.
- Liquid limit < 90%.
- Maximum particle size of 75 mm.
- Moisture content must be greater than plastic limit.

After determining the properties such as moistre content, specific gravity, particle size distribution and Atterberg limit, the actual performance of liner system with 5% cement against Leachate was observed by permeability test. The leachate used in the performance study was collected from an active ultimate disposal site of MSW, namely, Rajbandh. Several tests such as pH, conductivity, alkalinity, hardness, sulphate, nitrate and phosphate were performed for raw leachate in the Environment Laboratory, KUET, Khulna.

This liner system consists of a compacted clay liner with 5% cement of 76 mm and a sand layer of 25 mm thickness. The compacted clay liner with 5% cement and sand layer were set up in PVC pipe which had diameter of 8 inches. The leachate collection container was set up with the pipe. The sand layer was placed on porous disk at the bottom of the pipe and clay samples with 5% cement as a liner was placed on sand layer in order to justify the quality of infiltrated leachate. Leachate was able to move through the liner system and stored in infiltrated leachate collection container. The longitudinal cross sectional view of typical test setup of clay liner are shown in the Figure 1.

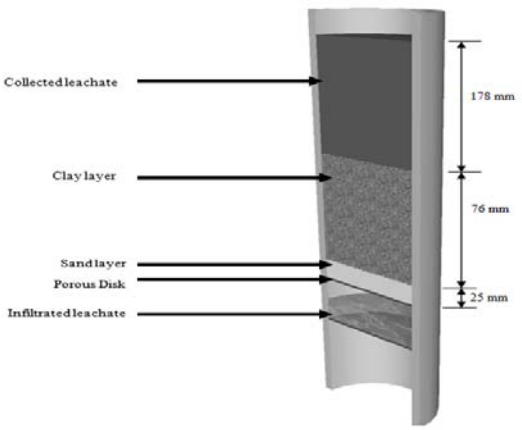

Figure 1: Longitudinal cross section of test setup of clay liner system

For three different samples, three test setups were performed. The test setups were kept under observation until sufficient amount of infiltrated leachate collected. Then pH, conductivity, alkalinity, hardness, sulphate, nitrate and phosphate test were repeated for infiltrated leachate in the Environment Laboratory, KUET, Khulna. The charectericties of raw leachate were analyzed and compared with infiltrated leachate. The three laboratory test setups of clay liner system are shown in the Figure 2.

Figure 2: Three laboratory test setups of clay liner system

3. RESULTS & DISCUSSION

3.1 Physical Characteristic

The performance of clay liner depends on the basic characteristics of the soils. The evaluated index properties of three clay samples were listed as water content of 47.00%, 33.80% and 40.68%; plastic limits of 23.72%, 24.34% and 25.33%;Liquid limits of 44.40%, 34.50% and 32.33%; Plasticity index of 20.70%, 10.27% and 7.0%, respectively. It is observed that the moisture contents were greater than plastic limit. The plasticity of a soil refers to its capability to behave as a plastic material. Plastic clay is a typical suitable material for liner. Clays with high liquid limit generally have low hydraulic conductivity. To achieve a hydraulic conductivity as 1 x 10^{-7} cm/s, the plasticity index must be low. The plasticity index must be more than 7% (Daniel, 1993). Clay content of three samples were 27.34%, 10.34% and 14.33%, respectively. It is observed that clay content of three samples are greater than 10%. The Average Physical Characteristics of three clay samples are listed below in Table 1.

Table 1: Average physical characteristics of soil samples

Sample No.	1	2	3
Water Content, W (%)	47.00	33.80	40.68
Specific Gravity, Gs	2.61	2.79	2.70
Plastic Limit, Wp (%)	23.72	24.34	25.33
Liquid Limit, W_l (%)	44.40	34.50	32.33
Shrinkage Limit, (%)	27.70	10.70	14.22
Plasticity Index, Ip (%)	20.70	10.27	7.00
Clay, (%)	27.34	10.34	14.33
Sand, (%)	1.14	6.60	7.00
Silt, (%)	71.52	83.06	78.67

3.2 Compaction Behavior

The behaviors of compaction for three samples were determined in the laboratory. Three tests were performed for each sample. Standard Proctor Tests were used. To prepare compaction curve, maximum dry density points for each sample prepared over a range of water contents were plotted and a curve was drawn between the points. The maximum dry density or dry unit weight occurred at optimum water content. The main reason for developing a compaction curve was to determine the optimum water content and maximum dry density. The compaction properties of three clay samples are listed below in Table 2.

Table 2: Average Compaction Properties of Three Samples

Sample No.	1	2	3
Optimum Moisture Content, (%)	21.00	16.50	19.40
Maximum Dry Density, kN/m^3	15.00	17.10	14.93

3.3 Hydraulic Conductivity

Hydraulic conductivity is a main parameter of clay liner for the construction of landfill. It depends on several factors such as size of soil perticles, void ratio, degree of saturation, composition of soil particles, soil structure, viscosity of the permeant, density and concentration of the permeant etc.

Hydraulic conductivity is very much related to molding water content. The hydraulic conductivity of clay samples were measured using rigid wall perimeter under falling head condition. Liner soil should have at least 30% fines and 15% clay to achieve hydraulic conductivity in the range of 1 x 10^{-7} cm/s (Daniel, 1993).

Liner soil samples were found 71.52%, 83.06% and 78.67% of Silt, and 27.34%, 10.34% and 14.33% of clay, respectively. The hydraulic conductivities at optimum water contents without cement were found of 0.230 x10^{-7} cm/s, 13.20x10^{-7} cm/s and 15.30x 10^{-7} cm/s respectively. Among the soil samples, only the first sample can be used for natural barrier to achieve a hydraulic conductivity in the range of 1 x10^{-7} cm/s.

In the most common sense, cement is a binder that can bind other materials together. Cement admixtures increase the strength and reduce the compressibility of the liner. An added benefit of cement admixture is the reduction of the soil permeability. For improving the engineering properties of soil samples, 5% cement was added with soil samples to use as a natural barrier. When 5% cement was added with samples, the hydraulic conductivities were found to be 0.228 x10^{-7}cm/s at 39.00% water content, 8.30 x10^{-7}cm/s at 16.50% water content and 9.55 x10^{-7}cm/s at 19.40% water content for sample1, sample2 and sample3, respectively.

The Variation of Coefficient of permeability at various water content without cement and with 5% cement addition of three clay samples are listed below in Table 3.

Table 3: Co-efficient of permeability in different water content of soil samples(without and with 5% cement)

Cases	Sample No.	Water content (%)	Co-efficient of Permeability (cm/s)
Without Cement	1	19.50	0.204×10^{-7}
	1	21.00	0.230×10^{-7}
	1	23.50	0.812×10^{-7}
	1	28.50	2.500×10^{-7}
	1	30.00	1.900×10^{-7}
	2	16.50	13.200×10^{-7}
	3	19.40	15.300×10^{-7}
With Cement	1	39.00	0.228×10^{-7}
	2	13.50	11.500×10^{-7}
	2	16.50	8.300×10^{-7}
	3	19.40	9.550×10^{-7}

3.4 Performance Study

The leachate used in the performance study was collected from an active ultimate disposal site of MSW, namely, Rajbandh, at Khulna. Different parameters of raw Leachate such as pH, conductivity, alkalinity, hardness, sulphate, nitrate and phosphate were tested in the Environment Laboratory, KUET, Khulna. The test setups were kept under observation until sufficient amount of infiltrated leachate was collected. In case of third sample, leachate was started to infiltrate and collected after 15 days and 30 days for second sample. But in case of first sample, 45 days were required to start infiltration. Because first soil sample consisted of high clay content. The charecterictises of infiltrated leachate were tested and the measured data were analyzed and compared with raw Leachate as presented in Table 4.

Table 4: Different parameters of raw leachate and infiltrated leachate from three samples

Test Parameters	**Sample 1**	**Sample 2**	**Sample 3**	**Raw leachate**
pH	7.34	7.25	7.10	7.66
Electro conductivity, (mS/cm)	4.00	6.42	7.28	6.88
Alkalinity, mg/l	200.00	100.00	80.00	1180.00
Hardness, mg/l	226.87	185.20	180.40	171.31
Sulphate, mg/l	30.40	43.20	55.80	50.00
Nitrate, mg/l	0.10	0.30	0.50	3.00
Phosphate, mg/l	0.38	0.52	0.81	10.60

3.4.1 Potentiality of clay layer

When raw leachate was placed on the liner systems, infiltrated leachate was able to move through the cement admixture. But leachate infiltration rate through the clay layer was also found very low. So, the performance of the clay liner system was found from the infiltrate leachate quality test comparatively better than raw leachate. But hardness of infiltrated leachate was increased than raw leachate. Because infiltrated leachate carries some chemical agents for destroying various components of the cement such as calcium. So, the soil samples can be used as the hydraulic barrier for the waste containment facilities.

4. CONCLUSIONS

From this study the following conclusions can be made:

- The first sample satisfied all the General requirements of clay liners without mixing cement with it whereas the second and third samples did not meet the acceptable limits.
- When 5% cement was mixed with soil samples, the properties of second and third samples were improved to an acceptable limit.
- From performance study in the laboratory test setup it can be observed that in case of first and second samples the concentration of harmful leachate constituents were more reduced than third sample because of low hydraulic conductivity.
- First and second samples with 5% cement can be used as liner system.

RECOMMENDATIONS

- Further study should be carried out by varying the different percentage of cement with various soil samples.
- The test setup methods can be improved by using a layer of cement and sand mixture between two clay layers and a sand layer at the top.

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MODEL STUDY TO IMPROVE LATERAL LOAD RESISTANCE OF PILES IN SAND USING GEOTEXTILE

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ABSTRACT

Lateral loads in pile come from earth pressure, wind, waves and earthquake extensively. Although piles are normally subjected to axial loads, it should have the capacity to resist that lateral loads. This paper presents the improvement of lateral load resistance of piles in soil. Laboratory model tests were done to find out lateral resistance capacity of pile under varying conditions such as number of pile, embedded pile length and pile spacing both for normal cohesion less soil medium and improved conditions placing geo-textile into the soil surrounding the pile in single layer and double layer. The load-displacement responses make clear the improvement of lateral load resistance of pile using geo-textile in every test. Lateral load resistance of pile is also increases with the increases of geo-textile layers in the foundation medium.

Keywords: *Pile-soil interaction, lateral loads, geotextile, group pile.*

1. INTRODUCTION

Deep foundations are used in high rise buildings and heavy structures if the bearing capacity of foundation materials is not available to carry out the total load of the super structure. Pile foundation is one particular type of deep foundation commonly used for large structures such as tall buildings, bridges, dams etc. A huge amount of structural load of those structures is carried by pile groups. In addition to the structural load, a significant amount of lateral loads may be present. These lateral loads can come from variety of sources such as wind forces, collisions, wave or ice impact, earthquake shaking, liquefaction and slope failure. The sources of lateral load on harbor structure are impact of ship and wave action and of shore structure are also subjected to wind and wave. High rise building, tower are subjected to lateral load due to wind and earthquake forces. Therefore, the structural load as well as lateral load capacity is certainly significant attribute in the design of piles under certain construction situations. From the previous studies, it is observed that a number of researches are available that describe the lateral bearing capacity of pile. Terzaghi (1955) took an attempt to rationalize the pile resistance by using a variable passive coefficient, K_{pm}, which is a function that depends on mobilized angle of shearing resistance. In 1960's, ultimate lateral load resistance advanced for rigid piles by assuming that the full passive Rankine earth pressures were mobilized. According to the method proposed by Brinch Hansen (1961), the pile is assumed to rotate about a single point, the ultimate lateral load is calculated and the shear force and bending moment diagrams are drawn. Broms (1964) described the methods to determine the ultimate lateral load in cohesive and cohesionless soils. Kasch et al. (1977), state that use of Rankine's passive states will result in a very conservative solution. In recent years, more extensive research and developments have been undertaken to predict theoretically the behavior of laterally loaded piles in different types of soil (Poulos and Davis, 1980; Meyerhof et al., 1981; Reese, 1984; Gandhi, and Selvam, 1997; Prasad and Chari, 1999; Patra and Pise, 2001; Rahman et al., 2003, Zhang et al., 2005). However very few researches (Matlock and Reese,1960; Maeda, et al., 2001; Maeda, et al., 2006, Maeda, et al., 2007;) are carried out to improve the lateral resistance of pile in loose and unstable foundation medium using various techniques. Most of the methods were not economical to apply in practical field. For that here we introduce a new technique to improve loose and unstable foundation medium which is more economical for field works. We used geotextile surrounding the pile in a single and double layers and find out the resistance of single and group piles against lateral load. The results are shown by plotting lateral load verses horizontal deflection curve where the failure of pile is indicated by continuous increase of horizontal deflection without any increase of lateral load.

2. MATERIALS AND METHODS

In this study, laboratory model experiment is done to investigate the lateral bearing capacity of pile under different conditions such as spacing of pile and embedded length of pile in a normal foundation medium as well as in the improved foundation using geotextile. To do this study, Padma river sand was collected to prepare the foundation medium and geotextile was used to improve lateral bearing capacity. A high strength Aluminium rod is used as a model pile. The laboratory experiment was done to find out the property of river sand and geotextile materials.

2.1 Materials

In this study, Padma River sand was used to prepare the foundation medium. Dry density of the river sand is 1640 kg/m^3 and the grain size distribution curve of the sand sample is shown in Figure 1. The properties of geotextile were also finding out by laboratory experiment as shown in Table 1. Hollow aluminum pipes of same diameters (2.5 cm) were used as model piles in this study. Two different lengths of piles (37.5 cm and 50 cm) were used to investigate the effect of pile length. Steel plate of uniform thickness (1 cm) was used as pile cap. A hollow concrete block of size 1m × 1m × 1m and wall thickness of 1 inch was used as a model test tank. The model test tank was filled by dry river sand which was used as foundation medium.

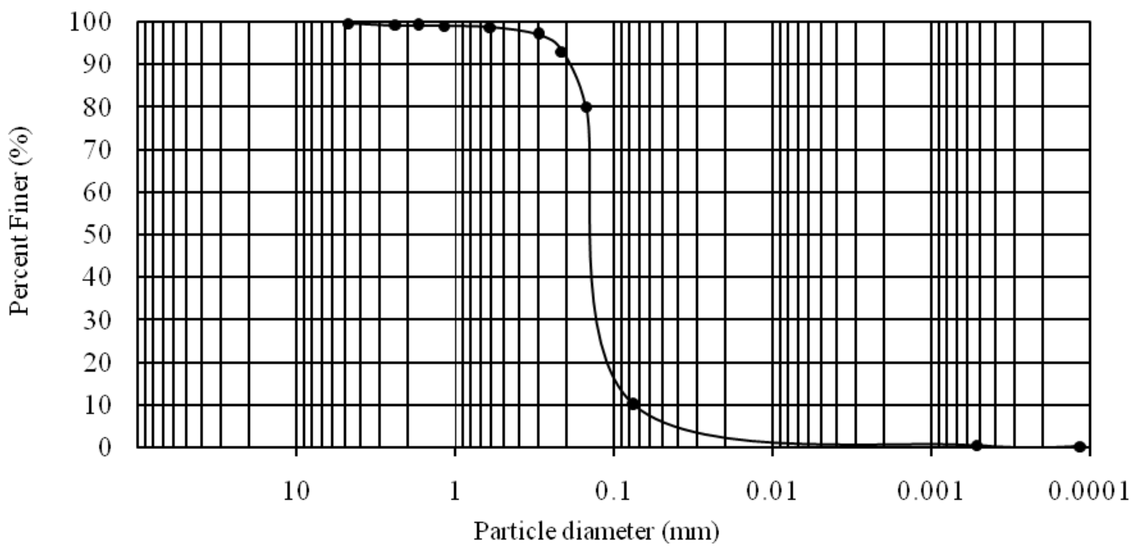

Figure 1: Grain size distribution of foundation materials

Table 1: Properties of Geotextile

Geotextile Properties	Unit	Test Value
Mass per unit area	gm/m^2	312.50
Thickness (Under 2 kPa pressure)	mm	3.06
CBR Puncture Resistance	N	3258
Effective Opening Size	mm	<0.075
Strip Tensile Strength (Machine Direction)	kN/m	26.11
Strip Tensile Strength (Cross Machine Direction)	kN/m	13.50

2.2 Experimental Procedure

At first, model piles were placed in the tank according to our requirement (embedded length of pile, number of pile and spacing of group pile) then the tank was filled by the dry river sand of same properties. To maintain a uniform density of the foundation medium in the tank, air pluviation technique is used. After filling the tank, the upper surface of sand was leveled and then pile cap was fixed on the pile top. For applying lateral load in the group pile a weir was attached in the pile cap and a vertical stand with frictionless pulley was used to change the direction of vertical load to lateral load. A high sensitive dial gauge was attached to the pile cap to calculate the lateral deflection of the pile. Load was applied gradually. The total experimental setup is shown in Figure 2. In case of understanding the improvement of lateral bearing capacity, geo-textile is used as a single layer and double layer in the foundation medium. Geotextile was provided at a distance 0.15 m from the pile face towards the horizontal loading direction. In case of double layers geo-textile, geo-textiles were placed at a regular interval 0.15 m from first layer of geo-textile towards loading side. Dial gauge having sensitivity of 0.001mm was used to calculate the lateral deflection. When load was placed in loading pan, the piles were being deflected in the direction of loading and dial gauge gave the reading of deflection of pile. Deflections for corresponding load were noted. By noting deflections values; load verses deflection curve was drawn. Ultimate lateral resistance of the pile is obtained from the curve where this curve shows a greater deflection without further increasing any load.

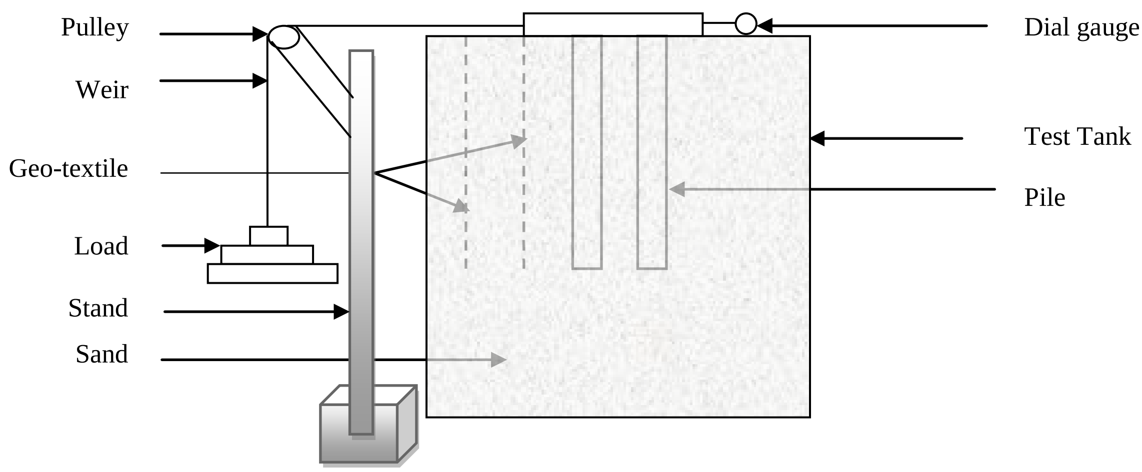

Figure 2: Schematic diagram of test setup

2.3 Group Pile Arrangement

Two types of group piles were used to make clear the effect of geotextile on the improvement of lateral load resistance. Arrangements of group pile are pile group [2×1] and pile group [2×1+1] is shown in Figure 3. Two different pile spacing (1.5 times and 3.0 times of pile diameter) and two different pile lengths (45 cm and 60 cm) were used for each pile group where the diameter of each pile was 2.50 cm.

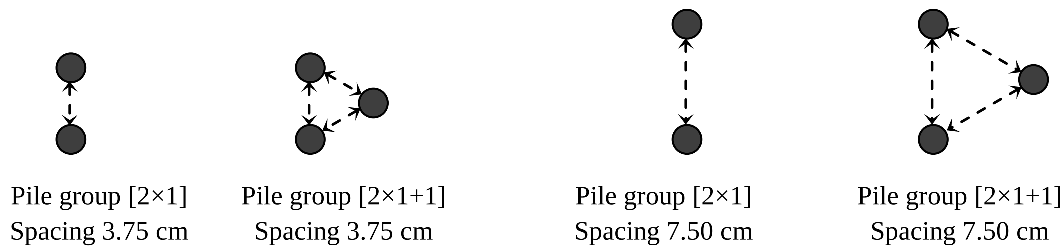

Figure 3: Arrangement of group pile in the foundation medium.

3. RESULTS

The improvement of lateral bearing capacity using geotextile was investigated in this study. To evaluate the improvement of bearing capacity, results were compared with when geotextile were not used in the foundation medium under different condition like single pile, group pile, pile spacing and embedded length of pile.

3.1 Horizontal loading in Single Pile

In the horizontal loading test at first single pile was used without improvement of foundation medium, then the tests were again done using single layer and double layer of geotextile in the foundation medium. Figures 4(a) and (b) show the relationship between horizontal loads and lateral deflection of single pile for different pile length like 45 cm and 60 cm respectively when other conditions remain same. It is noted that load carrying capacity of pile increases for providing geotextile in the direction of lateral loading of the pile. Load carrying capacity also increases with the increase of number of geotextile layers. Moreover, the load resisting capacity of a single pile of length 45 cm is less than the pile of 60 cm in individual cases. Therefore it clarifies that lateral load carrying capacity increases by embedded length of pile.

(a) (b)

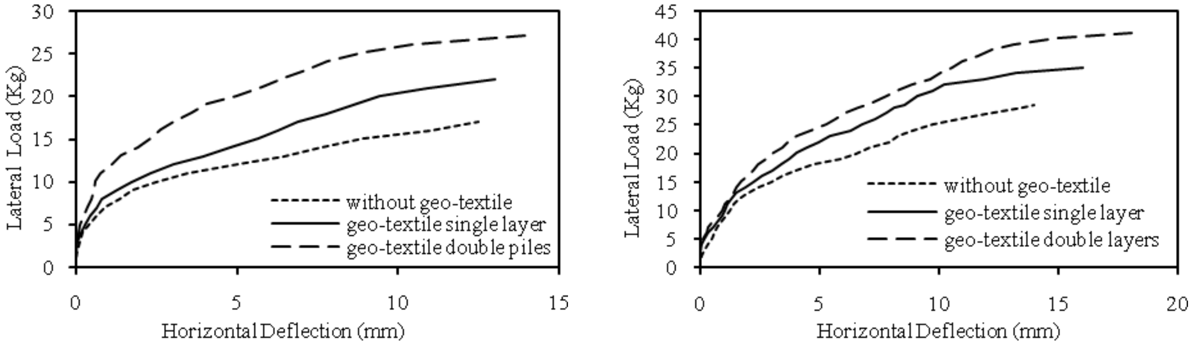

Figure 4: Variation of load with horizontal deflection for single pile when pile length (a) 45 cm and (b) 60 cm.

3.2 Horizontal loading in Group Pile

(a) (b)

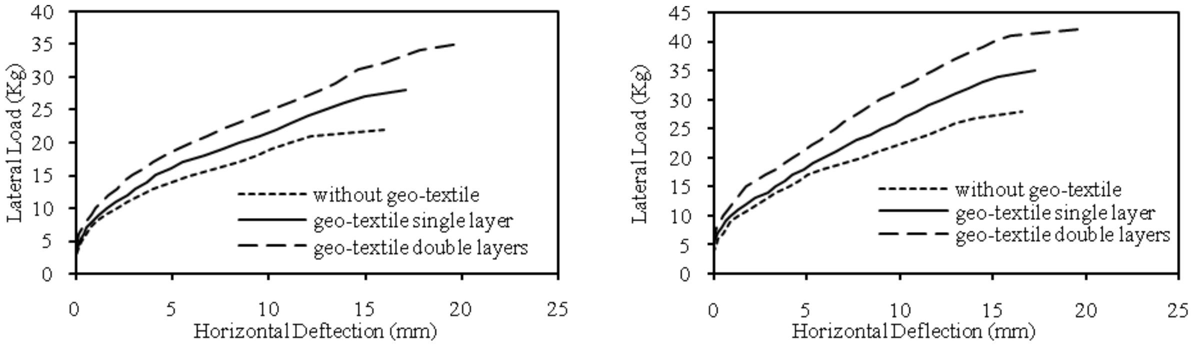

Figure 5: Effect of geo-textile for group pile [2×1] for pile embedded depth 45 cm when spacing (a) 3.75 cm and (b) 7.50 cm.

Figure 5(a) depicts the relationship between lateral loads and horizontal deflection of a group pile [2×1] for a pile spacing of 3.75 cm and embedded depth of pile 45 cm, while Figure 5(b) represent the variation of lateral load carrying capacity with horizontal deflection for a pile spacing of 7.50 cm. From these figures, it clear that the lateral load carrying capacity of pile increases by providing geotextile surrounding the pile wall in single and double layers. It is also noted that when pile spacing of a group pile is 1.5 times of pile diameter has less lateral load resistance capacity than a group pile having pile spacing 3 times of pile diameter.

(a) (b)

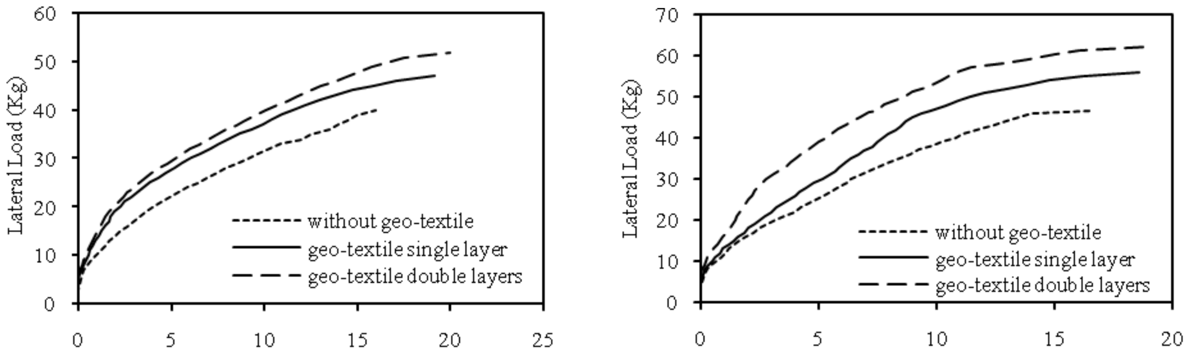

Figure 6: Lateral load and deflection relationship of group pile [2×1] for pile embedded depth 60 cm when spacing (a) 3.75 cm and (b) 7.5 cm.

Figures 6(a) and (b) illustrate the relationship between loads and horizontal deflection of group piles [2×1] for spacing of pile is 3.75 cm and embedded length 60 cm while Figure 6(b) shows the relationship of lateral load and lateral deformation pile spacing 7.5 cm. Both figures indicate that the load carrying capacity of pile increases with the increase of geotextile layer in the improved foundation medium and it is also noted from the comparison of two figures that with the increase of pile spacing, lateral stability also increases.

(a) (b)

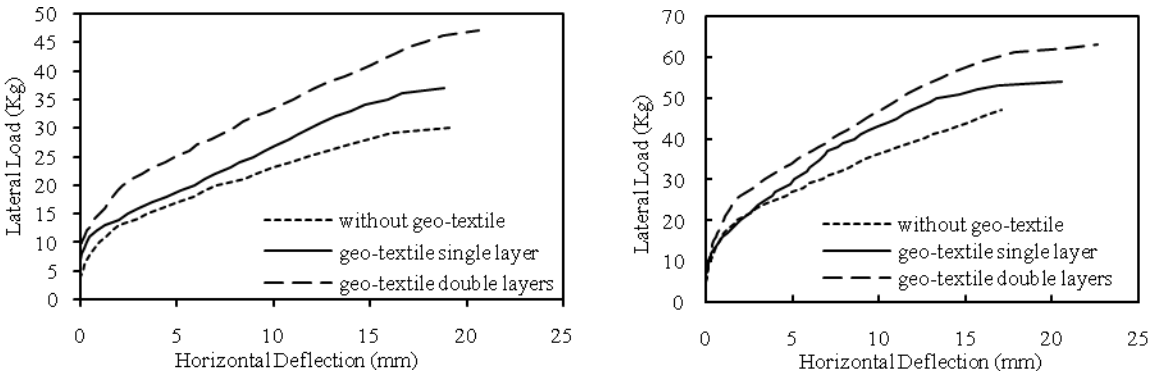

Figure 7: Relationship between lateral load and horizontal deflection of group pile [2×1+1] for pile spacing 3.75 cm and embedded length of pile (a) 45 cm and (b) 60 cm.

Figures 7(a) and (b) show the relationship between lateral load and horizontal deflection of group pile [2×1+1] for a pile spacing of 3.75 cm when embedded length of pile 45 cm and 60 respectively. With the comparison of two figures it clear that the load carrying capacity of 60 cm length pile group is higher than the pile group of 45 cm when other conditions remain same. Moreover, it is noted from individual figure that the stability against lateral load increases when surrounding soil is improved by geotextile and also increases with the increase of number of geo-textile layers.

(a) (b)

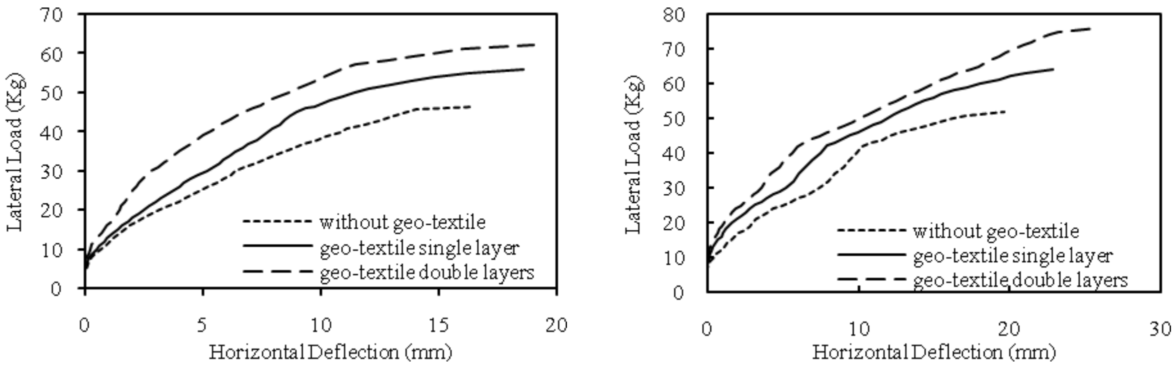

Figure 8: Variation of lateral load resistance with horizontal deflection for pile spacing 7.50 cm and embedded length 60 cm of (a) group pile [2×1] (b) group pile [2×1+1].

Figure 8(a) show the relationship between lateral load resistance and horizontal deflection when pile spacing 7.50 cm and embedded length 60 cm for group pile [2×1] while Figure 8(b) for group pile [2×1+1] when other conditions remain same. From the assessment of these figures, it understandable that the lateral load carrying capacity of group pile [2×1+1] is higher than the group pile [2×1] that means with the increase of number of pile stability to resist lateral load increases. In addition, it is clear from each figure that the stability against lateral load can be improved using geotextile surrounding the pile in soil.

4 CONCLUSIONS

In this study, laboratory model tests were done to find out lateral stability of pile under varying conditions in the normal foundation medium and in improved foundation medium using geotextile into the soil surrounding the pile in single layer and double layer. The following conclusions are drawn from the relationship between lateral load and displacement of this study:

a. Lateral load carrying capacity increases with the increase of number of pile and embedded length of pile.

b. Lateral stability of pile is higher when the pile spacing is three times of pile diameter instead of 1.5 times of pile diameter.
c. Lateral load resistance of pile in a normal foundation medium can be improved using geotextile surrounding the pile.
d. With the increases of geotextile layer surrounding the pile or pile group, increases the stability against lateral load.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

MODEL STUDY ON THE BEHAVIOUR OF PILE IN SAND SUBJECTED TO LATERAL LOAD

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ABSTRACT

Piles are normally subjected to axial loads. However, in many cases lateral loads have also to be transferred in addition to axial loads. Till now, it is not clear how important the interaction effects of lateral load, dependent on various factors such as Number of pile, embedded pile length, spacing of pile, Size of pile, soil type, loading conditions etc. In this paper the effect of number of pile, embedded pile length and pile spacing on the lateral load resistance capacity of pile is investigated by doing a set of laboratory experiments on model pile. To do the laboratory experiments Aluminum model piles of diameter 2.5cm have driven into dry river sand of same density varying pile number, pile length and pile spacing. In every laboratory experiment a load-displacement relation is obtain. The load-displacement response makes clear the lateral load resistance of pile or pile group in every condition. Lateral load carrying capacity increases with the increases of number of pile as well as embedded pile length. Lateral load resistance also increases when pile spacing increases from 1.5 to 3.0 times of pile diameter.

Keywords: *Lateral resistance, embedded pile length, group pile, pile spacing, lateral displacement, number of pile.*

1. INTRODUCTION

Pile foundation is used to transfer a load from a structure through an upper weak layer of soil to a stronger deeper layer of soil and to resist a significant lateral load like others deep foundation. The lateral load resistance of pile foundations is an important factor in the design of structures which may be subjected to earthquakes, high winds, wave action, and ship impacts. To keep the structures safe against lateral load, it is a grand design factor for the construction under such condition. As conducting lateral load tests on pile groups is logistically difficult and cost is too high very few studies was performed in past. From the previous studies, it is observed that few researches are available that show the distribution of load within a pile group (Broms, 1964; Anagnostopoulos, and Georgiadis, 1993; Gandhi, and Selvam, 1997; Rahman et al., 2003; Zhang et al., 2005). It is observed from these tests that the average load for a pile in a closely spaced group will be substantially less than that for a single isolated pile at the same deflection and that leading pile or group piles carry significantly higher loads than trailing row piles at the same deflection. So these tests give a confusing result of the effect of Pile number and it is not clear yet. Like pile number another important term the effect of embedded pile length on the lateral load resistance of pile foundation and we did not have a clear result yet. Pile spacing is also an important factor which has a significant effect on lateral load resistance of pile which is not understandable from previous studies. To improve our understanding of pile group behavior as a function of pile number a series of lateral load tests were performed on four different pile, single, double, triple and tetra. Similar lateral load tests were also performed on group and single piles of pile length (L) 45 cm and 60 cm respectively to make clear our understanding of embedded pile length effects on lateral load resistance. To make clear our concept about effect of pile spacing three pile groups at spacing of 1.5 and 3 times of pile diameters (D) were used.

2. MATERIALS AND METHOD

In this study laboratory model experiments were done to investigate the effect of number of pile, embedded length of pile and spacing of pile on the lateral load resisting capacity in a foundation. To do this study, Padma river sand was collected to prepare the foundation medium. The laboratory experiments were done to find out the property of river sand materials. A high strength hollow Aluminium pipe was used as a model pile.

2.1 Experimental Materials

A concrete block of (1m x 1m x 1m) wall thickness 2.5 cm was used as a model test tank. The model test tank was filled by dry river sand which was used as foundation medium. The dry density of sand and specific gravity were determined by laboratory experiment. The angle of internal friction of the sand was determined by direct shear test method in the laboratory. All the test results are shown in Table 1 below. Hollow aluminum pipes of same diameters (2.5 cm) were used as model piles for the test. Two different lengths of piles (45 cm or 60 cm) were used to investigate the effect of pile length. Steel plate of uniform thickness (0.6 cm) was used as pile cap.

Table 1: Properties of sand

Specific gravity	Dry Density	Angle of internal friction
2.63	1640 kg/m^3	39°

2.2 Experimental Setup

At first, model piles were placed in the tank according to demand (embedded length, spacing and number of pile) then the tank was filled by the dry river sand of same properties as shown in Table 1. To maintain a uniform density of the foundation medium in the tank, sand was placed by air pluviation technique. The relative density of the sand box was kept 50% for all tests to maintaining a consistent density. After filling the tank, the upper surface of sand was leveled and then pile cap was fixed on the pile top. For applying lateral pull in the pile a weir was attached in the pile cap and a vertical stand with frictionless pulley was used to change the direction of vertical load to lateral load. One dial gauge was attached to the pile cap to measure the lateral deflection of the pile. Load was applied step by step. The whole arrangement is shown in Figure 1. Dial gauge having sensitivity of 0.01mm was used to calculate the lateral deflection. When load was placed in loading pan, the piles were being deflected in direction of load and dial gauge gave the reading of deflection of pile. Deflections for corresponding load were noted. By noting deflections values; load verses deflection curve is drawn. Ultimate lateral resistance of the pile is obtained from the curve where this curve shows a greater deflection without further increasing any load.

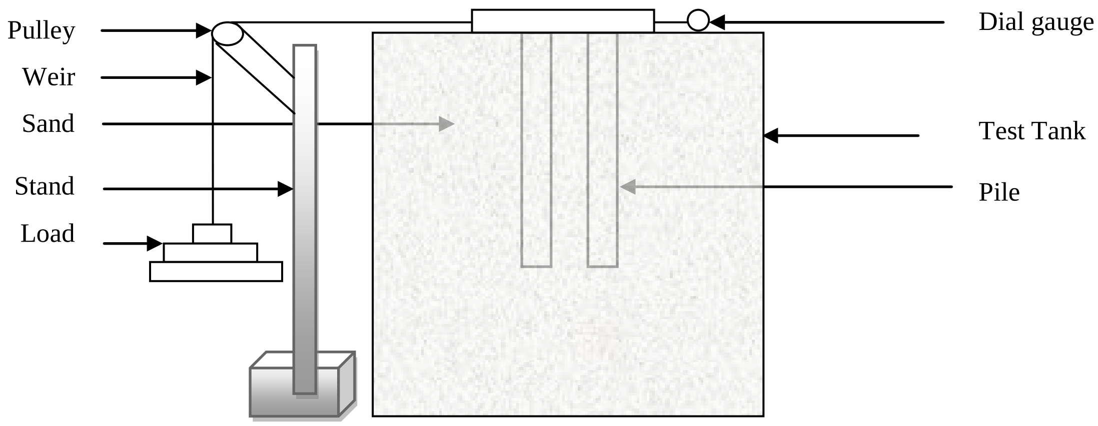


Figure 1: Schematic diagram of test setup

3. RESULTS AND DISCUSSIONS

The results are analyzed by plotting typical graphs for the variation of lateral displacement with lateral load for all the testing conditions. The load-displacement curves are, in general, similar and nonlinear. The lateral loads versus lateral displacement diagrams are drawn to study the effect of number of piles, pile length and pile spacing.

3.1 Effect of Number of Piles

Lateral load versus horizontal deflection curves were plotted for Single and three different types of pile group to analyze the effect of Number of piles. Single, double [2×1], triple [2×1+1] and tetra [2×2] piles were used as four different pile groups. The results obtained from the test are shown in figure 2.

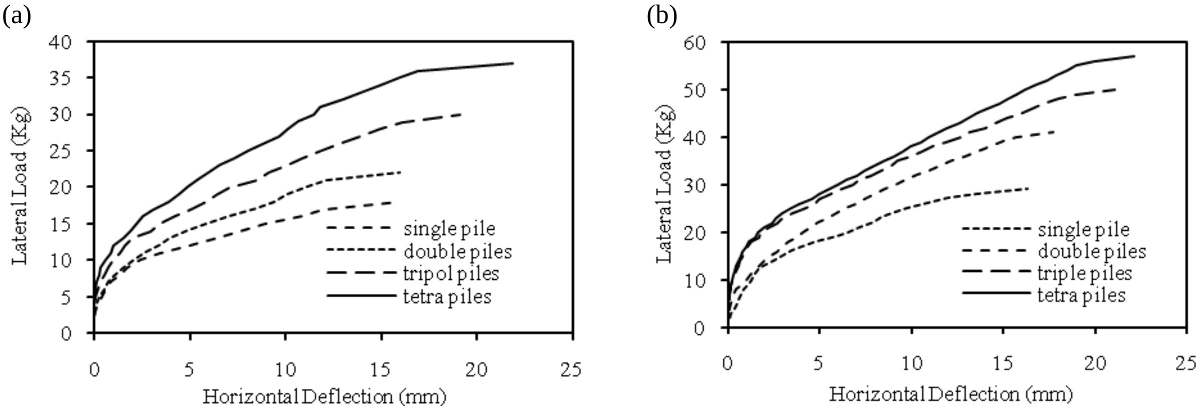


Figure 2: Effect of number of pile when embedded pile length (a) 45 cm and (b) 60 cm.

Figure 2(a) and (b) describe the relationship between load and horizontal deflection for different pile length. These figures shows that the lateral load gradually enhanced with the increase of number of piles but it not uniform that means the lateral resistance of group pile [2×2] is not double of group pile [2×1] and lateral resistance of group pile [2×1] is not double of single pile. In both cases when the embedded pile length 45 cm and 60 cm lateral load resistance increases with increase of pile number in a group.

3.2 Effect of Embedded Length of Pile

To study the effect of embedded length of piles on the lateral load carrying capacity, Aluminum model piles of different lengths such as 45 cm and 60 cm were used. Four different types of pile arrangement such as single

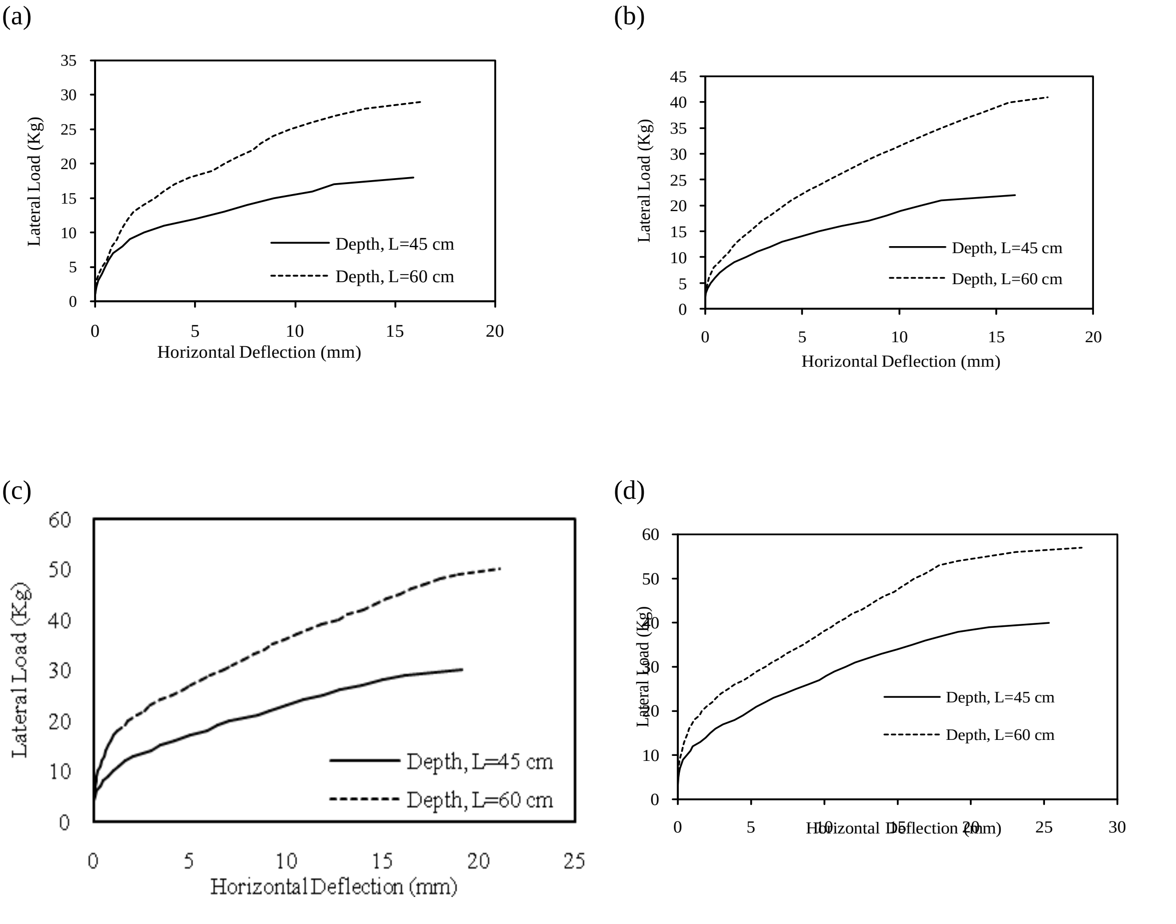
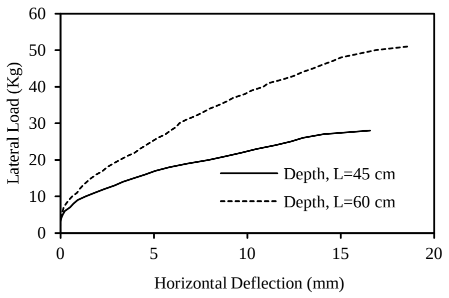


Figure 3: Variation of lateral load with lateral deflection for different group pile condition such as (a) Single pile (b) group pile [2×1] (c) group pile [2×1+1] and (d) group pile [2×2].

pile, group pile [2×1], triangular group pile [2×1+1] and square group pile [2×2] were used to study the effect of embedded depth of pile. The results are shown in figures 3 and 4.

Figure 3(a) illustrates the relationship between loads and horizontal deflection of single pile for different embedded lengths of pile while Figure 4(b), (c) and (d) show the lateral resistance of group pile [2×1], [2×1+1] and [2×2] respectively where the spacing of pile is 3.75 cm. In every case, it is observed that the load carrying capacity of 45 cm pile is less than 60 cm pile.

(a)



(b)

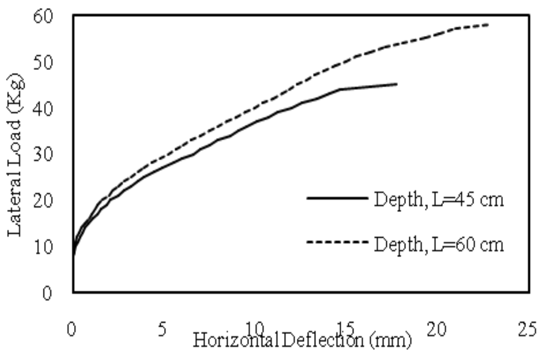


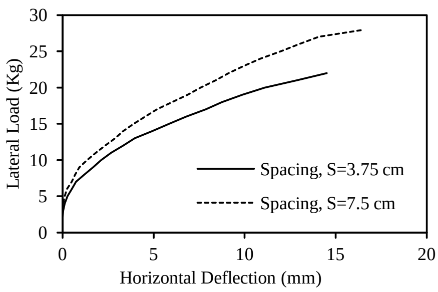
Figure 4: Lateral load verses lateral deflection relationship of (a) group pile [2×1] and (b) group pile [2×1+1] for spacing of pile 7.5 cm.

Figure 4(a) depicts the relationship between lateral loads verses lateral deflection of group pile [2×1] and [2×1+1] respectively where the spacing of pile was 7.5 cm. It is observed that with the lateral load resistance of group pile is higher when the embedded length is 60 cm. Therefore it is clear from the investigation of effect of embedded pile length that lateral load carrying capacity increases with the increase of length of pile stayed in the foundation medium.

3.3 Effect of Pile Spacing

To understand the lateral load carrying capacity of group pile as a function of pile spacing, a series of lateral load tests were performed. In that case, two types of group piles [2×1] and [2×1+1] were used for analyzing the effect of pile spacing. At the first time, the spacing of pile in both cases of group pile was 1.5 times of pile diameter and another cases pile spacing was 3.0 times of pile diameter. To maintain different pile spacing in each group pile, two different spaced pile caps were used.

(a)



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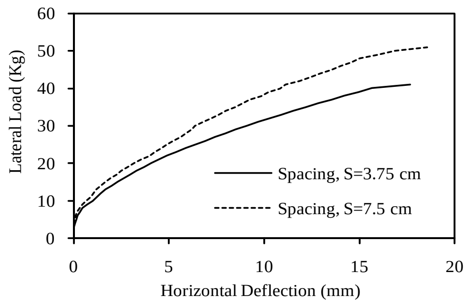

Figure 5: Variation of lateral load with horizontal deflection curve of group pile [2×1] for pile length (a) 45 cm and (b) 60 cm.

Figure 5(a) and (b) show the relationship between lateral loads with horizontal deflection when pile spacing is asymmetrical. These figures make clear that the lateral load carrying capacity of group pile [2×1] progressively increases with the increases of pile spacing from 3.75 cm to 7.5 cm in both cases when pile length 45 cm and 60 cm.

Figure 6(a) and (b) depicts the relationship between loads verses horizontal deflection of group piles [2×1+1] when pile spacing is different. It is noted that the load carrying capacity of group piles [2×1+1] is less when pile spacing is 3.75 cm and it increases with rise of pile spacing up to 7.5 cm in both condition of embedded pile length 45 cm and 60 cm. This tendency is similar with the previous Figure 5(a) and (b). Therefore it is clear that the lateral load resistance of piles increases with the increases of pile spacing up to 3.0 times of the pile diameter.

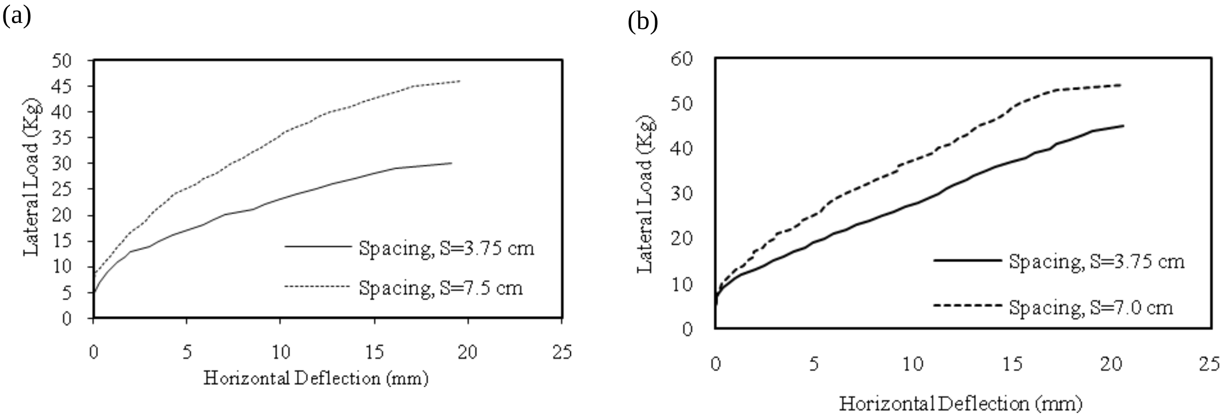

Figure 6: Variation of load with horizontal deflection of group pile [2×1+1] when embedded length of pile (a) 45 cm and (b) 60 cm.

4. CONCLUSIONS

In this study, laboratory model tests were carried out to find out lateral load resistance of pile foundation under varying conditions such as number of pile, embedded length of pile and pile spacing in the foundation medium of dry river sand. From this study, the following conclusions are drawn from the relationship between lateral load and displacement:

i. Horizontal load carrying capacity of piles increases with the raise of number of pile.

ii. The Horizontal load capacity of pile increases with increase in pile length for same diameter since passive resistance is mobilized on increase in embedment depth of pile.

iii. The ultimate lateral resistance of pile group increases with an increase in pile spacing. It has been found that lateral load resistance at spacing 1.5 times pile diameter is less than that 3.0 times of pile diameter.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EFFECT OF ANCHOR SIZE AND CABLE DIAMETER ON ANCHORED EARTH RETAINING SYSTEM.

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ABSTRACT

A parametric study was undertaken to investigate the behavior of a new type of anchored earth wall system using a three dimensional explicit finite difference program, FLAC 3D. A 5m high and 4m wide anchored earth wall supporting a simultaneously constructed roadway was considered as standard wall and modeled using FLAC 3D. The wall system consists of reinforced soil and retained soil, four layer of reinforcing steel in both horizontal and vertical direction as cable element and concrete block as anchor. The parameters studied were (i) anchor size (ii) cable diameter. Emphasis was placed to investigate the variation of (i) mid section deflection of the wall (ii) stress in the cable (iii) pull in the anchor with the above mentioned parameter. The study revealed that anchor size and cable diamater has very little effect on the deflection of the wall, stress in cable and pull in the anchor. It was evident from the study that smaller size anchor and reinforcing steel(used as cable)having smaller diameter, provide better economy and serviceability.

Keywords: *Parametric study, anchored earth wall, anchor, cable, FLAC 3D*

1. INTRODUCTION

In the recent years many geotechnical construction like roadways and bridge abutments use retaining walls especially mechanically stabilized earth (MSE) walls. Among the MSE wall systems, Anchored Earth Wall is another type reinforced soil system, where the mode of stress transfer from backfill to reinforcement is by passive resistance in addition to friction, was patented by the Transport and Road Research Laboratory of United Kingdom in1981(Ali et al., 2008a). In anchored earth systems, a combination of the techniques used in the reinforced soil and the soil anchoring (Yoo and Lee, 2003) is utilized.

Although the design, analysis and construction techniques or approaches in the field of reinforced soil wall systems have been developed over the years (Bathurst and Simac, 1994; Collin, 1997; Elias and Christopher, 1997; Leshchinsky, 1993), the basic design methodology remains the same which is the limit equilibrium method. In design and analysis of reinforced earth wall, the limited equilibrium technique has been used from the first time while the reinforced earth was commercially constructed (Vidal, 1978).

Recent days have observed an increasing trend of construction of anchored earth wall system over other conventional systems in many countries of the world. Recently, a 13 m high wall is constructed in Malaysia (Lee and Oh, 1997). Anchored soil wall may be an alternative to conventional retaining walls with height above 3.5 m in Bangladesh. But the use of anchored walls in Bangladesh is impeded by various unavoidable reasons such as cost incurred by the unavailability of suitable backfill as well as lack of quality control of constructions etc. Therefore, the paper aims to formulate the parametric study of anchored earth wall supporting moderately compacted local soil using elasto – perfectly plastic Mohr-Coulomb model.

2. MODEL GENERATION

FLAC 3D, a 3D explicit finite difference programme is used to generate the proposed model. The software is able to simulate the behavior of structures built of soil, rock or other materials that undergo plastic flow when their yield limits are reached. The explicit, Lagrangian calculation scheme and the mixed discretization zoning technique used in FLAC 3D ensures that plastic collapse and flow are modeled very accurately.

A 5 m high and 4 m wide earth wall supporting a simultaneously constructed roadway is modeled as standard wall (Fig. 1). Horizontal and vertical spacing of cables is kept to 1 m and cable length is taken to be equal to the height of the wall. The length of the wall is taken as twice the length of the cable in the model and the concrete block having cross section 0.90 m x 0.90 m is used as anchor. It is assumed that Fill soil fills the space between the wall front and anchor and natural soil is considered behind the anchor. Reinforcing steel having diameter 32.26 mm is used as cable. Table 1, and Table 2 describes the properties of the materials used in the modeled wall.

Two types of structural elements – cable element and shell element are used in the model generation. Cable elements (reinforcing steel) have a deformable connection to the FLAC 3D grid along their length. Cable elements start from the face of the wall extending up to the anchor face. Frictions developed throughout the length of the cable retain the soil. To ensure this cable is extended beyond the potential failure plane. In the model shell element (concrete block) is used at the interface of fill soil and retained soil.

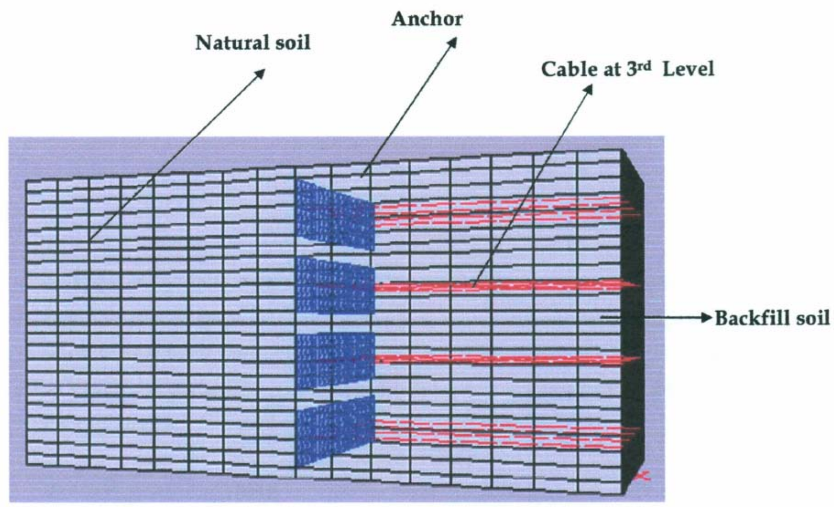

Figure 1. Geometry of the designed standard anchored earth wall system

Table1: Material properties of concrete and soil for the designed standard wall system

Properties	Unit	Concrete	Fill soil	Natural soil
Young's Modulus	MPa	24900	20	20
Poisson's ratio	-	0.25	0.45	0.45
Cohesion (c)	kPa	-	40	40
Friction angle (φ)	Degree	-	14.5	14.5
Dilatancy angle (ψ)	Degree	-	0	0
Density	kg/m3	2500	-	-

Table 2: Material properties of cable (reinforcing steel)

Density (kg/m3)	78.5×10^3
Young's Modulus (MPa)	199×10^3
Cohesive strength (N/m)	2×10^5
Friction Angle (Degree)	14.5
Stiffness (N/m)	1.75×10^7
Exposed perimeter (m)	0.1
Cross sectional area (m2)	819.4×10^{-6}
Compressive yield strength (N)	161×10^3
Tensile yield strength (N)	338×10^3

3. PARAMETRIC STUDY

A parametric study have been carried to examine the effect of variation of parameters such as anchor size, cable diameter, with and without surcharge on the behavior of the standard wall system to find out the optimum performance. Variations of these parameters are delineated in Table 3 - Table 4.

Table 3. Variation of anchor size in parametric study

	1	2	3	4	5
Anchor size (m^2)	0.50 x 0.50	0.60 x 0.60	0.70 x 0.70	0.80 x 0.80	0.90 x 0.90

Table 4. Variation of cable diameter in parametric study

	1	2	3	4	5
Cable diameter (mm)	25.4	28.65	32.26	35.81	43.00

4. RESULTS & DISCUSSION

4.1 Effect of variation of anchor size

Figure 2 and Figure 3 shows the variation of midsection deflection of the wall with height for different anchor sizes considering two cases without surcharge and with surcharge respectively. From Figure 2 and Figure 3, it is observed that midsection deflection of wall front is independent of anchor size regardless of surcharge. Maximum deflection occurs at mid depth of the wall. For better performance and economy anchor size of 0.6m x0.6m should be used. Comparison of Figure 2 and Figure 3 reveals that deflection increases with the application surcharge and it is prominent at the top of the wall.

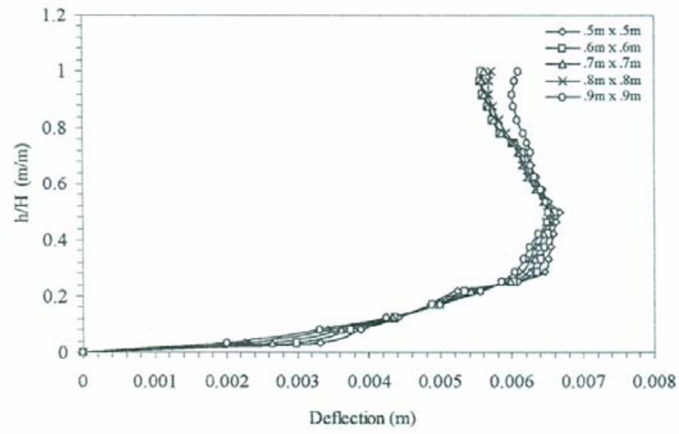

Figure 2. Variation of horizontal midsection deflection of wall front with height without surcharge (Parameter: anchor size)

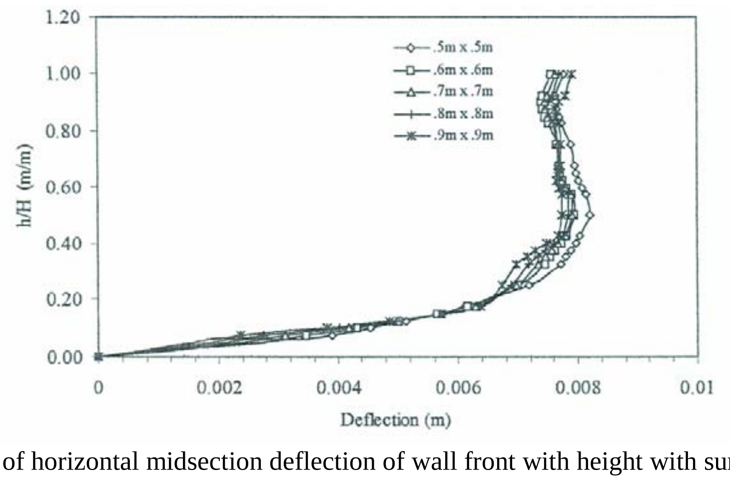

Figure 3. Variation of horizontal midsection deflection of wall front with height with surcharge (Parameter: anchor size)

In the analysis variation of cable stress along the length of the cable at third level of reinforcement is studied for variation of anchor size (Fig.4 and Fig.5). From the analysis it is clear that cable stress is independent of anchor size and it is very low near the face of the wall which gradually reaches maximum at mid length of the cable irrespective of application of surcharge. This is attributed to the fact that mid length of the cable falls beyond the assumed failure surface. For most of the length o the cable, cable stress is about 60% of the maximum cable stress which indicates that strength of the cable is fully utilized throght the length of the cable.

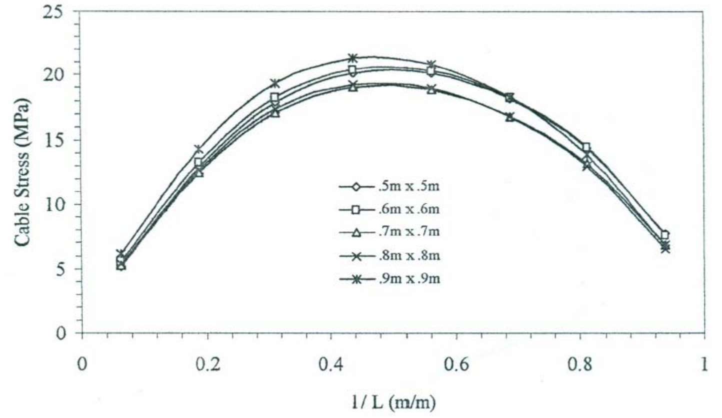

Figure 4. Variation of cable stress at 3rd level of reinforcement with length without surcharge (Parameter: anchor size)

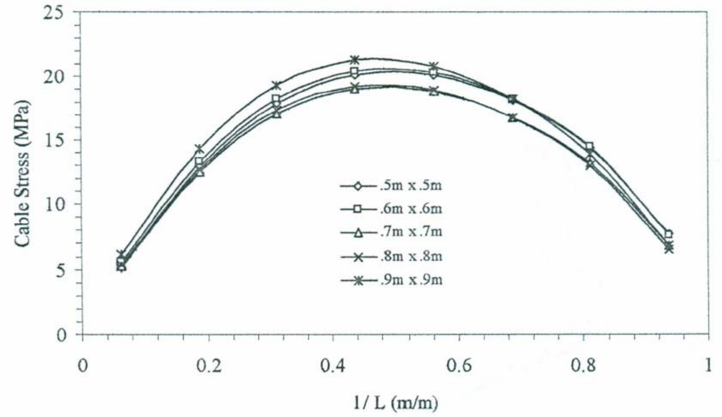

Figure 5. Variation of cable stress at 3rd level of reinforcement with length with surcharge (Parameter: anchor size)

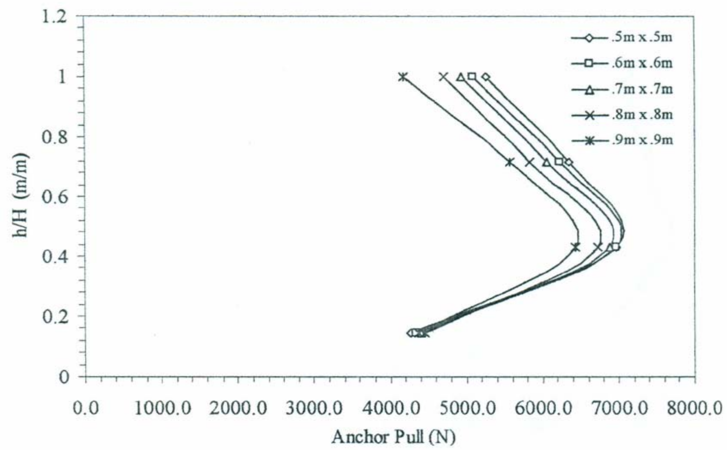

Figure 6. Variation of anchor pull with height without surcharge (Parameter: anchor size)

Figure 6 and Figure 7 show the variation of pull in the anchor with different anchor size, without and with surcharge respectively. It is evident that maximum pull occour for anchors located at mid depth of the wall and near the bottom of the wall, pull in the anchor is independent of the anchor size. It is also clear that pull in the anchor is less for anchors with bigger size but the variation in stress in different anchors is very little. So smaller size anchor will give better economy.

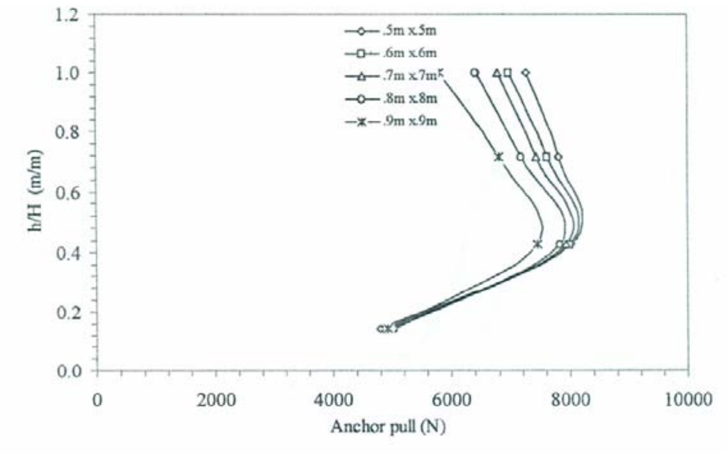

Figure 7. Variation of anchor pull with height with surcharge (Parameter: anchor size)

4.2 Effect of variation of cable diameter

The second parameter considered in the study is the cable diameter (listed in Table 4). It is observed from the analysis (Fig.8 & Fig.9) that cable diameter has little effect in the deflection of the wall front with height, for both the cases of without and with surcharge except for very large diameter cable. Larger deflection occurs at the top of the wall for larger diameter cable. So for better economy and performance smaller size cable having diameter less than 35.81mm should be used.

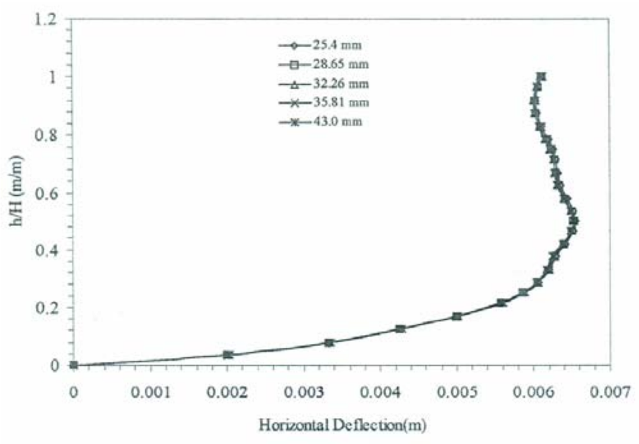

Figure 8. Variation of horizontal midsection deflection of wall with height without surcharge (Parameter: cable diameter)

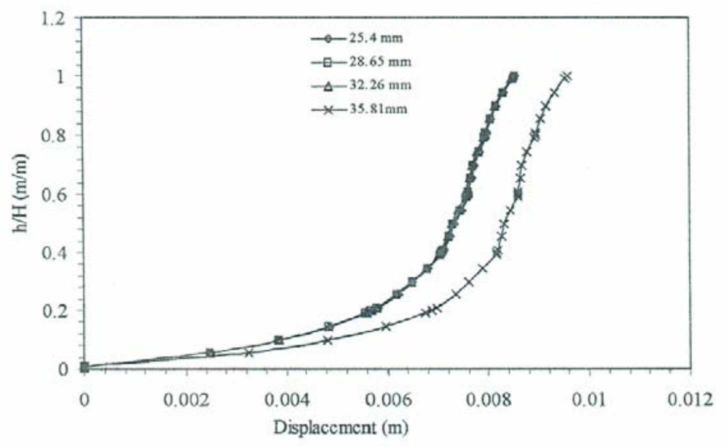

Figure 9. Variation of horizontal midsection deflection of wall with height with surcharge (Parameter: cable diameter)

Cable stress curves (Fig. 10 & Fig. 11) shows that cable stress varies in inverse proportion with cable diameter irrespective of surcharge. Maximum stress occur for smallest diameter cable (25.4 mm) which is about 50 Mpa when surchare load is applied. The tensile yield strength of cable is $338x10^3$ N which translates to 668 Mpa for 25.4 mm dia cable and is much higher than the maxium stress.

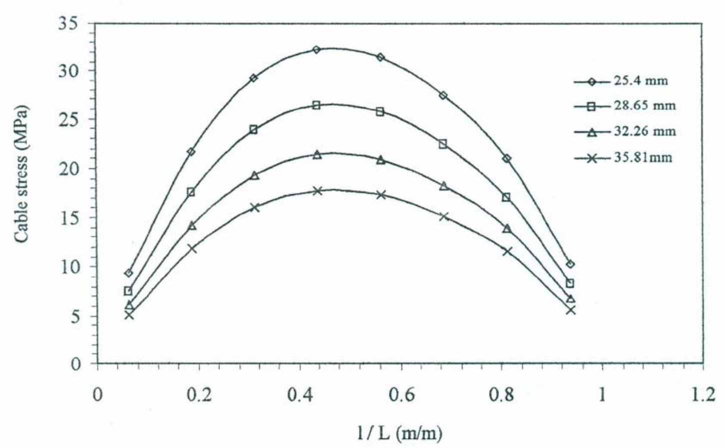

Figure 10. Variation of cable stress at 3rd level of reinforcement with length without surcharge (Parameter: cable diameter)

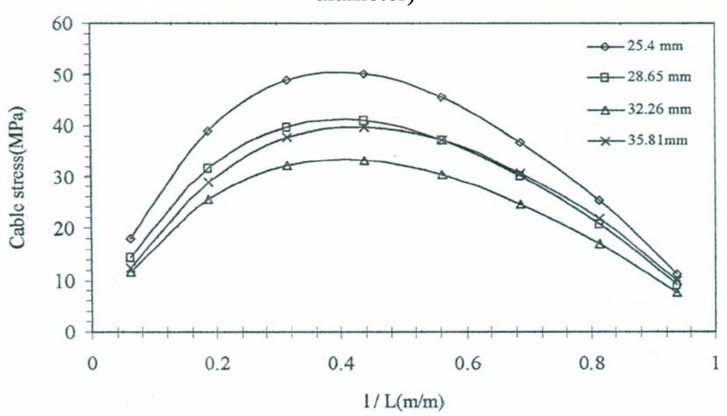

Figure 11. Variation of cable stress at 3rd level of reinforcement with length with surcharge (Parameter: cable diameter)

An analysis regarding anchor pull reveals that pull in the anchor increases with increase in cable diameter (Fig. 12 & Fig. 13). If we consider 43mm dia cable, maximum pull in the anchor would be 7000 N which will produce stress equivalent to 0.028 MPa for anchors having cross section of 0.5 m x 0.5m where as yield stress of concrete anchor block is 24900 Mpa. So, anchor having cross section less than 0.5 m x 0.5m gives better performance , factor of safety and economy. Application of surcharge increase the anchor stress leading to smaller value of factor of safey.

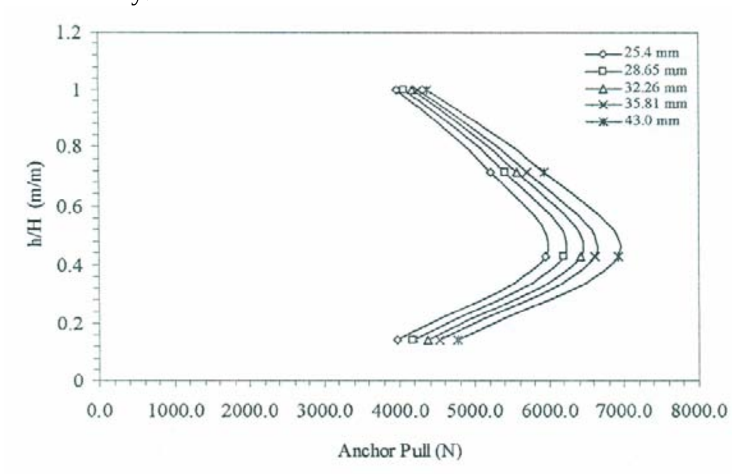

Figure 11. Variation of anchor pull with height without surcharge (Parameter: cable diameter)

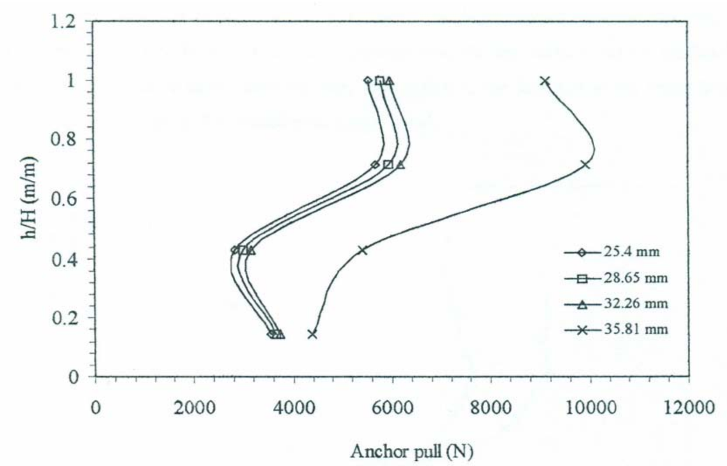

Figure 12. Variation of anchor pull with height with surcharge (Parameter: cable diameter)

5. CONCLUSIONS

In this study the behavior of anchored earth wall system is analyzed through parametric study using a three dimensional explicit finite difference program. The emphasis of the parametric study is placed on the effects of various components of proposed wall system on deflection patterns, cable stress & pull in the anchor. The study revealed that anchor size and cable diamater has very little effect on the deflection of the wall, stress in cable and pull in the anchor. It was evident from the study that smaller size anchor and reinforcing steel(used as cable)having smaller diameter, provide better economy and serviceability. For future study in line with the present observation, the following developments are envisaged:

- Instead of static and uniform surcharge dynamic and repetitive load should be incorporated to visualized its effects and to closely represent the condition that prevails in the road way.
- A comparative economic study may be performed to compare costs between proposed wall system and other alternatives.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

EVALUATION OF LIQUEFACTION POTENTIAL USING CONE PENETRATION TEST

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ABSTRACT

This paper presents liquefaction potential evaluation at six land development projects in Dhaka city. Bangladesh has been identified as a moderate vulnerable country according to world seismic map. The country sufferred five large earthquakes having a magnitude greater than 7.0 within and just outside the country from 1865 to 1930. Recent numerous small earthquakes within the country point to the high probability of occuring a large earthquake. Most of the private land development projects of Dhaka city rested mainly on sand deposits. The ground water table is also shallow for these locations. The sites are susceptible to liquefaction during ground-shaking. Infrastructures constructed on land fill has the potential to suffer from liquefaction. To identify liquefaction potential of such areas, in situ Cone Penetration Test (CPT) has been conducted for six locations of different land development projects. The liquefaction analysis based upon force equilibrium concept cannot be directly performed in this study, because there are no accelerometers deployed in those areas. The acceleration values back-calculated by liquefaction analysis are recommended in this article. The analysis have been carried out using CPT data that have been collected from the testing sites. The Peak Ground Aaccelerations (PGA) from 0.21 g to 0.35 g will generate liquefactions at all those locations.

Keywords: *CPT, Dhaka City, Earthquake, Liquefaction Potential, PGA.*

1. INTRODUCTION

Bangladesh is a seismically moderate country, Dhaka is its Capital. Dhaka City is centrally located in Bangladesh, in the southern part of the district of Dhaka. It is situated between latitudes 24º40´ N to 24º54´ N and longitudes 90º20´ E to 90º30´ E and bounded by Buriganga river in the south; the Balu and the Shitalakhya rivers in the east; Tongi Khal in the north and the Turag river in the west. The city has developed on the higher elevated Pleistocene terrace land of the central part of Bangladesh, otherwise referred to as the Madhupur-Bhawal Garh Region (Hossain, 2008). Present populations of this mega city are more than ten millions.

The legal and administrative boundaries of the city nowadays have however been extended to a great extent. A substantial portion of the adjoining low-lying areas have recently been brought under the structured zones of the city due to the accelerated rate of the urban growth in Dhaka. To cope up with ever-increasing pressure Dhaka has started going upwards, little scope exists for horizontal expansion due to topographical reasons. As a result, most of the development projects have been undertaken by land filling of low-lying marshy areas. This phenomenon is common for eastern fringe, western areas and southern periphery. According to Rahman (2010), these landfill areas are being developed on loose soils (mostly filled by sands) and developed gradually.

To identify the susceptibility of liquefaction for those new development project areas, liquefaction potential analysis has been done using CPT data for six locations. PGA value has been considered for a maximum size of Earthquake M_w 7.5. Figure 1 illustrates procedure of CPT concerning liquefaction potential evaluation. Proposed PGA map by Comprehensive Disaster Management Program (CDMP) has been used in order to calculate liquefaction potential (Figure 2). Figure 3 shows a typical CPT test being carried at a site in Dhaka.

2. METHODOLOGY

The CPT has major advantages over traditional methods of field site investigation such as drilling and sampling since it is fast, repeatable and economical. In addition, it provides near continuous data and has a strong theoretical background. These advantages have lead to a steady increase in the use and application of the CPT in many places around the world. One of the major applications of the CPT has been the determination of soil stratigraphy and the identification of soil type. This has typically been accomplished using charts that link cone parameters to soil type (Robertson, 2009).

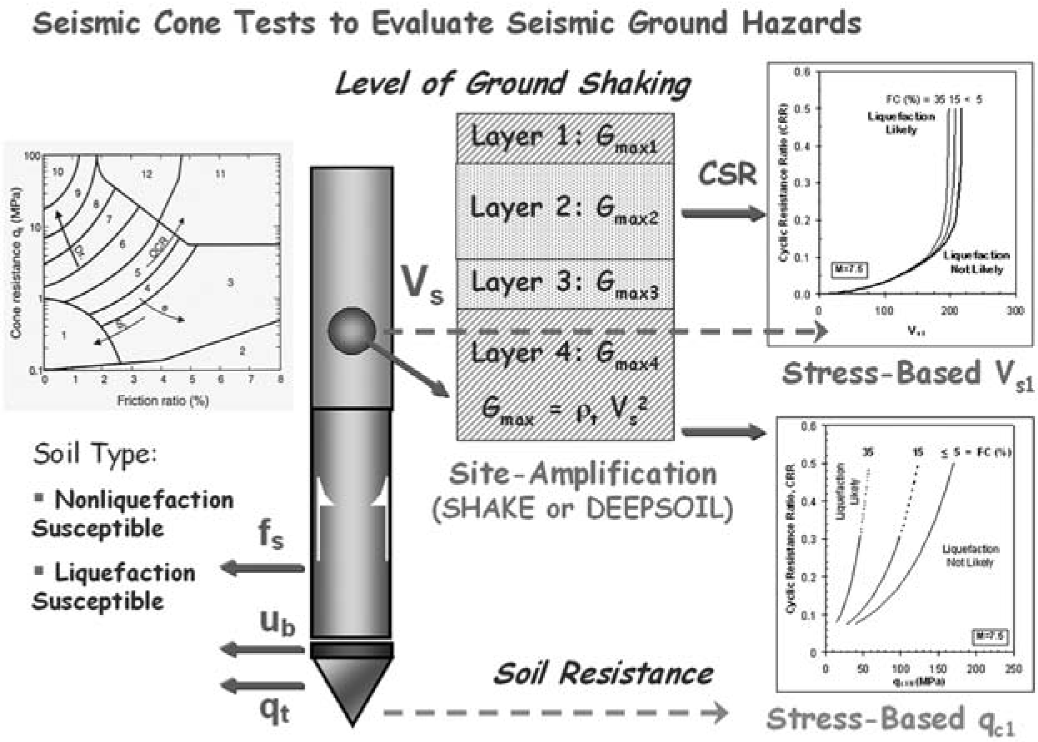


Figure 1: Cone Penetrometer for evaluating site-specific soil liquefaction concerns.

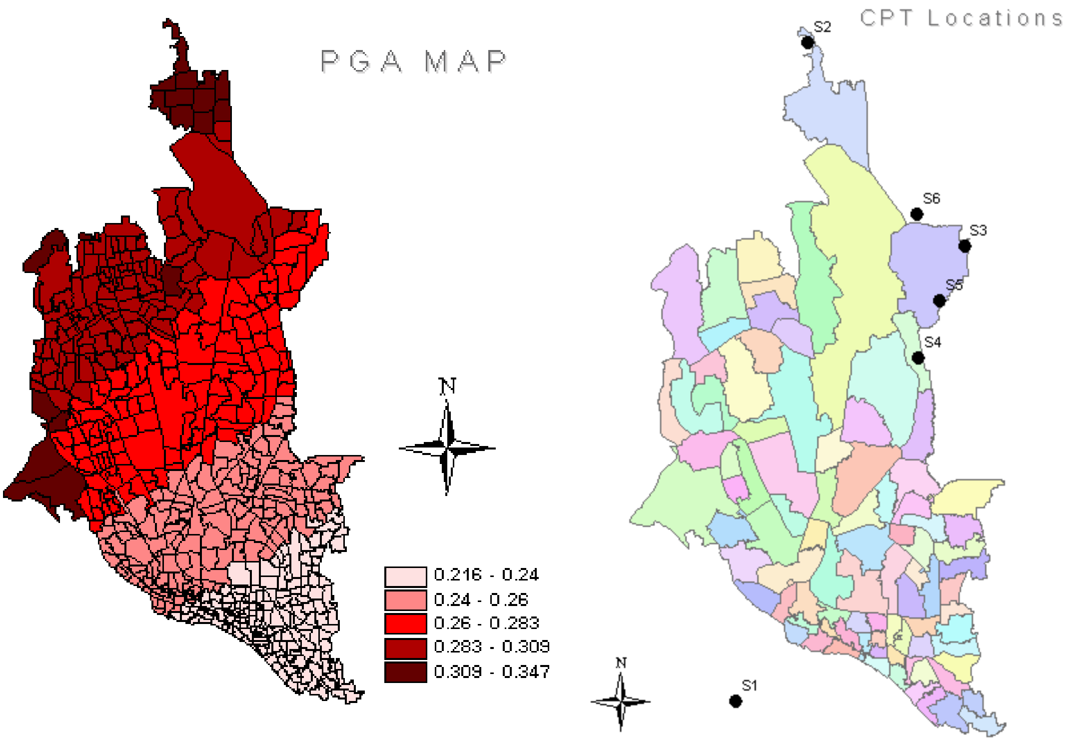


Figure 2: PGA map for Dhaka (CDMP, 2009) and locations of CPT (right).

Figure 3: Cone Penetration Test demonstration at BUET campus.

To evaluate liquefaction resistance of soils, two variables are required for evaluation. The level of ground shaking from seismic loading is expressed in terms of Critical Stress Ratio (CSR) and the capacity of the soil to resist liquefaction, expressed in terms of cyclic resistance ratio (CRR). The CSR can be estimated using seismic ground hazard maps or alternatively evaluated more properly using site specific Peak Ground Acceleration (PGA) data within relevant codes. Using conventional simplified procedure, the CSR can be calculated by using peak ground surface acceleration (a_{max}) at the site (Seed and Idriss, 1971). This simplified approach can be summarized as follows:

$$CSR=\tau_{ave}/ \sigma'_v =0.65(a_{max}/g)\ r_d\ (\sigma_v/\sigma'_v) \quad (1)$$

where τ_{ave} is the average equivalent uniform shear stress generated by the earthquake (assumed to be 65% of the maximum induced stress), a_{max} is the peak horizontal ground surface acceleration, g is the gravitational acceleration (g = 9.8 m/s^2), σ_v is total vertical overburden stress, σ'_v is effective overburden stress at the same depth and r_d is shear stress reduction coefficient which is dependent on depth. By using recommendation of the National Center for Earthquake Engineering workshop on soil liquefaction (Youd et al., 2001), r_d can be obtained with depth z (meters) as follows:

$$r_d = \begin{cases} (130 - z)/131 & \text{if } z<9.15 \text{ m} \\ (44 - z)/37 & \text{if } 9.15\text{m} \le z \le 23\text{m} \\ (93 - z)/125 & \text{if } 23\text{m} \le z \le 30\text{m} \\ 0.5 & \text{if } z > 30\text{m} \end{cases} \quad (2)$$

The value of a_{max} is taken from the CDMP (2009) proposed PGA map of Dhaka. Robertson (1990) stressed that the CPT-based charts were predictive of Soil Behaviour Type (SBT), since the cone responds to the in-situ mechanical behavior of the soil and not directly to soil classification criteria based on grain-size distribution and soil plasticity e.g. Unified Soil Classification System (USCS). Grainsize distribution and Atterberg Limits are measured on disturbed soil samples. Fortunately, soil classification criteria based on grain-size distribution and plasticity often relate reasonably well to in-situ soil behaviour and hence, there is often good agreement between USCS-based classification and CPT-based SBT (e.g. Molle, 2005).

The modified CPT soil behavior type index, I_{SBT} is defined by Robertson and Wride (1998) using only CPT Q and F data because porewater pressure are often near hydrostatic for loose firm clean sands (Figure 4). The normalized SBT index I_c, is given by

$$I_c = \sqrt{[(3.47-\log Q)^2 + (1.22+\log F)^2]} \quad (3a)$$

Where,

$$Q = [(q_c - \sigma_v)/P_a]\ (P_a/\sigma'_v)^n \quad (3b)$$

$$F = [f_s/(q_c - \sigma_v)]\ 100 \quad (3c)$$

The value of Q is adjusted to q_{c1n} for stress normalization. This requires iteration process as below:

$q_{c1n} = (q_c/P_{a2})\,(P_a/\sigma'_v)^{0.5}$ if $I_c < 2.6$ (4a)
$q_{c1n} = Q$ if $I_c > 2.6$ (4b)

Again, I_c is calculated by

$I_c = \sqrt{[(3.47 - \log q_{c1n})^2 + (1.22 + \log F)^2]}$
(5)

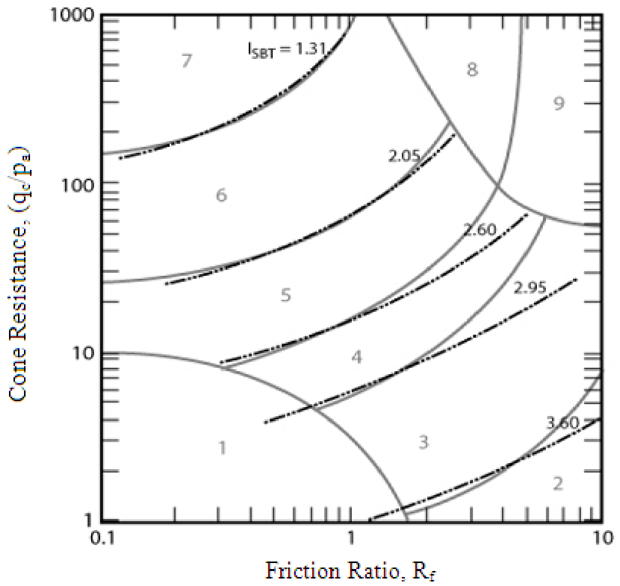
Now, if $I_c > 2.6$ then q_{c1n} has to be recalculated,

$q_{c1n} = (q_c/P_{a2})\,(P_a/\sigma'_v)^{0.75}$ if $I_c > 2.6$ (6)

Finally,

$I_c = \sqrt{[(3.47 - \log q_{c1n})^2 + (1.22 + \log F)^2]}$
(7)

Where, q_c is measured cone tip resistance, f_s is measured cone sleeve resistance, P_a is a reference stress of 100 kPa (or 1 atm), P_{a2} is reference stress in MPa and n = an exponent that depends on soil behaviour type. The values of q_c, f_s, P_a, σ_v, and σ'_v are all in the same units. The value of n ranges from 0.5 for clean sands to 1.0 for clays (Olsen et al., 1997), and can be approximated through an iterative approach. The profiles of non-normalized SBT index (I_{SBT}) and the SBT descriptions, based on the chart shown in Figure 4.



SBT zone Robertson (1990)	*Soil Behavior Type*
1	*Sensitive fine-grained*
2	*Clay - organic soil*
3	*Clays: clay to silty clay*
4	*Silt mixtures: clayey silt & silty clay*
5	*Sand mixtures: silty sand to sandy silt*
6	*Sands: clean sands to silty sands*
7	*Dense sand to gravelly sand*
8	*Stiff sand to clayey sand**
9	*Stiff fine-grained**

** Overconsolidated or cemented*

Figure 4: Updated non-normalized SBT chart based on dimensionless Cone Resistance, (q_c/p_a) and Friction Ratio, $R_f = (f_s/q_c\text{-}\sigma_v)$ 100%, showing contours of I_{SBT}.

K_c is a correction factor for the apparent fines content and is empirically calculated from a modified CPT soil classification index, I_c. Specifically K_c is evaluated from:

$K_c = 1.0$ if $I_c \leq 1.64$ (8a)
$K_c = -0.403I_c^4+5.581I_c^3-21.63I_c^2+33.75I_c-17.88$ if $I_c > 1.64$
(8b)

Normalized Cone Tip Resistance,

$(q_{c1n})_{cs} = K_c(P_a/\sigma'_v)^n(q_c/P_a)$.
(9)

Corrected Critical Stress Ratio Resisting Liquefaction (CSR_L) was calculated using different correction factor.

CSR_L = CSR k_m k_α k_s
(10)

Where,

k_m = Correction factor for earthquake magnitude other than 7.5 (1.00 for Mw 7.5)
k_α = Correction factor for initial driving static shear (1.00 for no initial static shear)
k_s = Correction factor for stress level larger than 96 kPa

The level of ground motion CSR and the adjacent tip resistance (q_{c1n}) are compared with the CRR to determine whether liquefaction will or will not occur. CRR is calculated by the following equation for an earthquake moment magnitude of 7.5 (Youd et al., 2001; Robertson and Wride, 1998):

$CRR = [0.833\,(q_{c1n})_{cs}/1000] + 0.05$ if $(q_{c1n})_{cs} < 50$
(11a)
$CRR = 93\,[(q_{c1n})_{cs}/1000]^3 + 0.08$ if $50 \leq (q_{c1n})_{cs} \leq 160$
(11b)

The Cyclic Resistance Ratio (CRR) is the threshold for liquefaction and used to compare the available soil resistance with level of ground shaking represented by the CSR. Therefore if the CSR value is higher than CRR, the soil has a high likelihood of liquefaction. If the CSR falls beneath the CRR, the likelihood of liquefaction is small. The CRR can be expressed using conventional approaches that give a binary decision (liquefaction or no liquefaction), or alternatively, in terms of probabilistic curves of increasing risk of liquefaction.

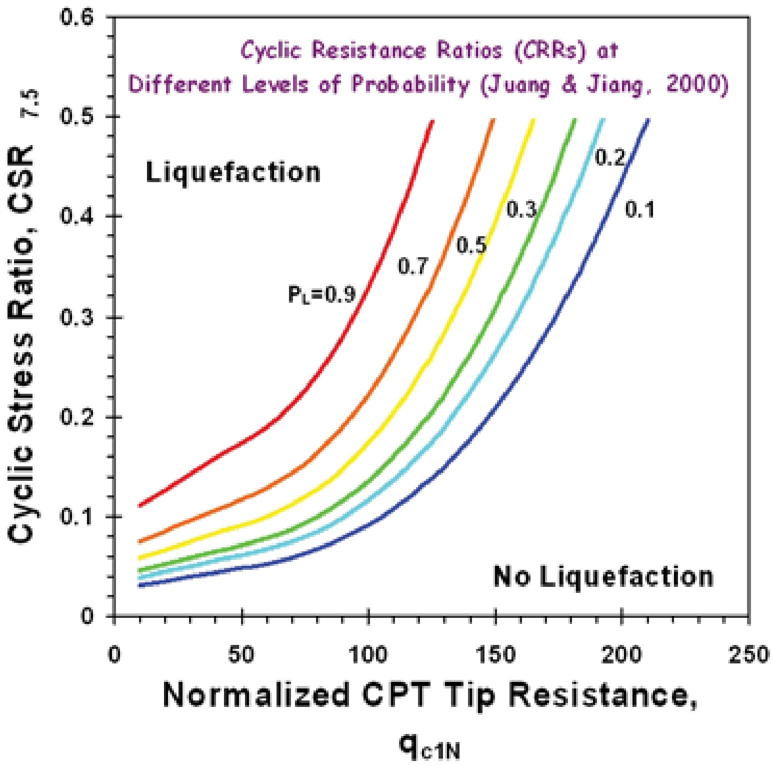


Figure 5: Probabilistic cyclic resistance ratios (CRRs) for clean sands based on (upper) normalized cone tip resistance and (lower) normalized shear wave velocity (after Juang and Jiang 2000).

A calculated factor of safety (F_S) can be defined as F_S= CRR/CSR_L for a particular earthquake magnitude and a set of data. In more recent evaluations, CRR curves of different probabilities of occurrence have been developed from mapping functions (Juang and Jiang, 2000) to relate the safety factors F_S to the liquefaction probability P_L. Based on database of 225 CPT-based cases reported by Juang and Jiang (2000) for q_{c1n} probability curves as shown in Figure 5 may be developed using the following equation:

P_L=1/ [1+ $(FS/1.0)^{3.34}$]
(12)

3. RESULTS

CPT tests were carried out up to a maximum depth of 23 m. The water table at ground level has been considered. The peak horizontal ground acceleration value for the site has been taken corresponding to zone factor according to CDMP. a_{max}/g for Dhaka has been taken between 0.21 and 0.35, the extent to which liquefaction is expected for 7.5 magnitude earthquake. The saturated unit weight of the soil taken 18 kN/m^3 and unit weight of water is taken 9.8 kN/m^3.

Figure 6 presents the measured CPT parameters in terms of cone resistance (q_c), friction ratio (R_f) and pore pressure (u_2), as well as the profiles of non-normalized SBT index (I_{SBT} or I_c) and the SBT descriptions, based on the chart shown in Figure 4. The SBT zones are colour coded to aid in visual representation (Geologismiki, CPeT-IT, 2011). The pore pressure sensor appears to have lost saturation when passing through the thin dense sand layer up to 12 m, but regained saturation again at a depth of about 19 m.

Figure 7 presents the normalized CPT parameters as defined and updated by by Robertson (2009), as well as the profiles of normalized SBT index (I_c) and the SBT_n descriptions, base on Robertson SBT zone 1990. Figures 6 and 7 show that there is little difference between the soil behavior type interpretation for this example profile.

Figure 8 shows the CPT data plotted on both the updated non-normalized SBT chart and the normalized SBT_n (Robertson, 1990) chart. Each layer is represented by different colour data points to aid identification. Figure 8 shows that the CPT data are more closely clustered on the normalized charts, as expected, but that both charts provide similar interpretation of soil behaviour type. Note that the “normally consolidation” region suggested by

Robertson (1990) can only be shown on the normalized SBT chart, since a similar region is only valid on the non-normalized chart when the effective overburden stress is close to 1 atmosphere.

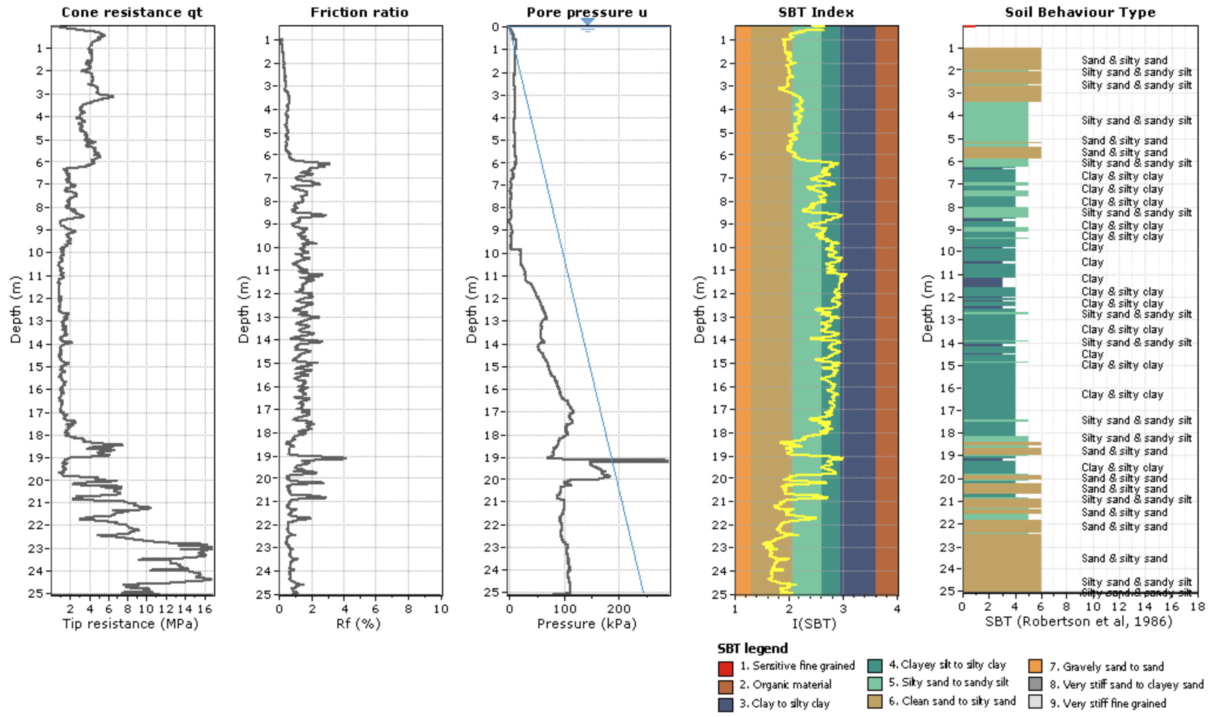


Figure 6: Example CPT Profile to illustrate non-normalized SBT and SBT Index, I_{SBT}.

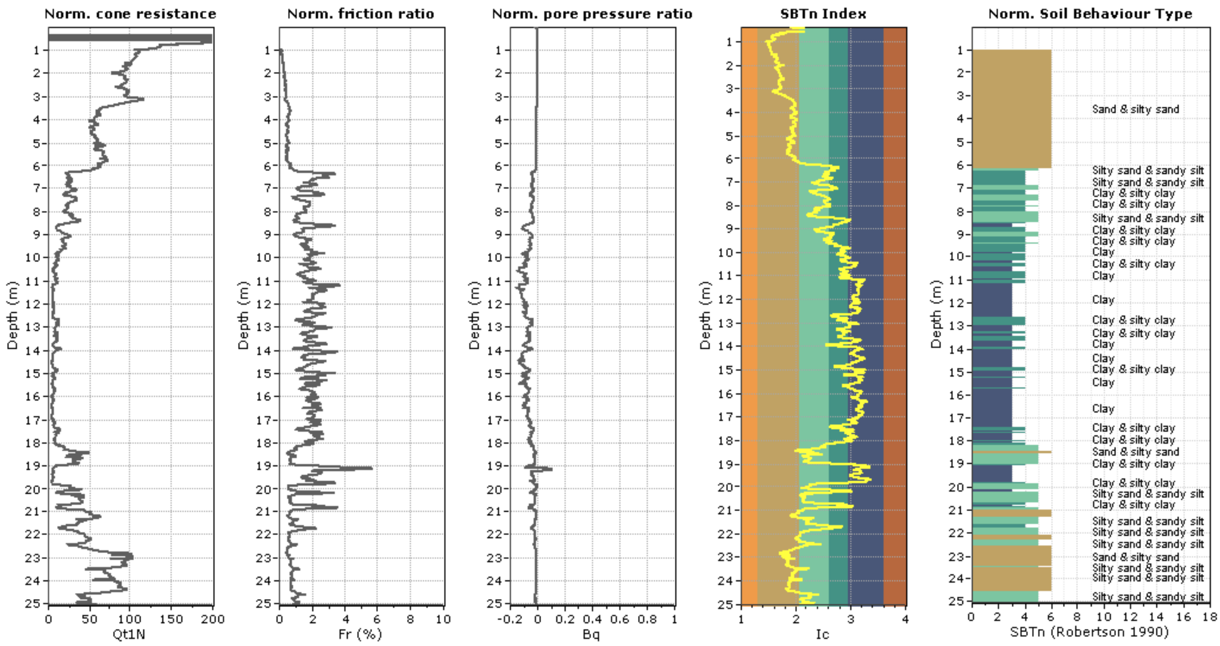


Figure 7: Example CPT Profile to illustrate normalized SBT and SBT Index, I_c.

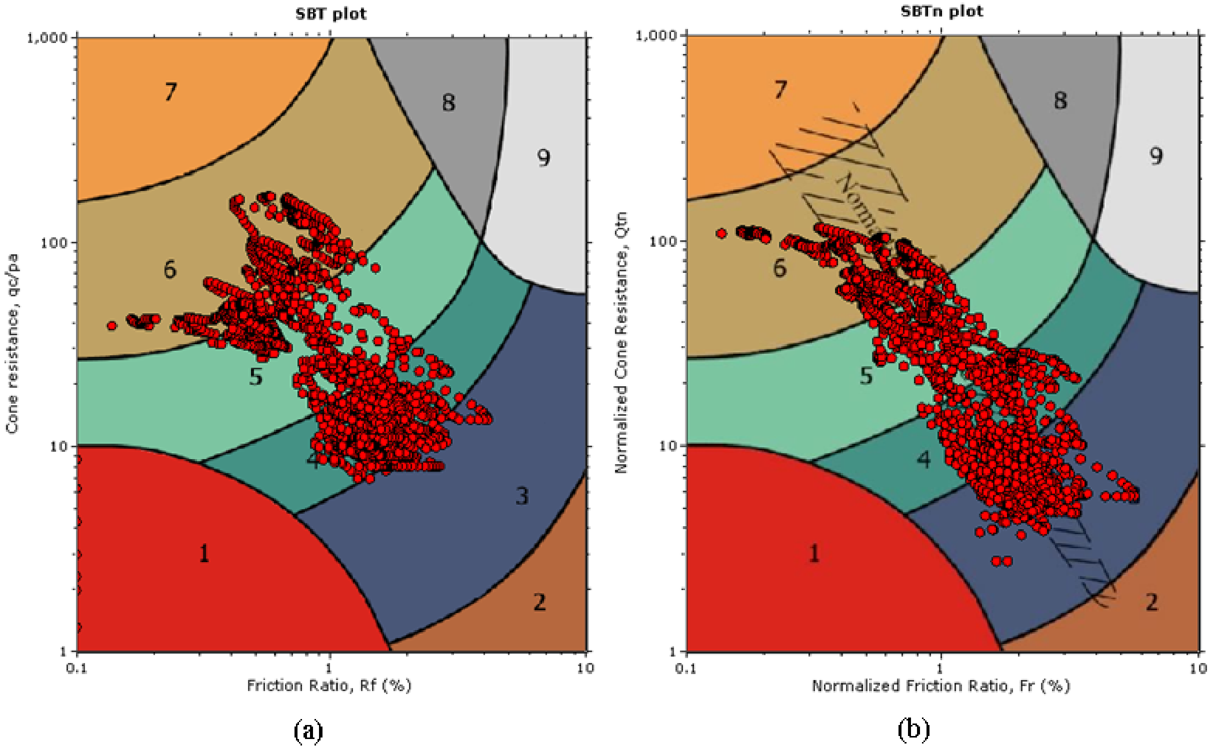


Figure 8: Comparison between (a) updated non-normalized SBT and (b) normalized SBT_n (Robertson, 1990) for the CPT profile shown in Figure 6

In order to identify the soil constitution, Grain size analysis have been performed for collected soil samples where CPT test was performed. Figure 9 shows result of sieve analysis performed in the laboratory for site S1. From the grain size analysis percentage finer than 200 sieve is 11.9, C_u = 3.43 and C_z = 1.34 was found. The finer porton was found as silt. According to USCS classification the soil is poorly graded silty sand (SP-SM). Liquefaction is a problem associated with this kind of soil.

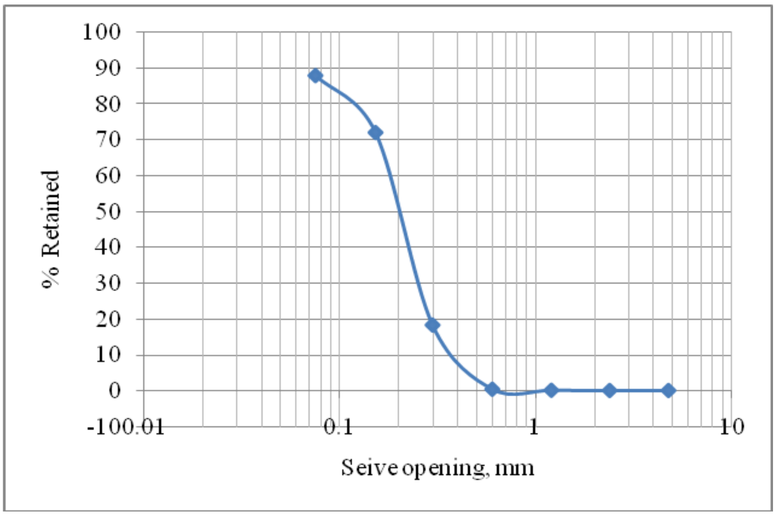


Figure 9: Seieve analysis for a site.

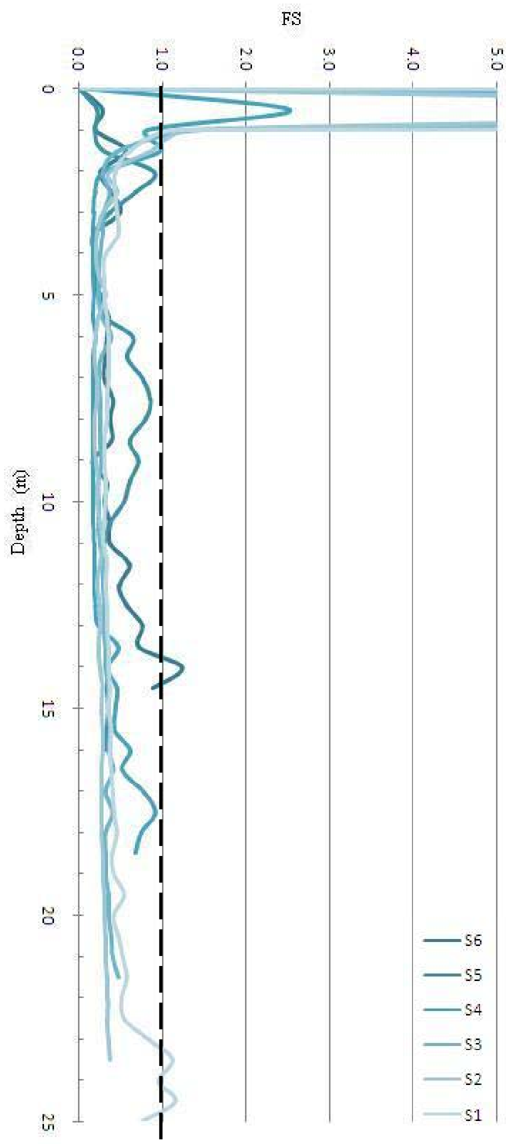


Figure 10: Factor of Safety for six CPT locations considering maximum PGA value for relevant zone.

Normally, the top most layers of soil become hard because of repeated loading and unloading due to sun and pedestrians. It become compacted and dense (in case of sand) or over consolidated (in case of clay). From figure 10 it can be observed that for 4 of our sites at top 1 m layer factor of safety against liquefaction is higher than 1, which indicates non liquefaction zone. But where the foundations are made below 1m from surface the factor of safety is less than 1. So the foundation zone is liquefiable. The probability of liquefaction can be found from Figure 5 too.

4. CONCLUSIONS

For extending a city like Dhaka, there is no alternative of land reclamation. Land reclaiming is easy in Dhaka because of presence of 5 rivers surrounding the city. The soil properties are almost same for all river bed sands. This study presents liquefaction potential for a magnitude 7.5 (M_w) earthquake. From the study, it can be found that for the first 1-2 meters, factor of safety against liquefaction is greater than 1, which represents non liquefiable zone, but below that, liquefaction probability is quite high. For ordinary residential structures, foundation level lies within 1- 2 m depth, so any kind of foundation can be affected by liquefaction during an earthquake with PGA value of 0.21g to 0.36g (M_w=7.5). For larger earthquakes like 1950 Shillong earthquake (M_w = 8.6) or 1897 Assam earthquake (M_w= 8.1) the liquefaction probability will be much higher. Engineers engaged in construction sector in Dhaka must be aware about liquefaction as well as other earthquake related hazards and their possible remedial measures.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

CHARACTERIZATION AND STATISTICAL ANALYSIS OF GEOTECHNICAL PARAMETERS OF STABILIZED ORGANIC SOIL AT SOUTH WESTERN REGION OF BANGLADESH

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ABSTRACT

This study illustrates the physical and mechanical characteristics of stabilized soil prepared in the laboratory by mixing of cement, lime and bentonite at varying content of 5, 10, 15, 20 and 25% of the dry mass of organic soil. Moreover, the effect of admixtures content on the unconfined compressive strength (q_u), change of liquid limit (w_L) in relation to the variation of mixing water content, variation of laboratory compaction parameters with admixture and organic content as well as develop a linear regression model using SPSS of stabilized soils were also highlighted by the author. To these attempts, organic soil samples from four selected locations at Khulna region were collected. In the laboratory, required ASTM (2004) methods were followed to measure and monitor the relevant parameters of the stabilized soils. Results reveal that q_u of stabilized soil increases in relation to the increasing of admixtures content as well as the curing days. However, the w_L of all the stabilized soils was found to decrease with increasing the admixture content and curing days. In contrast, the maximum dry density increases, while, the optimum moisture content decreases with the increasing of admixture content. Here, it is interesting to note that to depict the reliability and accuracy of the developed model, the predicted values of q_u from regression model were compared with the laboratory measured values of q_u. Based on the regression analysis, it can be concluded that the R^2 was ranging from 0.726 to 0.965 for bentonite, 0.536 to 0.930 for lime and 0.909 to 0.984 for cement stabilized soils.

Keywords: *Chemical admixtures, organic soil, stabilization, geotechnical parameters, regression model.*

1. INTRODUCTION

In the south and south-western Asian region like Khulna, due to the increasing trend of urbanization and industrialization, marginal sites need to be utilized for the development of infrastructure facilities. Moreover, the subsoil in this region consists of recent alluvial deposits and organic and organic composition and often creates problem to geotechnical engineers in designing economic foundations for strength and high compressibility (Alamgir et al. 2001). The terms organic soils, used for describing soils with an organic content, were once synonymous but organic soils is presently used for superficial deposits or soils that contain organic matter (Haut 2004 and Rafizul et al. 2006). Organic soils are believed to be geotechnically problematic due to their very high compressibility, very low shear strength and difficult accessibility. Organic soil represents the extreme form of soft soil deposits and it is subjected to instability, slip failure as well as massive primary and long term settlement, constructing of infra-structure on this layer (Alamgir 1996 and Porbaha 2000). On the other hand, the subsoil of this region consists of fine-grained soils with a considerable part of decomposed and semi-decomposed organic matter (Rafizul et al. 2010; Alamgir et al. 2006). In this region, the soft soil deposit extends up to a considerable depth, as a result the recent alluvial deposits with organic composition creates problem to geotechnical engineers in designing economic foundations to construct the required infrastructure (Mahamud et al. 2007 and Rafizul et al. 2007). Today the deep mixing method is accepted world-wide as a soil improvement method which is performed to improve the strength, deformation properties and permeability of organic peat soil (Costas 2008). It is based on mixing binders, such as cement, lime, bentonite, and other additives with the natural soil. On the other hand, chemical admixtures used for the stabilization of organic soil always chances the physical and engineering properties of soil, volume stability and strength characteristics (Bujang et al. 2005).

Chemical admixture always involves treatment of the soil with some kind of chemical compound, which when added to the soil would result in chemical reaction. The chemical reaction modifies or enhances the physical and engineering properties of a soil, such as, volume stability and strength. Chemical stabilized like cement, lime and bentonite has two folds effect on the soil characteristics of fluctuation, the clay particles are electrically attracted and aggregated with each other. This results in an increase in the effective size of the clay size aggregation. Such aggregation converts clay into the mechanical equivalent of fine silt. Also, a strong chemical bonding force develops between the individual particles in such aggregation. The chemical bonding depends upon the type of stabilizer employed (Bujang et al. 2005). The physical and mechanical properties of stabilized soils depend on several factors, mainly the properties of base material and the environmental aspects. The strength development of admixture stabilized soil depends on many factors such as type and properties of soil, quantity and type of admixture, soil moisture content, mixing and compaction method, condition and curing time, temperature, soil minerals and used admixture.

The main focus of this study formulated as, (i) investigate the effect of chemical admixture and admixtures content on the q_u ; (ii) investigate the effect of curing period on q_u; (iii) change of w_L in relation to the variation of mixing water content, admixtures content and organic content; (iv) the effect of organic content on q_u ; (v) change of compaction properties at varying admixtures content and organic content as well as (vi) develop a new model of linear regression analysis through SPSS, based on measured q_u of stabilized soil in the laboratory.

2. MATERILS AND METHODOLOGY

The preparation stabilized soil through chemical admixture (cement, lime and bentonite) in the laboratory within a short elapsed time is very difficult, therefore, a mechanical study of cement, lime and bentonite stabilized soil was occasionally carried out in the laboratory to unserstand the mechanical properties of stabilized soils (Yamadera 1999). Organic soil samples were collected for laboratory routine test at a depth of 10, 7.5, 5.5 and 12 ft, from the existing ground surface from the four selected location in Khulna region, namely, Teligati, Rangpur, Sonadanga as well as Khulna University campus, respectively. The physical and index properties of the collected soil sample were investigated in the laboratory through the standards methods (ASTM 2004) and presented in the Table 1.

Table 1 Physical and index properties of organic soils used in this study

Properties	Teligati	Rangpur	Sonadanga	KU campus
Water content, w (%)	96	87	65	126
Liquid limit, w_L (%)	145	122	95	170
Plastics limit, w_P (%)	65	57	45	80
Plasticity index, I_p (%)	80	65	50	90
Specific gravity, G_s	1.23	1.46	1.71	1.12
Organic content, OC (%)	66	53	32	71

Cement particle is heterogeneous substance, containing tri-calcium silicate (C_3S), di-calcium silicate (C_2S), tri-calcium aluminate (C_3A), and solid solution described as tetra-calcium alumino-ferrite (C_4AF). Di-calcium silicate (C_2S) is responsible for the progressive strength of cement. In this study for preparing the stabilized soil in the laboratory, the amounts of cement, lime and bentonite as percentage of dry soil mass, were ranging of 5-25%. The basic ingredients of cement used in this study obtained from routine laboratory test are given in Table 2. Moreover, the basic ingredients of used lime and bentonite used in this study are also given in Table 3.

Table 2 Physical properties of cement used in this study

Physical properties	Values
Normal consistency (%)	22.40
Initial setting time	2 hours 10 minutes
Final setting time	4 hours 50 minutes
Fineness (%)	0.60

Table 3 Basic ingredients of lime and bentonite used in this study

Basic ingredients of lime	Composition (%)	Basic ingredients of bentonite	Composition (%)
Calcium oxide	50	Magnasium	25
Magnesium oxide	35	Aluminium	35
Aluminum oxide	3	Silica	67
Silica	22	Iron	3

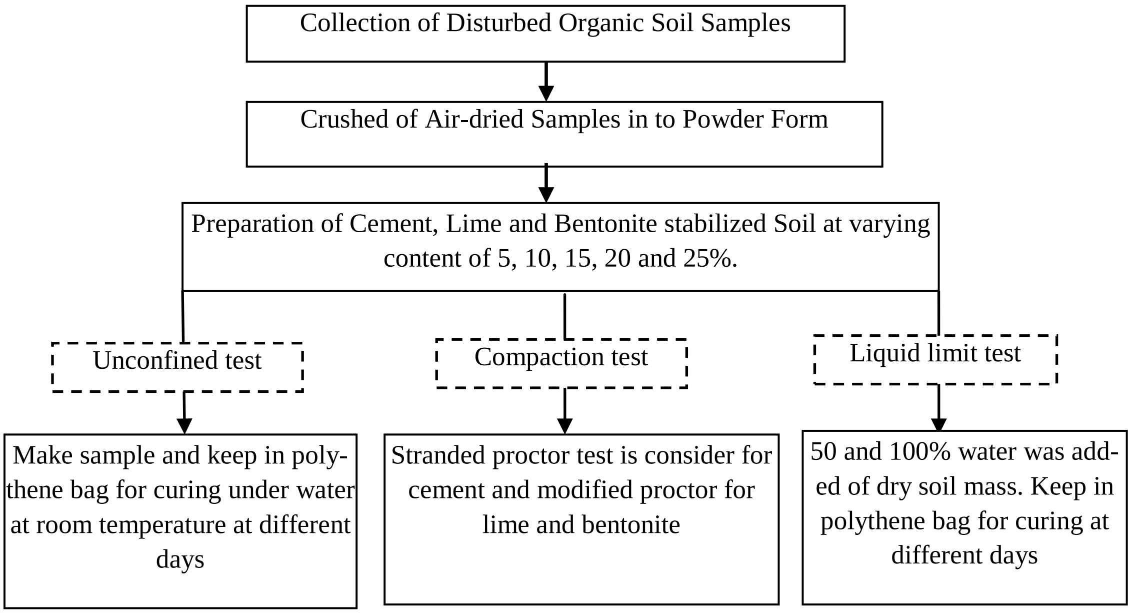


Figure 1 Flow chart of laboratory investigation

A sequence of test is performing in order to observe the effect of cement, lime and bentonite on the physical and mechanical properties of stabilized organic soil. The collected undisturbed samples were first brought to the laboratory and spreader it over the floor to get air- dry soil samples. After drying, then soil chucks were broken and grinded by using wooden hammer as fine as possible without applying unnecessary pressure. The soil powder was passed through # 40 standard sieves to avoid the chuck. Air dry soil powder free from any chuck of foreign materials was used as the main ingredients to prepare stabilized soil-cement/lime/bentonite specimen**.** The water content of air-dry soil sample was measured at a range of 3.75%-5.5%. The mixing was done properly to get a uniform mixture of soil cement, lime and bentonite and to prepare a uniform paste free from lumps and other foreign particles. The soil paste was then poured in to the cylindrical plastic mould by the help of fingers so that no air voids entrapped into the soil sample. After six hour the specimen removed taken out from the cylindrical mould and it was tightly wrapped in polythene bags to prevent the loss of moisture from the stabilized soil due to evaporation. After 24 hours, the wrapped specimens were placed under water in the room temperature until testing at the designated rest period of 1, 3, 7, 14 and 28 days. Here, it can be noted that to investigate the effect of chemical admixture through the liquid limit test, the samples were prepared in the laboratory by mixing of water content at 100 and 50%. Moreover, for investigate the laboratory compaction properties of stabilized soil, samples were compacted in accordance to the standard proctor test for cement and bentonite, modified proctor test for lime admixture. The methodology followed for the laboratory investigations as presented in Figure 1.

3. RESULTS AND DISCUSSIONS

The effect of chemical admixture in terms of admixture content and curing period of compressive strength, liquid limit as well as the laboratory compaction properties on the stabilized soil prepared in the laboratory were analyzed and hence discussed in following.

3.1 Effect of Curing Period on the Unconfined Compressive Strength of Organic Soil

The variation of unconfined compressive strength in relation to the increasing of elapsed period from sample preparation at the varying percentages of cement, lime and bentonite content of 5, 10, 15, 20 and 25%, is evident in Figures 2~5. Based on this figures, it was observed that the unconfined compressive strength increases with the increasing of elapsed period and it was continued. Moreover, studies are carried out to examine the effect of chemical admixtures in terms of admixture content and curing period as well as the influence of organic content on q_u of stabilized soil. The effect of cement content varying from 5 to 25%, on q_u of cement stabilized soils at varying organic content from 32 to 66% shown in Figure 2 and Figure 3, respectively. Figures 2 and 3 reveals that the values of q_u increases in relation to the increasing of elapsed period of cement stabilized soil for all the percentages of organic content. However, result reveals that stabilized soil at lower organic content showing more q_u than that of stabilized soil at higher organic content (Figures 2 and 3).

It of interest to note that for case of cement stabilized clay, Bergado (1996) found that pozzolanic reaction can continue for months or even years after mixing, resulting in the increase in strength of cement stabilized clay with the increase in curing time. Based on Figures 2~5, it can be noted that unconfined compressive strength increases with the increasing of elapsed period and the findings of the present study are in well agreed with the postulation given by Bergado (1996).

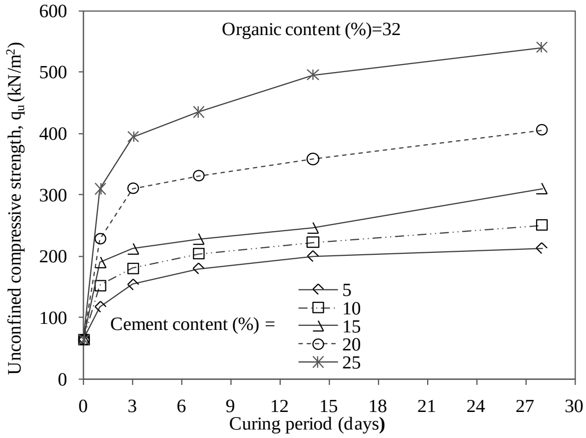


Figure 2: Effect of curing period on q_u of cement stabilized soil (*OC=32%)*

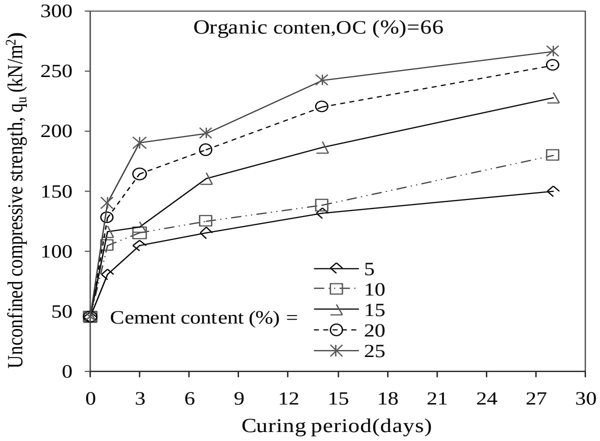


Figure 3: Effect of curing period on q_u of cement stabilized soil (*OC=66%)*

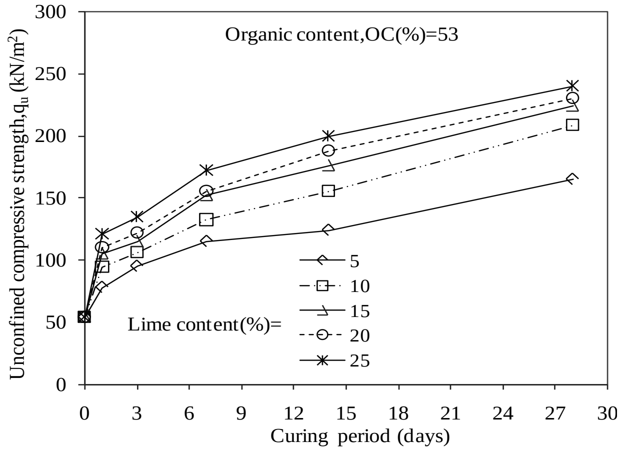


Figure 4: Effect of curing period on the unconfined compressive strength of organic soil (OC= 53%)

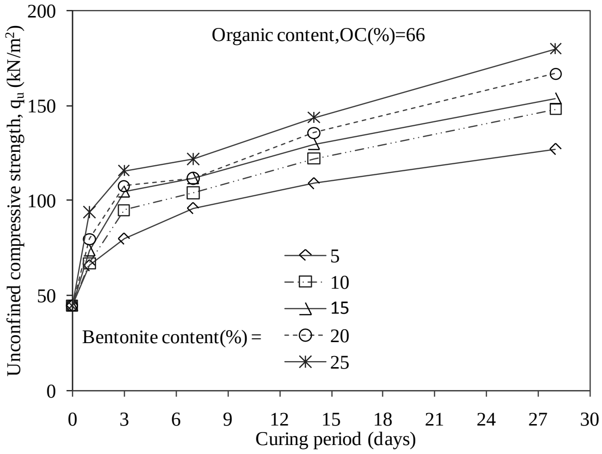


Figure 5: Effect of curing period on the unconfined compressive strength of organic soil (OC= 53%)

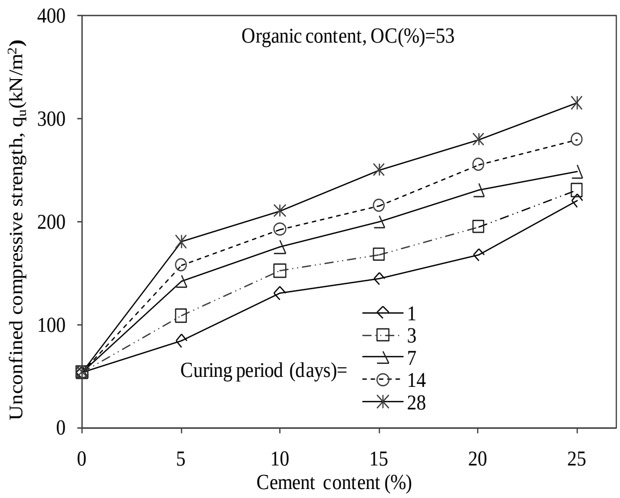


Figure 6: Effect of cement content on unconfined compressive strength of organic soil (OC= 53%)

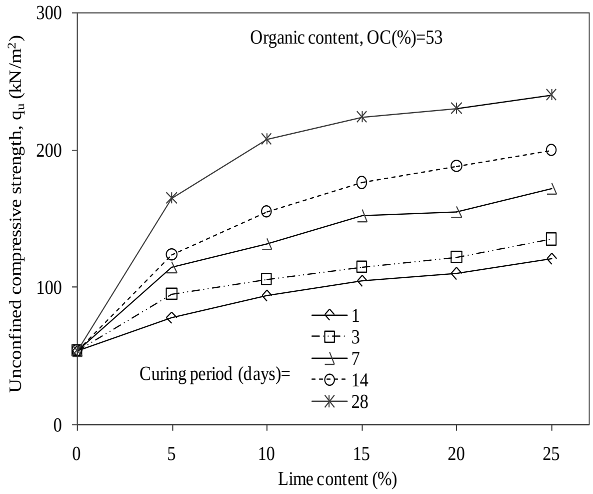


Figure 7: Effect of lime content on unconfined compressive strength of organic soil (OC= 71%)

3.2 Effect of Organic Content on the Unconfined Compressive Strength of Soil at Different Curing Period

The variation of unconfined compressive strength mainly depend on the amount of organic content are present in the soil. High organic content stabilized soil show low strength than less organic content stabilized soil. The value of unconfined compressive strength increases with increasing admixture content and curing period are shown in Figure 6~ 9.

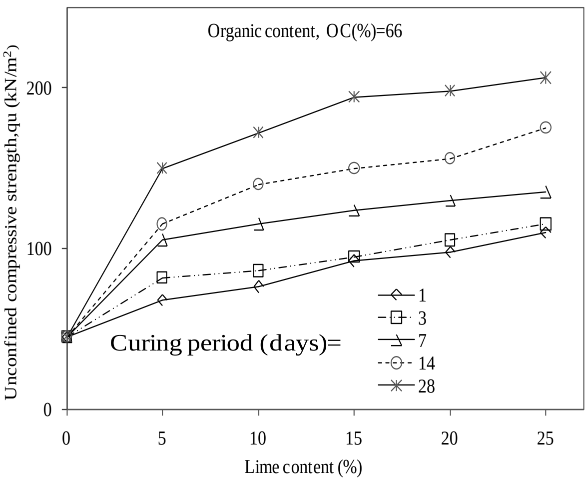


Figure 8: Effect of lime content on unconfined compressive strength of organic soil (OC= 66%)

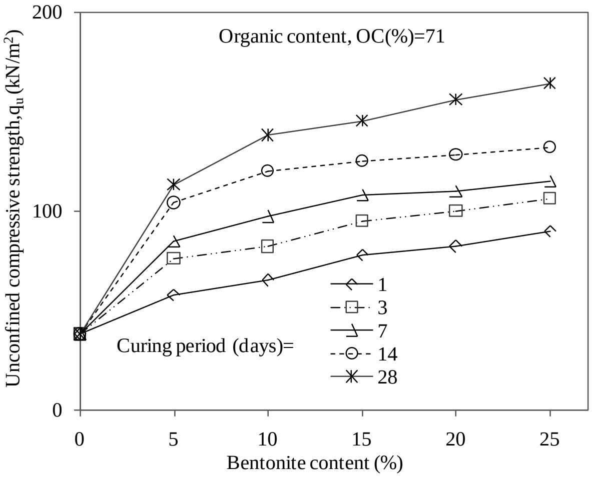


Figure 9: Effect of bentonite content on unconfined compressive strength of organic soil (OC= 71%)

3.3 Effect of Admixture Content and Curing Period on the Liquid Limit of Organic Soils

The addition of admixture decreases the soils liquid limit. The results also show that the decrease in liquid is more pronounced for soils with lower organic content and higher amount of mixing water. The chemical reaction between additives such cement and soils are also known as time dependent. Figure 10~15 shows plot of liquid limit with curing time in days. As shown, the liquid limit of the soil-cement mixed decrease with increased in the curing duration. The variation of liquid limit with in relation to the increasing of elapsed period from sample preparation at the varying percentages of cement content of 5, 10, 15, 20 and 25%, is evident in Figure 10~ 15.Based on Figure 10~ 15 it was observed that the value of liquid limit decreased with the increasing of elapsed period up to the end of this trial. Moreover, the value of liquid limit also decreases with 100% water than 50% water. However, it was not possible to conduct any more liquid limit test after 7-days curing as the soil samples have become too hard. Figure 10~ 15 also shows the effect of mixing water on the liquid limit of the soil-cement/lime/ bentonite mixed. Mixing water of 50 and 100% were examined in this study. As shown, for a particular soil-cement/lime/ bentonite mixed, there is a bigger reduction in liquid limit when more water is made available for the chemical reaction to take place.

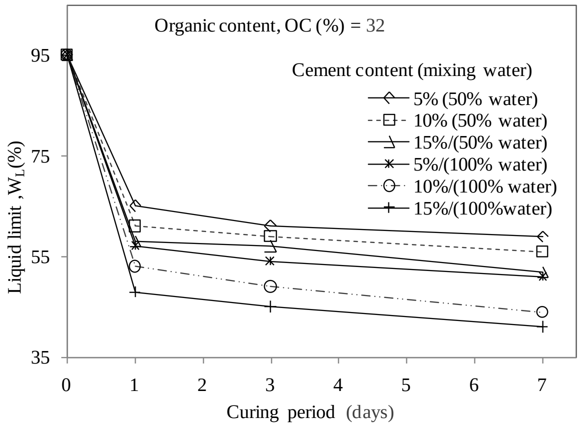


Figure 10: Effect of cement content at varying mixing water and curing period on liquid limit of stabilized soil

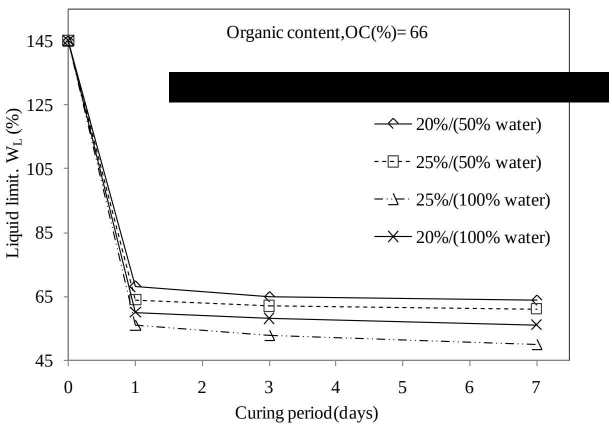


Figure 11: Effect of cement content at varying mixing water and curing period on liquid limit of stabilized soil

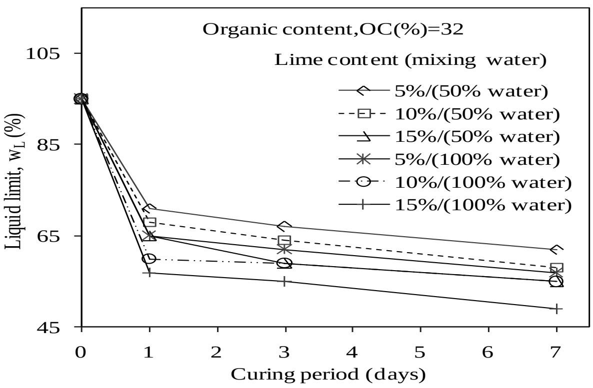


Figure 12: Effect of lime content at varying mixing water and curing period on liquid limit of stabilized soil

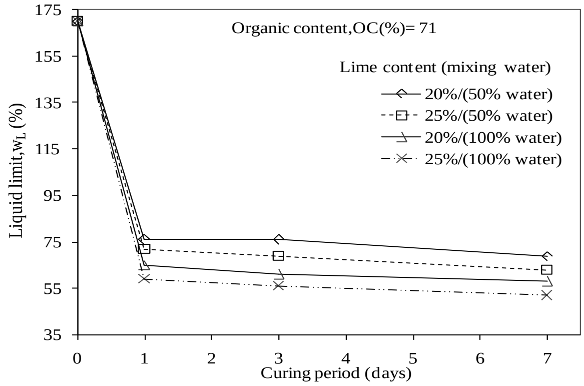


Figure 13: Effect of bentonite content at varying mixing water and curing period on liquid limit of stabilized soil

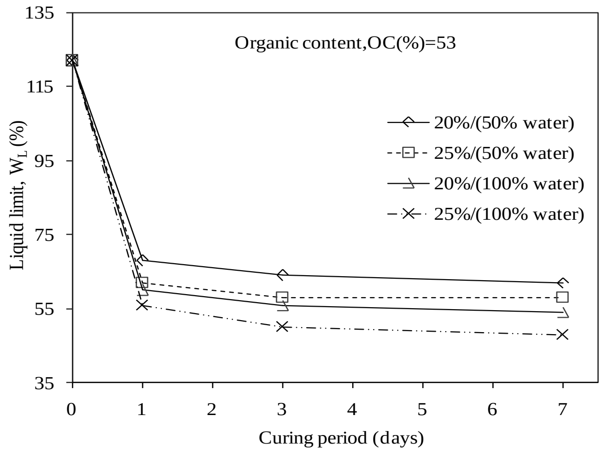


Figure 14: Effect of bentonite content at varying mixing water and curing period on liquid limit of stabilized soil

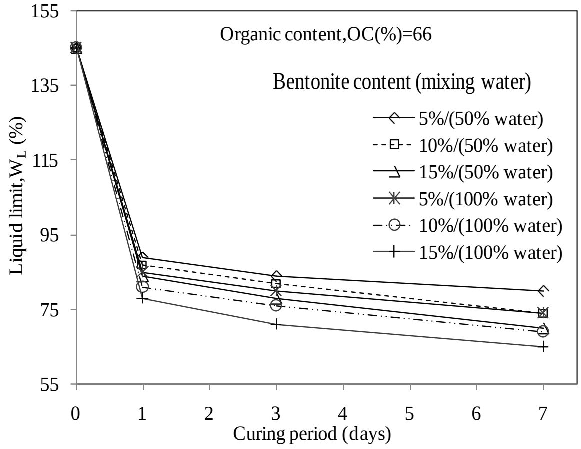


Figure 15: Effect of bentonite content at varying mixing water and curing period on liquid limit of stabilized soil

3.4 Effect of Compaction on Maximum Dry Density of Organic Soils

A series of test is conducted to study the effect of cement on the compaction characteristics of peat soil. These samples were compacted in accordance to the standard proctor test whereby the samples were compacted in three layers with a 5.5 Ib hammer that delivers 25 blows to each layer for cement, bentonite admixture and samples were compacted in five layers with a 10.0 lb hammer that delivers 25 blows to each layer for lime admixture. As shown, the addition of admixture has an influence in reducing the optimum water content and increasing the maximum dry unit weight of the organic soil. The variation of dry unit weight and optimum moisture content with the addition of varying percentage admixture 5%,10%, 15%, 20% and 25% are evident in Figure 16~ 19. Based on figures, it was observed that the values of dry unit weight of admixture mixing organic soil increases, while, the optimum moisture content decreases with the increasing of admixture content. However, the value of dry unit weight decreases and optimum moisture content increases with the increasing of organic content.

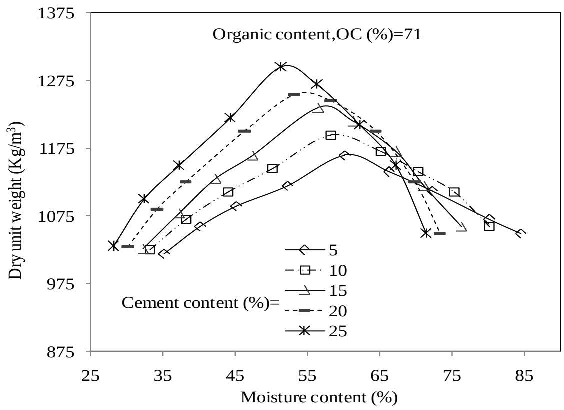


Figure 16 Effect of cement content on dry density of stabilized soil at varying organic con-

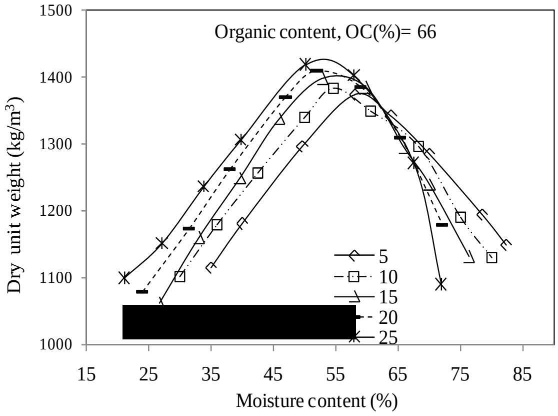


Figure 17: Effect of cement content on dry density of stabilized soil at varying organic

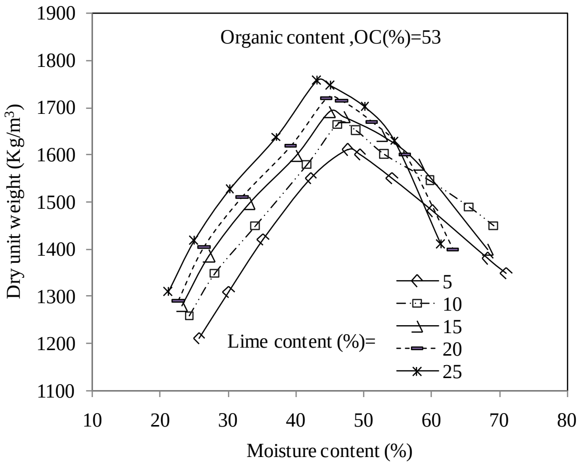


Figure 18: Effect of cement content on dry density of stabilized soil at varying organic content.

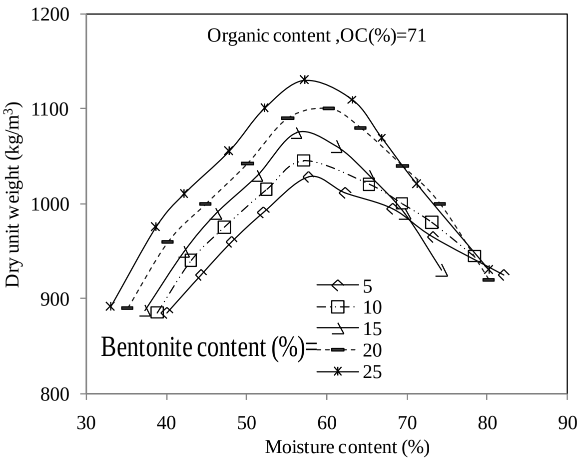


Figure 19: Effect bentonite content on dry density of stabilized soil at varying organic content

4. REGRESSION MODEL ANALYSIS

Based on the laboratory test result and using SPSS 16.0 software a model was developed which correlates the measured compressive strength (q_u) and calculated compressive strength from developing equation. To depict the validity of the measured q_u, and model was developed and then calculate the value of q_u and analyzed.

[redacted] (1)

Where, a_1, a_2, a_3, a_4, a_5 and a_6 are coefficient of regression line and w=Water content (%), w_L=Liquid limit (%), *C*=Admixture content (%), *CT*=Curing days, *S*=Sand content (%), In order to obtain a more accurate regression model the curing time was left out as a descriptor variable in the regression equation. The model that gives the best correlation is of the following from again.

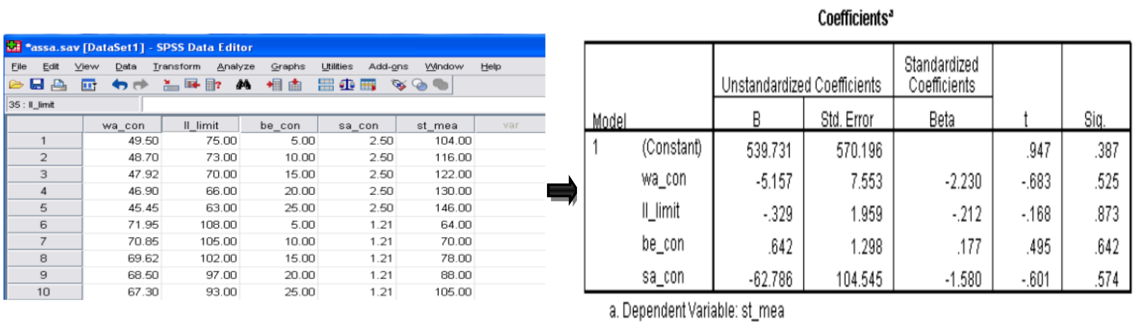
[redacted] (2)

The reliability and accuracy of the model were checked by comparing the predicted values of compressive strength from this model and the measured values, and computing the correlation coefficient. In this study, water content, liquid limit, admixture content and sand content are consider as independent variables, measured strength are consider as a dependent variable.

4.1 Regression Model of Stabilized Soil at Varying Curing Period

4.1.1 Bentonite stabilized soil at curing period of 1 day and 3 days

For regression analysis, water content, liquid limit, admixture content and sand content are considered as independent variables, while, the measured strength are considered as a dependent variable is evident in Figure 20. Moreover, to depict the validity of the measured compressive strength against the computed values, the following Equation 3, was developed using the unstandardized cofficients as presented in Figure 20.


*assa.sav [DataSet1] - SPSS Data Editor

	wa_con	ll_limit	be_con	sa_con	st_mea
1	49.50	75.00	5.00	2.50	104.00
2	48.70	73.00	10.00	2.50	116.00
3	47.92	70.00	15.00	2.50	122.00
4	46.90	66.00	20.00	2.50	130.00
5	45.45	63.00	25.00	2.50	146.00
6	71.95	108.00	5.00	1.21	64.00
7	70.85	105.00	10.00	1.21	70.00
8	69.62	102.00	15.00	1.21	78.00
9	68.50	97.00	20.00	1.21	88.00
10	67.30	93.00	25.00	1.21	105.00

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	539.731	570.196		.947	.387
	wa_con	-5.157	7.553	-2.230	-.683	.525
	ll_limit	-.329	1.959	-.212	-.168	.873
	be_con	.642	1.298	.177	.495	.642
	sa_con	-62.786	104.545	-1.580	-.601	.574

a. Dependent Variable: st_mea

Figure 20: Variables and coefficients for model of SPSS analysis

[redacted]

(3)

(4)

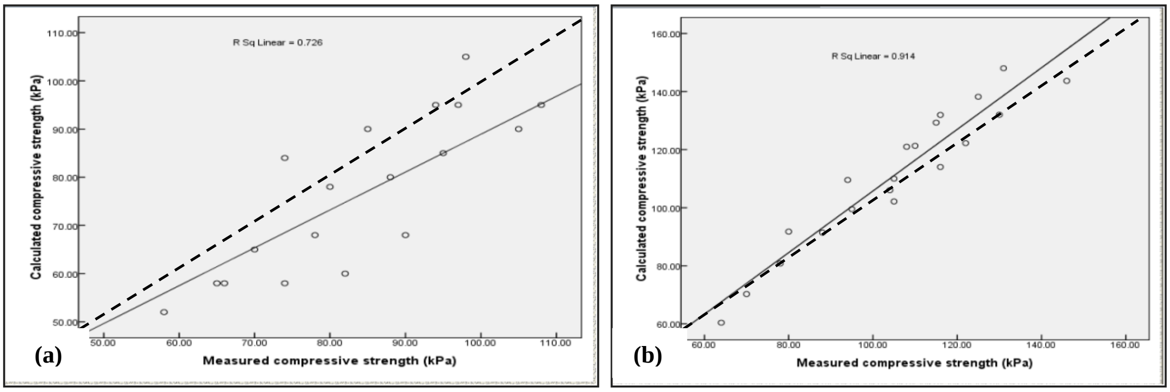


Figure 21: Cross plot of measured compressive strength and computed values for bentonite admixture at (a) 1 day and (b) 3 days

The cross plot of the values of computed compressive strength obtained from the application of equation 3 and 4 against the measured values using the linear regression model. Figure 21 illustrate a plot of the values of computed compressive strength with measured values using the linear regression model. The dot straight line in the figure represents the line of perfect equality, where the values being compared are exactly equal. The correlation coefficient (R^2) at 95% confidence interval was 0.726 (Fig. 21 (a)) and 0.914 (Fig.21 (b)), meaning roughly that 72.6 and 91.4%, for 1 and 3 days curing, respectively, of the variance in compressive strength is explained by the model.

4.1.2 Lime stabilized soil at curing period of 14 day and 28 days

On the basis of input value of water content, liquid limit, lime content and laboratory measured strength in SPSS 16.0 , the develop equation 5 and 6 are given below.

[redacted] (5)

[redacted] (6)

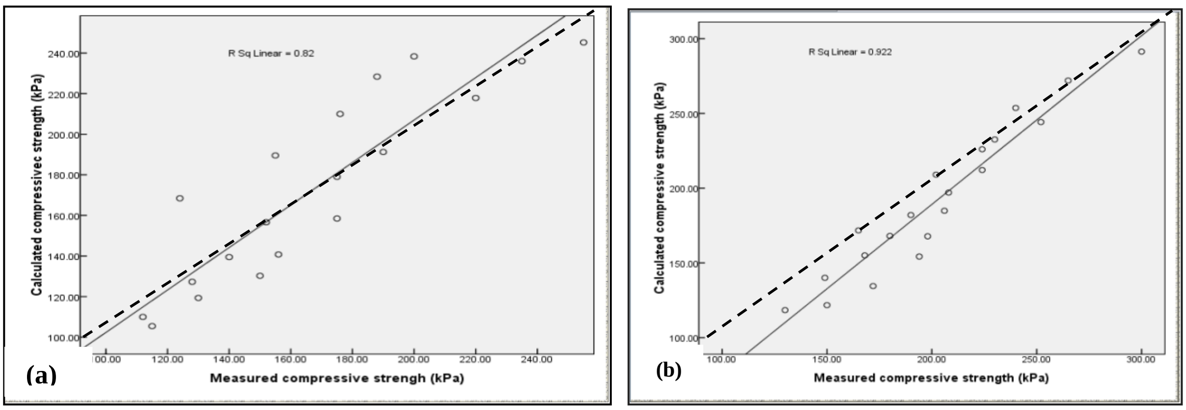


Figure 22: Cross plot of measured compressive strength and computed values for lime admixture at (a) 14 days and (b) 28 days

4.1.3 Cement stabilized soil at curing period of 3 days and 14 days

On the basis of input value of water content, liquid limit, cement content and laboratory measured strength in SPSS 16.0 , the develop equation 7 and 8 are given below.

[redacted] (7)

[redacted] (8).

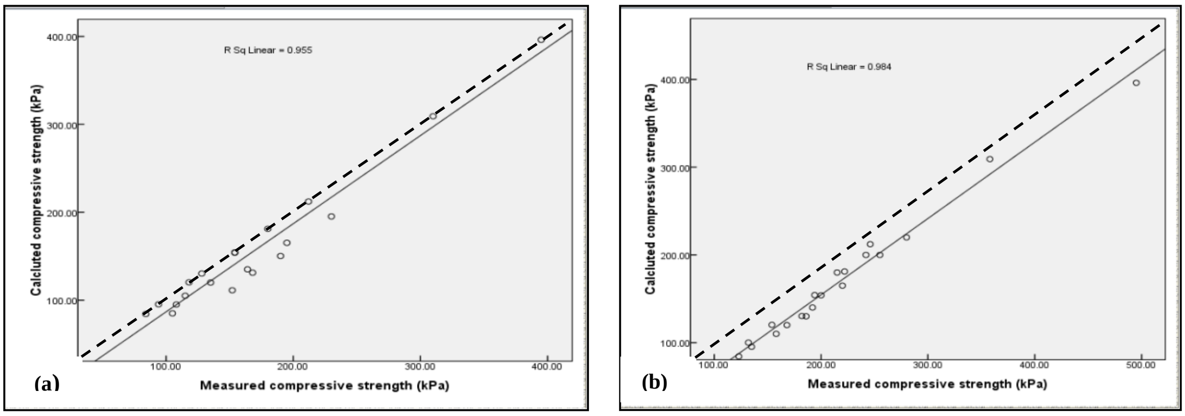


Figure 23: Cross plot of measured compressive strength and computed values for cement admixture at (a) 3 days and (b) 14 days

4.1.4 Cement and bentonite stabilized soil at curing period of 28 days and 14 days

On the basis of input value of water content, liquid limit, cement and bentonite content and laboratory measured strength in SPSS 16.0 , the develop equation 9 and 10 are given below.

[redacted] (9)

[redacted] (10)

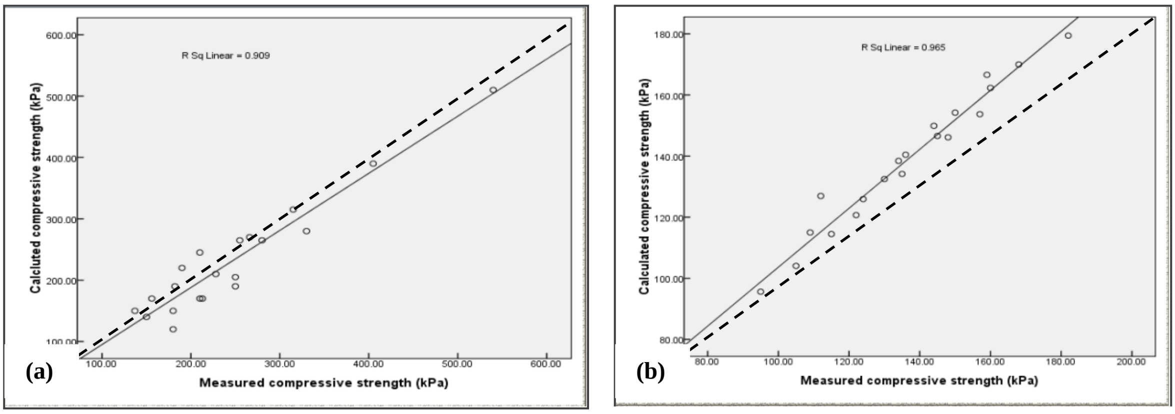


Figure 24: Cross plot of measured compressive strength and computed values for (a) cement admixture at 28 days and (b) Bentonite admixture at 14 days

5. CONCLUSIONS

From the observation, the research concluded that, the chemical admixtures improve the engineering properties of stabilized soils, especially after long curing period. The liquid limit of stabilized soil decreases with the increases of admixtures content, however, for 100% mixing water soil; the liquid limit was more than 50% mixing water condition. The maximum dry density is found to increase while the optimum water content is found to decrease with increase of admixtures content. The computed q_u was almost same as the laboratory measured value and the degree of accuracy was more reliable. The higher strength is obtained from samples that have been cured for 28 days compared with the 1, 3, 7, 14-days cured samples and also obtained that cement stabilized samples shows high strength than lime and bentonite stabilized organic soil samples.

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EFFECT OF ACID RAIN ON GEOTECHNICAL PROPERTIES OF COMPOSITE FINE-GRAINED SOIL

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ABSTRACT

Generally the atmospheric releasing gaseous pollutants like sulfur dioxide and oxides of nitrogen travels very far away from their sources. These gaseous pollutants combined with moisture in the atmosphere to form sulfuric acid and nitric acid solutions that fall to earth in rain is termed as acid rain. Though there is no significant record of acid rain in Bangladesh but it may be a happen due to the increasing urbanization, heavy industrialization, climate change and global warming. The main objectives in this study is to pridict the probable effect of acid rain on geotechnical properties of soil inorder to increase our understanding and taking proper consideration during construction of substructure. At first, the collected soil samples were treated with two different acid of varying concentration through three setups. After that, the geotechnical properties of original and treated samples were determined. These test result shows that the physico-chemical behaviours, free swell index and consistency of soil were strongly affected by acid treatment due to rapid leaching of cations and the diminution of inter-particle repulsion. Besides that the unconfined compressive strength (UCS) and shear strength parameters decrease with the addition of acid rain and their increasing concentration because of the changing of internal structural of soil and reduction of the electric forces as concentration of exchangeable cations reduced on acid treatment.

Keywords: *Acid rain, Physico-Chemical Properties , Consistency, Swelling and strength properties.*

1. INTRODUCTION

Acid deposition into the soil in the form of acid rain is the result of sulfur and nitrogen dioxide being emitted into the air caused by the growth of industrialization. This leads to the creation of acidic compounds that are absorbed by clouds, which in turn makes rain or snow more acidic, traveling and landing in a different place, and changing the acidity of the water or land on which the chemicals fall. As most of the Asian countries are in the process of development, they are not responsible for acid rain. Indeed, while China and India account for 8 per cent and 2 per cent respectively of the cumulative CO_2 emissions over the period 1900-2005, the US and the EU are responsible for more than half of these emissions. It not only affects the place where the pollution is emitted but it can be transported over thousands of miles. For example, Japan and Korea are suffering from Chinese pollution through acid rain, while Bangladesh suffers from Indian pollution. For example, in developing countries, people tend to rely on wood, dung or crop residues for domestic energy (www.articleinput.com, 2009). Exposure to this polluted air leads, among other things, to have adverse impacts on forests, freshwaters and soils, killing insect and aquatic life-forms as well as causing damage to buildings and having impacts on human health.

Several processes can result in the formation of acid deposition in to the soil. In dry deposition, Nitrogen oxides (NOx) and sulfur dioxide (SO_2) released into the atmosphere from a variety of sources fall to the ground and converted into acids when these deposited chemicals meet water. In the case of wet acid deposition, nitrogen oxides (NOx) and sulfur dioxide (SO_2) are converted to nitric acid (HNO_3) and sulfuric acid (H_2SO_4) through oxidation and dissolution (Pidwirny, M. 2006). The H+ content of acid rain falling for a small duration is very low yet its influence on the physical, chemical and engineering properties of soil will be alarming in near future as ever growing industrialization/pollution will keep on substantially decreasing its pH. The enhanced rate of leaching of cations and absorption of H^+, $SO4_2^-$, NO_3^- and CO_3^{2-} alter its physical, chemical and engineering properties (Masashi, K., et al., 1997). It is thus one of the most influential environmental factors which directly affect the properties of soil (Yamaguchi, H, 1993). The present study is carried out on soil, collected from Khanjahan Ali Hall at Khulna University of Engineering & Technology (KUET) in Khulna, Bangladesh to find out physico-chemical and the engineering properties on exposure to acid rains of various simulated strengths.

2. METHODOLOGY

2.1 Geotechnical properties of Collected Soil

To analyse the effect of acid concentration on soil nearly greyish coloured silty clay was used in this study, collected from backside of Khanjahan Ali Hall at Khulna University of Engineering & Technology (KUET) in Khulna, Bangladesh. The collected sample was oven dried at about 105^0C to remove the moisture content and suppress the microbial activities. Soil samples were screened through 4.75 mm sieve to obtain the geotechnical properties of original sample. The geotechnical property of original soil samples is shown in Table 1.

Table 1: Characteristics of original samples

Characteristic	**Result**
P^H	6.83
Organic matter (%)	8.57
Liquid Limit (%)	54
Plastic Limit (%)	30
Shrinkage Limit (%)	18
Free Swell Index (%)	20
Specific Gravity	2.78
Unconfined Compressive strength (UCS), q_u (MPa)	8.09
Cohesion, C (KPa)	44.4
Angle of Internal Friction, φ (degree)	18.26

According to the AASHTO classifiation systems, the soil is characterized as A-7-6 and acoording to the Unified soil classification sytems, samples are naming as CL (Clay with low plasticity). The particle size distribution of the collected original soil is show in figure 1. The diagram below shows the grain size distribution of the clayey soil collected for this study.

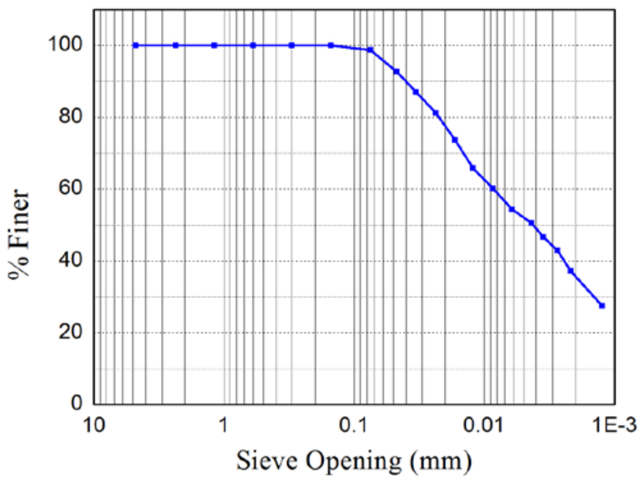

Figure 1: Particle size distribution of original soil

2.2 Preparation of Tested Sample

The oven dried samples were grinding and passing through 4.75 mm sieve. Soil samples which are passing through 4.75 mm sieve were mixed with different concentration of H_2SO_4 and HNO_3 acid and distilled water as shown in Table 2. The proportion of H_2SO_4 and HNO_3 acid were taken as near as to the concentration of these acids in acid rain. These samples were shaken with mechanical shaker for about 8 hours to obtain a homogeneous mixture of soil and acid. After proper shaking of the samples were kept overnight at room temperature. The treated samples were then filtrated for the extraction of water from it and kept it on the ground for sun drying. The drying samples were then grinding and sent to the laboratory for determining the geotechnical properties of that soil.

Table 2: Combination scheme of soil samples

	Samples ID	Samples Description				Strength of Acid
		Soil sample(Kg)	1N H_2SO_4(ml)	1N HNO_3(ml)	Distilled Water(ml)	
1st setup	Original soil	1	--	--	--	--
	S-1	1	5	--	995	0.005 N
	S-2	1	10	--	990	0.01N
	S-3	1	20		980	0.02N
	S-4	1	50	--	950	0.05N
2nd setup	N-1	1	--	5	995	0.005 N
	N-2	1	--	10	990	0.01N
	N-3	1	--	20	980	0.02N
	N-4	1	--	50	950	0.05N
3rd setup	SN-1	1	5	5	990	--
	SN-2	1	10	10	980	--
	SN-3	1	20	20	960	--
	SN-4	1	50	50	900	--

2.3 Testing protocol

A series of tests were conducted to determine the effect of acid on the consistency, swelling, shrinkage and strength characteristics of soil through three test setup. The pH of the soil is a useful variable in determining the solubility of soil minerals and the mobility of ions in the soil. The P^H of the original and the acid treated soils were determined by ASTM D4972. To perform these test 10g of air dried samples were mixed with 10mL of distilled water thoroughly and kept it about one hour. After that the value of P^H were recorded by P^H meter.

Consistency of soil is expressed in terms of liquid limits and plastic limits are extensively used to characterize the fine-grained fractions of soils. Factors which significantly affect the consistency of soil depends on the type of soil, electrical charge of exchangeable cations absorbed by soil particles and concentration of cations in soil water(Masashi, K., et al.,1997). Acid rain increases the exchange between H^+ and cations like potassium (K), magnesium (Mg) and calcium (Ca) in the soil. These cations are liberated into soil and can be rapidly leached out in soil solution along with sulphate from acid input (Van Breeman et al., 1984). The effects of shrinkage of fine grained soils are of considerable significance to cause serious damage to small building and highway pavements. Consistency of the original and treated soil samples were determined by Atterberg limit test(ASTM D-4318-10). Specific gravity is is the indirect measurement of density of soil and used in phase relationship of soil such as void ratio and degree of saturation. Specific gravity of original and ash treated soil samples were determined according to ASTM D854-10. Free swell index of soil samples were determined as per as IS: 2720(Part-40). Ten grams of soil samples were put into two 100ml glass cylinder and the remaining 90ml of the cylinder was filled with distilled water and kerosene respectively. After keeping it overnight the volume of soil in each cylinder was measured and from this volume free swell index of soil was determined from the equation shown in below.

Free swell index, %= $$\frac{Volume\,of\ in\,distilled\ water - Volume\,of\ in\,distilled\ \mathrm{ker}\,osene}{Volume\,of\ in\,distilled\ \mathrm{ker}\,osene}x100$$

All the test specimens were prepared at the optimum moisture content before performed the remaining selected properties. After adding water the test specimens were prepared with the standard Proctor compaction test (ASTM D 698). The specimens were demolded after completion of compaction and samples of different size were prepared as per requirement for performing the selected test in uncured condition.

The uncured unconfined compressive strength of the specimens was determined according to ASTM D-2166-98 and cylindrical specimens of 36-mm-diameter and 71-mm-lengthwere used for this test. Drained shear strength parameters(c & φ) were determined by direct shear test (ASTM D3080-3) of the compacted uncured soil specimens. To perform these tests, three samples of 60mm diameter and 25 mm height were prepared from the compacted samples.

3. RESULTS AND DISCUSSIONS

3.1 Physico-Chemical Properties

The variations of P^H of original and treated soils are illustrated in Figure 2. The value of P^H decreases as the concentration of acid increases in all the three cases. This Figure shows that the value of P^H decreases more rapidly in case of 1st setup containing H_2SO_4 acid treated soil than 2nd setup containing HNO_3 treated soil for the same acid concentration. In case of 3rd setup where both H_2SO_4 and HNO_3 were using for treating the soil is much more susceptible to decrease the value of P^H than the 1st and 2nd setup. The decrease in soil P^H is attributed to increases in the exchange between H^+ in acid rain and cations like magnesium (Mg) and calcium (Ca) in the soil.

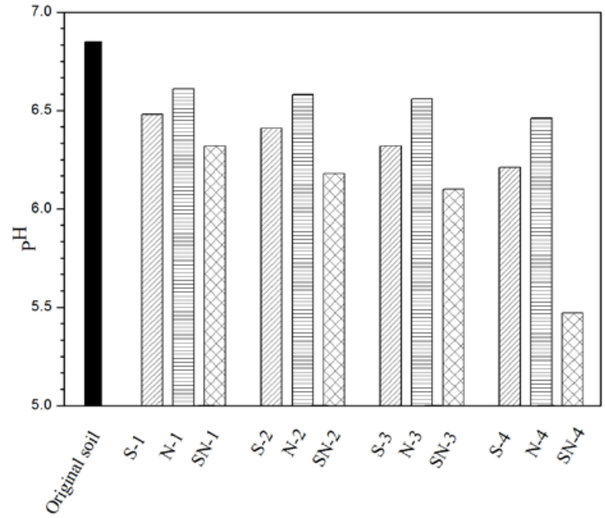

Figure 2: The effect of various acids with different concentrations on P^H of soil

The percentage of organic matter present in the soil is decreased for all the three setup and with the increase of acid concentration as shown in Figure 3. This Figure shows that the percentage of organic matter decreases more rapidly in case of 2nd setup than 1st setup for the same acid concentration. In case of 3rd setup, the decrease in percentage of organic matter is much more susceptible than the 1st and 2nd setup. These reductions in organic matter of soil depicts that it's digested as the concentration of acid increases and is more susceptible for HNO_3 than H_2SO_4.

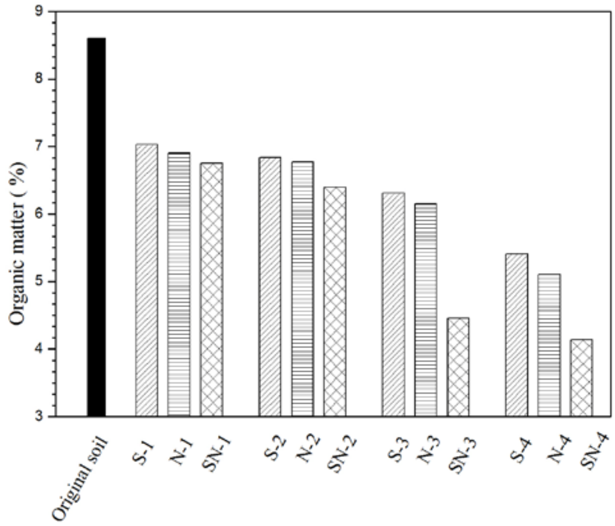

Figure 3: The effect of various acids with different concentrations on organic content of soil

3.2 Consistency Characteristics

Generally consistency of soil is expressed in terms of liquid limit, plastic limit and shrinkage limit. The change in liquid limit of soil for the various acid of different concentration is shown in Figure 4. This figure illustrated that the value of liquid limit decreased for the increase of acid concentration. The results indicate that using acid solutions as pore fluids decreases the liquid limit of the mixtures. The effect of H_2SO_4 acid on the liquid limit is more than HNO_3. In case of 3rd setup, the liquid limit is much more susceptible on acid than the 1st and 2nd setup. The similar trend were found in case of plastic limit is shown in Figure 5. This reduction in liquid limit and plastic limit of soil is the result of rapid leaching of cations and the diminution of inter-particle repulsion due to the addition of acid. Besides that, as the concentration of acid increases, the tendency of cation exchange increase which results a rapid movement of soil particles in a lower water content and thus the liquid and plastic limit of soil decreases.

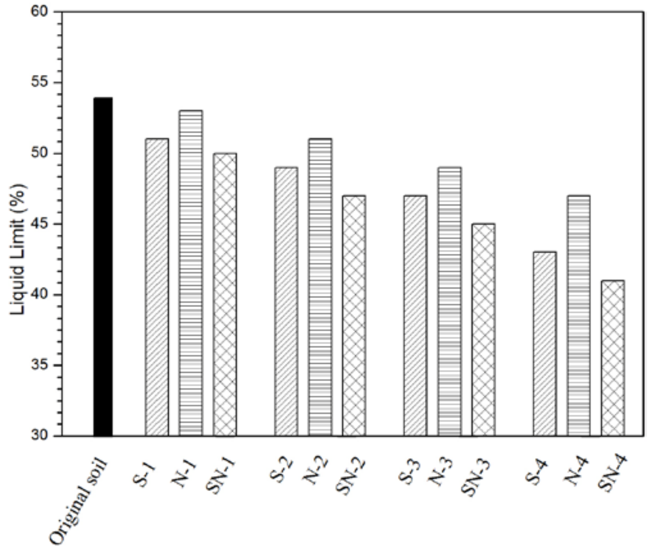

Figure 4: The effect of various acids with different concentrations on liquid limit of soil

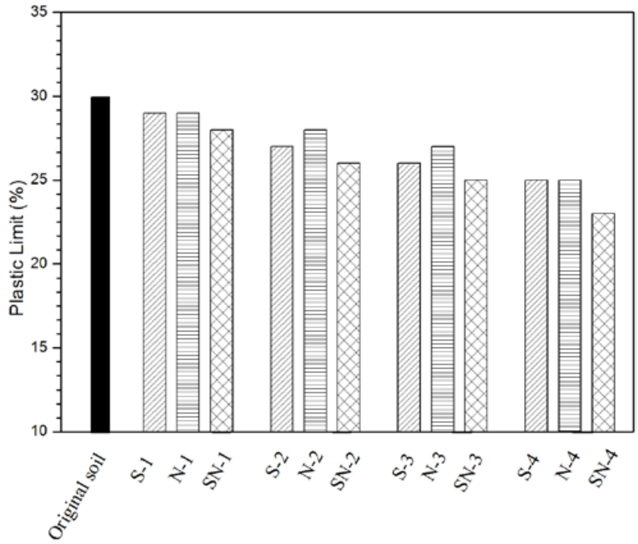

Figure 5: The effect of various acids with different concentrations on plastic limit of soil

On the other hand the shrinkage limit shows a different trend from the liquid limit and plastic limit as shown in Figure 6. In this illustration, the value of shrinkage limit increases with the increases of acid concentration. This increase in shrinkage limit is attributed to increase in inter particulate distances due to reduction in the forces between soil particles.

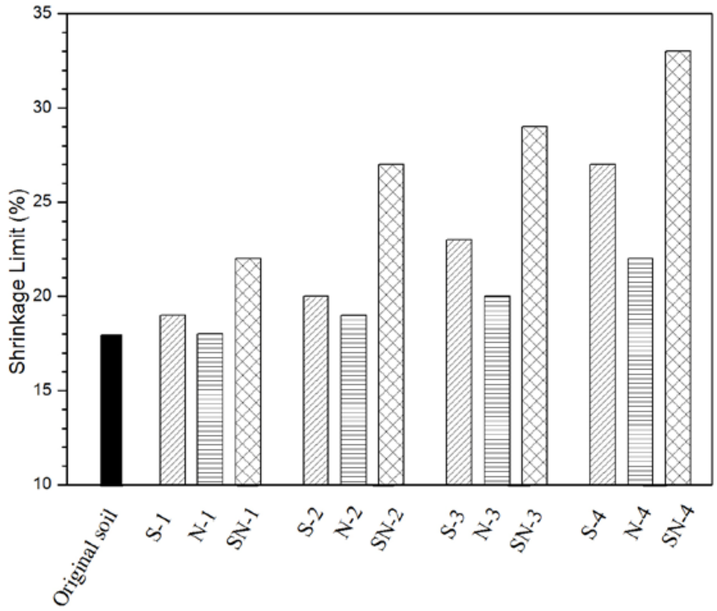

Figure 6: The effect of various acids with different concentrations on shrinkage limit of soil

3.3 Free Swell Index

The variation of free swell index with various acid of different concentration is illustrated in Figure 7. This figure depicts that the value of free swell index decreases with the addition of acid and increasing of acid concentration. The H^+ in the acid increases the rate of cation exchange which results a rise of free energy for forcing water to leave out from the inter-particle region and leads to a decrease in swelling volume. As the concentration of acid increases the free energy for removing water into the inter-particle region also increases as well as the swelling volume of soil decreases.

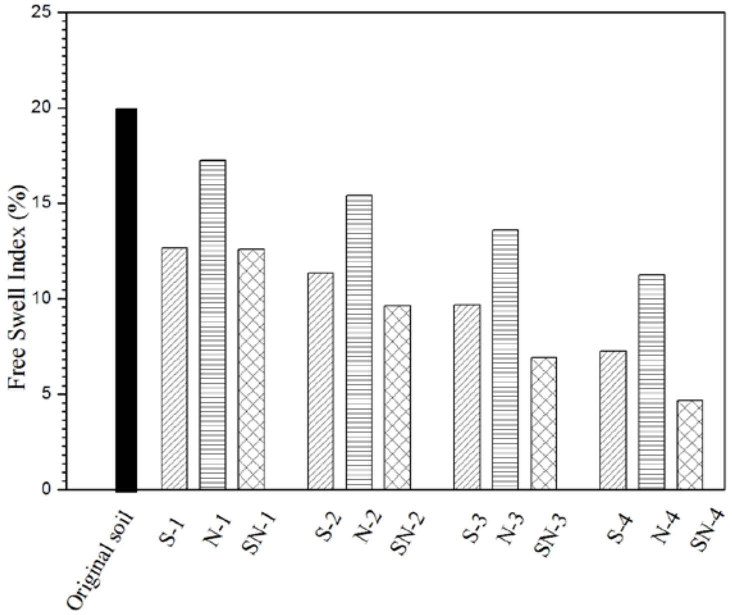

Figure 7: The effect of various acids with different concentrations on liquid limit of soil

3.4 Specific Gravity

The change in specific gravity for the addition of various acid of different concentration is illustrated in Figure 8. There is no significance change in specific gravity of soil for acid of low concentration. The specific gravity of soil decreases with the increases of acid concentration.

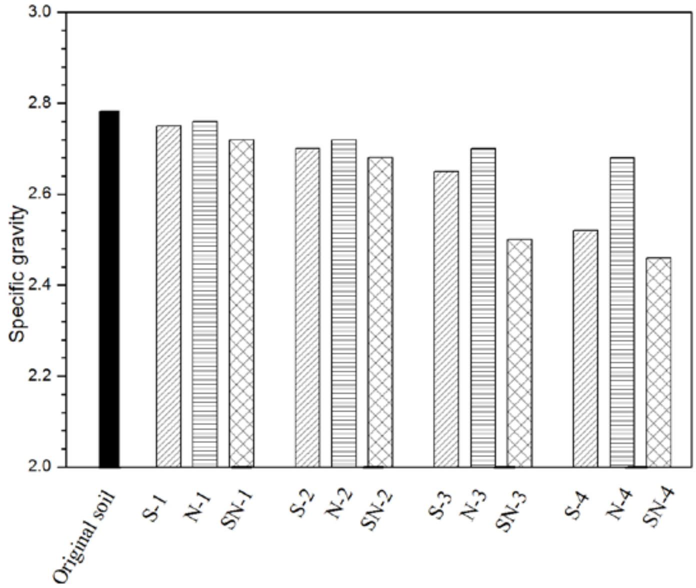

Figure 8: The effect of various acids with different concentrations on specific gravity of soil

3.5 Strength Characteristics

The UCS of original soil and acid treated soil are illustrated in Figure 9. This figure depicts that there is a little effect of acid on the compressive strength of soil. The effect of H_2SO_4 acid on the UCS is more than HNO_3. In case of 3rd setup, the UCS is much more susceptible on acid than the 1st and 2nd setup. The compressive strength of soil decreases as the addition of acid and for their increasing amount of acid concentration. When acid were mixed with soil, the inter-particle attraction of soil decreased. The particle of soil moves more rapidly and changes their internal structure through cations exchange with H^+ causes decrease in UCS.

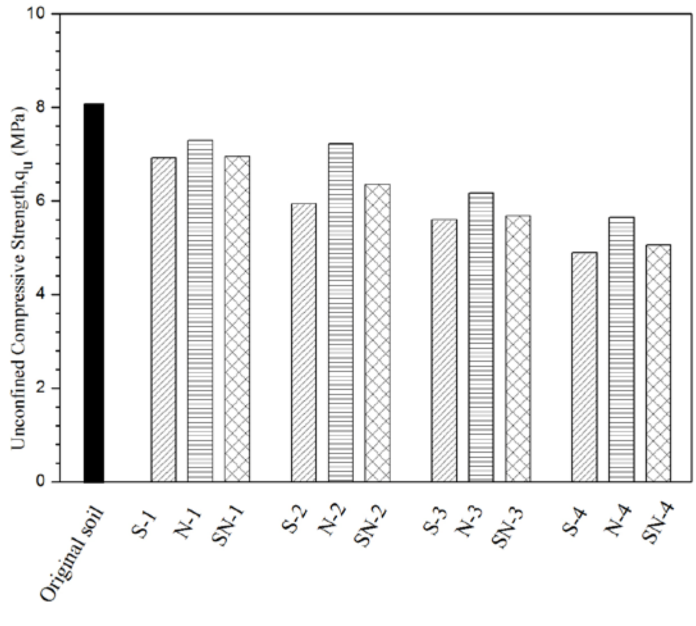

Figure 9: The effect of various acids with different concentrations on UCS of soil

The change in cohesion of soil due to the acid treatment of varying concentration is shown in Figure 10. This figure illustrate that the value of cohesion decreases with the addition of acid and to the increases of acid concentration.

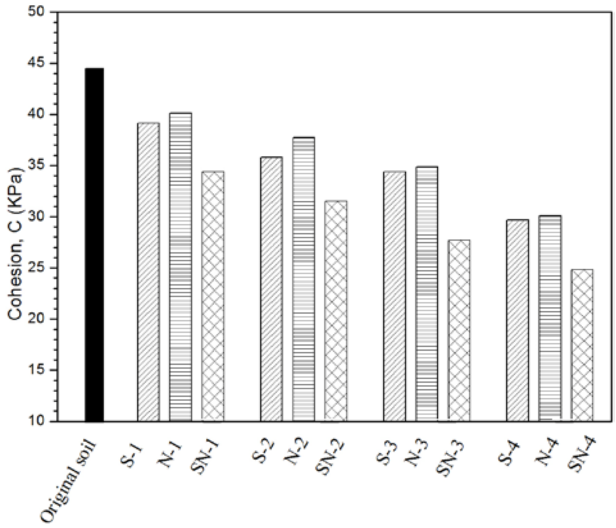

Figure 10: The effect of various acids with different concentrations on cohesion of soil

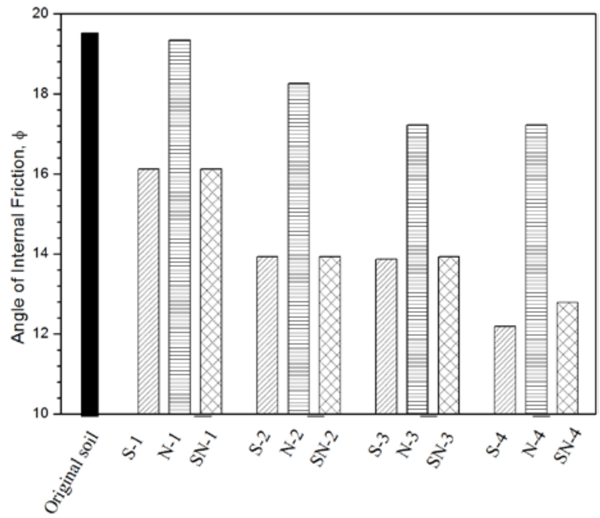

Figure 11: The effect of various acids with different concentrations on liquid limit of soil

4. CONCLUSIONS

A study has been conducted to investigate the probable effect of acid rain on the geotechnical properties of soil. Though there is no record of significance acid rain in Bangladesh but in the recent time Bangladesh is hardly affected by the climate change. Generally two acids are formed after acid rains (H_2SO_4 and HNO_3) which are responsible for the changing of geotechnical properties of soil. In this study three setups were conducted to observe the effect of acid on some geotechnical properties of soil. The following conclusions, based on the test results in this study, are drawn.

1. Acid rain results in changes in physico-chemical characteristics of soil due to cations exchange between H^+ and cations present in soil. P^H of the soil decreases as the addition of acid and with their increasing concentration.
2. Using acid solutions as pore fluids decreases the liquid limit and plastic limit of the mixtures. Higher cation valance and acid concentration cause higher decrease in liquid limit and plastic limit. On the other hand the shrinkage limit of soil shows opposite trend due to reduction in the forces between soil particles.
3. The swelling volume of soil decreases with the addition of acid and their increasing concentration by removing the water into the inter-particle region.
4. The value of UCS and shear strength parameters decreases with the addition of acid and their increasing concentration due to the changing of internal structural of soil and reduction of the electric forces as concentration of exchangeable cations reduced on acid treatment.

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DETERMINATION OF SHEAR STRENGTH OF COHESIVE SOIL BY ROD PENETRATION METHOD

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ABSTRACT

A rod penetration test was developed to determine the in-situ shear strength of cohesive soil. Two steel bar of 25mm and 16mm diameter were used for penetration. The rod was penetrated in a mold that was filled by the compacted soil sample. A static load of 80 kg was applied on the soil in the mold with 10kg increments. The rates of penetrations were measured. Then the unconfined compression test of the soil samples were done to get the shear strength of the each soil sample. Then a correlation graph of shear strength of the soil and rate of penetration was found. These graphs are the calibration curve for that two rod. In-situ shear strength of soil can be determined easily using these calibration curves in field.

Keywords: *Rod penetration, shear strength, static load, calibration curve.*

1. INTRODUCTION

In geotechnical engineering, Standard Penetration Test (SPT) and other laboratory tests are mostly used to know the subsurface information. But performing only these test it is not possible to know the soil condition of all the zone of a site since these tests are performed at some selected points of a site. It may happen that the soil condition is very stiff where the SPT is performed whereas soft soil may exist at other locations or soft pockets may exist directly underneath the shallow foundation. In case of individual footing, there is a possibility of differential settlement which is very dangerous for the structure. So it is necessary to check the bearing capacity of the soil under each footing. For this check, a testing device should be developed locally so that anyone can perform the test to get shear strength and bearing capacity of the soil. Besides this, for the shallow foundation up to 2 - 3 storied residential building in the remote area where SPT and laboratory tests cannot be performed, such a portable testing device can be used easily by local people.

2. BACKGROUND

There are some penetrometer devices which are used in subsoil investigation in field such as proving ring penetrometer, the hand-held electronic cone penetrometer, Swedish weight sounding method, dynamic cone penetrometer, vane shear test etc. The proving ring penetrometer (ASTM D 1558-99) is a cone type of penetrometer which is used to determine the penetration resistance of soils in shallow exploration work. In conjunction with the standard laboratory and field test, this instrument has proven to be an extremely useful tool for quick field checks. The hand-held electronic cone penetrometer (*Kees*, 2005) electronically records the force required to push the probe into the ground and depth reading for computer down-load and analysis. The Swedish weight sounding test (Suemasa et al, 2005) is one of the oldest and the most commonly used test to investigate shallow soil strata. There is, however, an inevitable disadvantage on the interpretation of the test results, which is derived from using two different indicators; load and the half turn's number. The half turn's number is, however, difficult to be related with load, except for using any empirical relationship. Vane shear testing (Bowels, 1996) is one of the most common in-situ methods for the estimation of the undrained shear strength of the soil. The shear strength of the material is calculated from the torque by dividing by a constant which depends on the dimensions and the shape of the vane. Torque measuring device is not readily available for local people. In this case, this portable penetrometer can be used widely. Its procedure is very easy. No need of well-trained person to do this. Anyone can find out shear strength of soil without any training. Not only this, it is also very cheap. So it can be used everywhere in the world to find out the shear of soil easily.

3. METHODOLOGY

If we want to penetrate a rod or some other things into soil, the soil will resist its penetration due to shear strength of it. The more shear strength exists in the soil, the more it will resist the bar (Aoki, 2007). It can also be said that for the same load, the more shear strength exists in the soil the less penetration of the rod will occur and vice versa. So there is relation between the shear strength of soil and the penetration. Therefore a correlation graph can be made by which shear strength of soil can be measured from the rate of penetration.

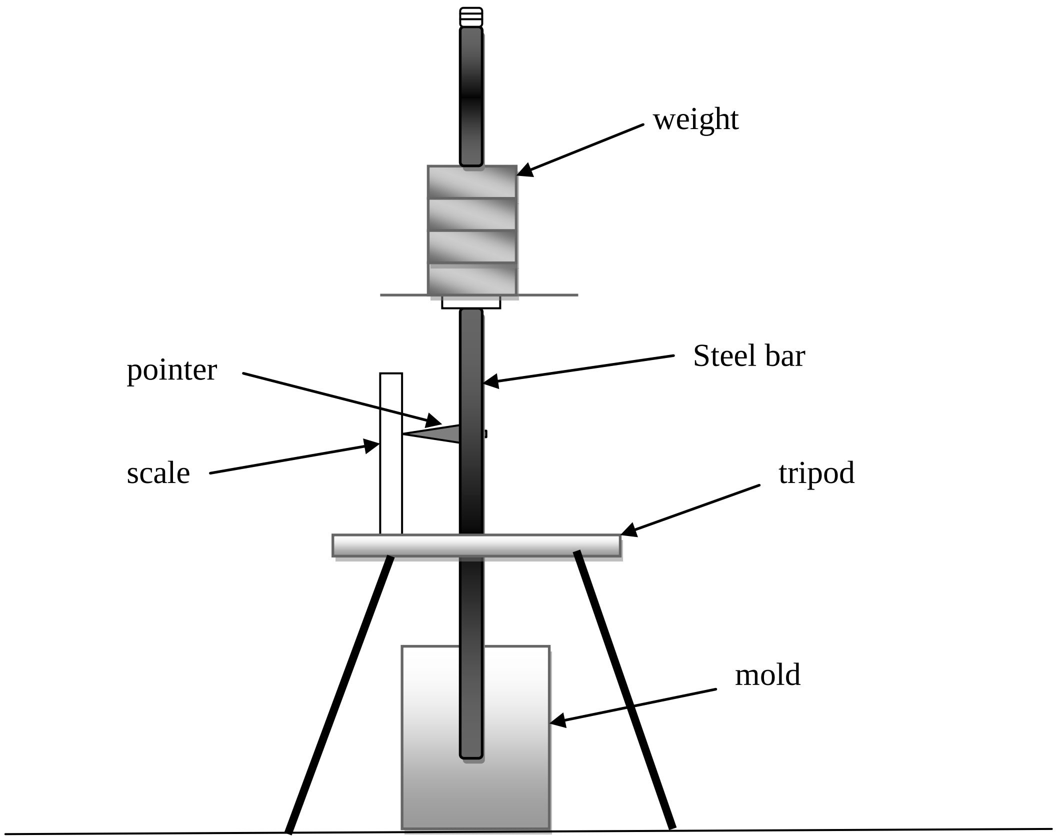

Fig 1: Experimental Setup for Penetration Test

To do this, two stainless steel bars of 25mm and 16mm diameter and 750mm length was used for penetration. The experimental setup is shown in Fig. 1. There is a provision to connect the extra rod of same diameter with them to measure the shear strength of soil at greater depth in the field. A tripod was used to hold the rod vertically. Bubble level was used in tripod to ensure the vertical load. Soil samples from were collected from different locations. Samples were sieved by no.4 sieve and their index properties were determined. Index properties were determined by following ASTM T89 guideline. The moisture content of the soil sample must be kept within the liquid limit and plastic limit. The penetration was occurred in a mold of 100mm diameter and 150mm height.

The mold was filled in three layers and each layer was compacted by standard hammer with 25 blows. The load of was given in 10kg each time up to 80kg. The rod penetrated through the soil of the mold. Depth of penetration was measured by a scale and a pointer. The pointer is attached with the rod and can move with the movement of the rod along the scale. Then cumulative depth of penetration vs. cumulative load graph was drawn. A best fitted line was found from the graph. The slope of this line shows the penetration rate of the rod into the soil sample. The Unconfined compression test of the soil samples were done following the ASTM D2166. Thus depth of penetration and shear strength of different soil sample were measured. Then a correlation graph of shear strength of the soil and rate of penetration was found.

4. RESULTS AND DISCUSSION

Different penetration for different loading was found in the test. Among the two of bar, 16mm bar can be used for stiff clay and 25mm for soft clay. Soil samples were taken from Mohakhali and Kollayanpur sites.
Index properties were:
Soil sample from Mohakhali site: Liquid Limit= 51, Plastic Limit= 15, Plasticity Index= 36
Soil sample from Kollayanpur site: Liquid Limit= 56, Plastic Limit= 18, Plasticity Index= 38
Keeping moisture content between the liquid limit and plastic limit different soil samples of different moisture content were made for penetration test. Cumulative penetration vs. cumulative load graph was drawn as shown in Fig 2 and Fig 3 :

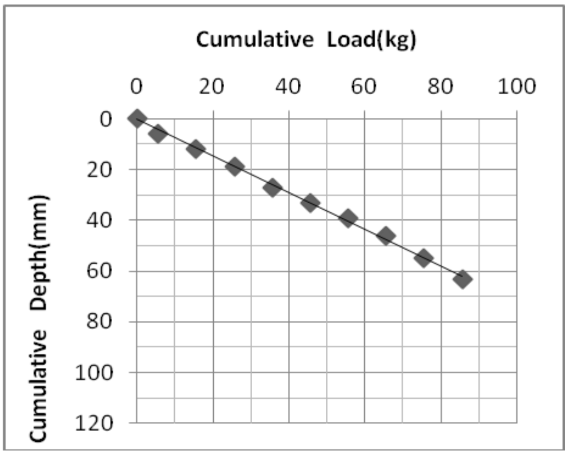

Fig 2. cumulative penetration vs. cumulative load graph for 25mm rod

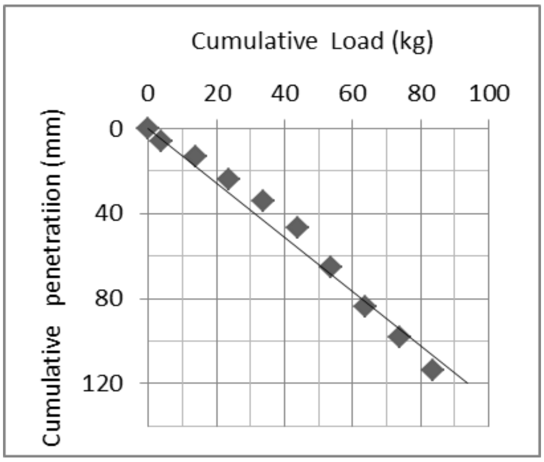

Fig 3. cumulative penetration vs. cumulative load graph for 16mm rod.

A best fitted straight line was found from the cumulative penetration vs. cumulative load graph. The slope of this line gives the penetration rate of the sample. Penetration rate was different for the two bars of the same sample for difference in pressure inserted on the soil sample. Unconfined compression test of the soil samples were done to get the shear strength of the soil. Now, the calibration curves are found by plotting shear strength vs. rate of penetration graph. The calibration curves for two bars were different. These calibration curves can be used in field.

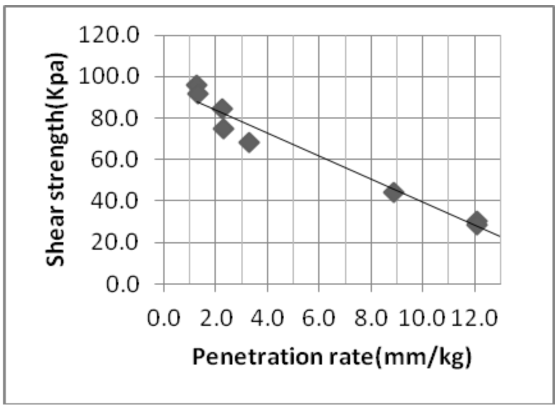

Fig 4: Calibration curve for 25mm rod

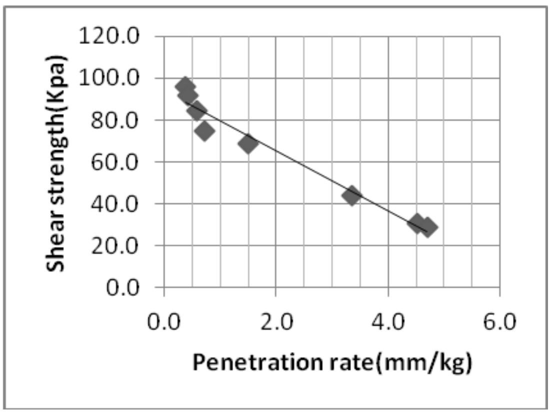

Fig 5: Calibration curve for 16mm rod

5. CONCLUTIONS

The portable rod penetrometer was calibrated to find in-situ shear strength of clayey soil. Both the 16mm and 25mm diameter rod penetrometer could be successfully calibrated in the laboratory. Therefore, the developed rod penetration test may be used to determine in-situ shear strength of clayey soil. However, more research is necessary to make in acceptable to practicing engineers in Bangladesh.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

3D FINITE ELEMENT ANALYSIS TO EVALUATE THE SHAPE EFFECT OF RECTANGULAR ANCHOR FOUNDATION IN DENSE SAND

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ABSTRACT

The anchor foundation is frequently utilized in many civil engineering structures to provide the uplift resistance. For the economic and safe design of such foundations, the engineers should understand the failure mechanism associated with it, very well. In this study, a three dimensional finite element model is validated for the evaluation of the shape effect of the square and rectangular vertically uploaded anchor foundations embedded in dense Toyoura sand with embedment ratio of two and width of 50 mm. The comparison of the experimental load-displacement relationships, peak loads, displacements at the peak loads, the extensions of the heaving of the ground surface with the numerical results shows satisfactory agreement, which increases confidence to propose such a numerical model for the practical design..

Keywords: *Rectangular anchor, Dense sand, Model test, 3D FEM, Shape effect.*

1. INTRODUCTION

The anchor foundation with different depth, size, inclination, and shape is widely utilized for towers, bridges, and other various structures to resist the tensile forces, so, a number of research works can be cited, regarding the pulling out of such kind of foundations. Most of the researchers (Majer, 1955; Mors, 1959; Balla, 1961; Baker & Kondner, (1966); Matsuo, 1968; Meyerhof & Adams, 1968; Vesic, 1971; Vermeer & Sutjiadi, 1985, Murray & Geddes, 1987, etc.) concentrated on two-dimensional (2D), axis-symmetric or plane strain, design approach, based on limit equilibrium, cavity expansion, and limit state theory. But, the failure in the real soil, having the elasto-plastic and strain softening constitutive nature, is highly progressive, and which cannot be described by limit state theory, as it is based on rigid elastic-perfectly plasticity assumption. Among the very few researchers who applied finite element method (FEM) for anchor problems, Rowe & Davies (1982) applied elasto-plastic FEM with Mohr-coloumb failure criteria and continuous dilatancy, providing highly conservative design charts, Vermeer & Sutjiadi (1985) used non-associated elasto-perfectly plastic FE model (Borst & Vermeer, 1984) to validate their proposed design equations, and Tagaya et al. (1983) used Lade's (Lade and Duncan 1975) original incorrect version of elasto-plastic constitutive law for their 2D anchor problems. Sutherland (1998) stated that the results obtained by above researchers using FEM were not satisfactory as they did not consider the progressive failure, linking to the development of narrow shear bands in the ground. Walters & Thomas (1981) showed the importance of the non-associated strain hardening-softening and mobilized dilatancy for proper simulation of the shear zone propagation above trapdoors, using elasto-pastic finite element numerical model. In this regard, Sakai & Tanaka (2007 & 1998) simulated the shear band propagation, accurately, in their 2D anchor and trapdoor problems using elasto-plastic FEM, considering non-associated strain hardening-softening with shear band effect and mobilized dilatancy. Ovesen (1981), Murray & Geddes, Dickin (1988), Sakai & Tanaka (2009) and many other researchers found experimetally that the isolated anchor is not a 2D problem having the differences of uplift resistance between isolated & strip anchor. Meyerhof & Adams, Dickin, Frydman & Shamam (1989) proposed empirical correction factor ("Shape factor") to the peak uplift load for the rectangular anchor, calculated based on plane strain assumptions. Murray & Geddes, Vermeer & Sutjiadi proposed analytical design equations and Ovesen & Stromann (1972) proposed empirical design charts for isolated anchors. To the knowledge of the author, no researcher till to date has evaluated the three-dimensional (3D) effect for the rectangular anchor foundations in dense sand using FEM, except, recently, Sakai & Tanaka (2009) who employed 3D FEM to analyze the circular and square anchor problems.

In this study, an elasto-plastic 3D FE model, incorporating non-associated strain-hardening softening constitutive law with shear-band effect, is used to find the uplift load-displacement relationships of rectangular anchor foundations with different aspect ratio. The failure mechanism in the rectangular anchor foundations is compared with that in strip anchor.

2. EXPERIMENTAL SETUP

The device which was used for the experiments is shown in the **Fig. 1**. During the experiments, the flat and rectangular steel anchor foundations (breadth, B=50 mm and L/B=1, 2, 3, 4, 5, 6, and 8, where, L=length) were installed on the bottom center of a circular mold, made of hard vinyl chloride and diameter of 590 mm (as it did not give any boundary effect). The air-dry Toyoura sand (G_s=2.64, D_{50}=0.16 mm, e_{max}=0.98 and e_{min}=0.61, no fine content less than 0.075 mm) was pluviated through two sieves in the mold to construct the ground above the anchor foundations. The density of the ground was controlled by differing the sand falling height and the sieve-size. To measure the density of the ground, six 100cc-cylindrical samplers (diameter=50 mm) were placed on the mold's bottom. In this study, the ground was made dense with relative density (D_r) of 93±5%. The test was conducted with an embedment ratio, H/B=2, H=the depth of sand mass. Then, the anchors were pulled up vertically by a D.C. motor until the residual condition were reached and the pullout loads were measured using a load cell connected to the anchor rod. Pulling out speeds of the foundations were 0.03 mm/min and the upward displacements were measured using displacement transducer installed on the top of the anchor rod. The data of the load cell and displacement transducer were recorded in a computerized data acquisition system.

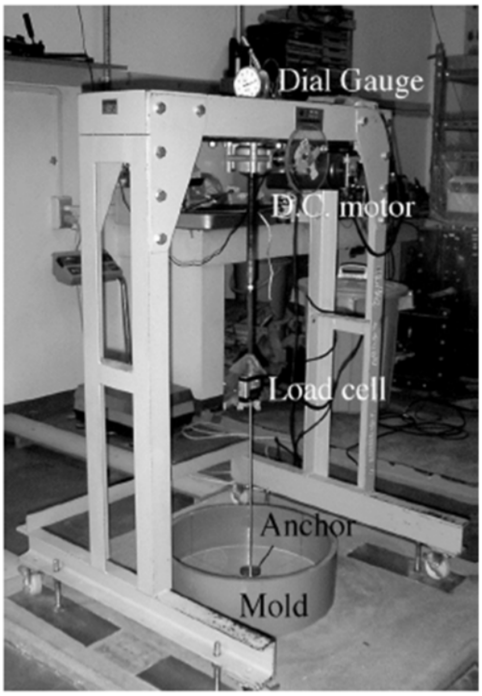


Figure 1: Setup for anchor pullout experiments.

3. NUMERICAL MODELING

Tanaka & Sakai (1993) applied the non-associated strain softening elasto-plastic constitutive law in a finite element code, considering the effect of shear band generation. The dynamic relaxation method (Tanaka & Kawamoto, 1988) was used for fast solution of the nonlinear equations, which is very essential for 3D problems to save computational time. To handle the mesh-size effect induced due to the softening of the material, a simplified version of the mesh-size dependent hardening modulus method was used by introducing a strain localization parameter, S, in the additive decomposition of the total strain increment:

$$S = \frac{F_b}{F_e} \tag{1a}$$

$$d\varepsilon = d\varepsilon^e + S d\varepsilon^p \tag{1b}$$

where, F_e is the volume of the each finite element, F_b is the volume of the shear band inside each finite element, $d\varepsilon^e$ and $d\varepsilon^p$ are elastic and plastic component, respectively, of the total strain rate, $d\varepsilon$.

In order to obtain the stable solution, yield function (*f*) Mohr-Coulomb type and the plastic potential function (Ψ) of Drucker-Prager type were employed. They can be defined by stress, $\boldsymbol{\sigma}$ and an unique softening parameter, k , as:

$$\mathbf{f(\sigma,\kappa) = 0} \tag{2a}$$

$$\mathbf{\Psi(\sigma,\kappa) = 0} \tag{2b}$$

From the normality rule, the plastic strain rate can be written as,

$$\mathbf{d\varepsilon^P = \lambda\frac{\partial\Psi}{\partial\sigma} = \lambda b} \tag{3}$$

where, λ is the proportionality factor. When the plastic flow occurs, $\mathbf{df = 0}$ is satisfied and the following the equations are obtained:

$$\mathbf{a^T d\sigma - A\lambda = 0} \tag{4a}$$

$$\mathbf{A = -\frac{1}{\lambda}\frac{\partial f}{\partial \kappa}d\kappa} \tag{4b}$$

where, $\mathbf{a = \frac{\partial f}{\partial \sigma}}$ **and** $\mathbf{A}$ is the softening modulus. The well known elastic constitutive relation is given by,

$$\mathbf{d\sigma = Dd\varepsilon^e} \tag{5}$$

where, $\mathbf{D}$ is a matrix of elastic constants. From the equations (1 to 5), the factor, λ, and the corresponding stress-strain relationship can be written as:

$$\lambda = \frac{\mathbf{a^T D d\varepsilon}}{\mathbf{A + Sa^T Db}} \tag{6a}$$

$$\mathbf{d\sigma = \left(D - \frac{SDba^T D}{A + Sa^T Db}\right)d\varepsilon} \tag{6b}$$

In this analysis, Mohr-Coulomb type yield function $\mathbf{f(\sigma,\kappa)}$ and Drucker-Prager type potential, $\mathbf{\Psi(\sigma,\kappa)}$ are used with the following generalized forms:

$$f = \frac{\sqrt{\mathrm{J_2}}}{g(\theta)} + 3\alpha(\kappa)\sigma_m = 0 \tag{7a}$$

$$\Psi = \sqrt{\mathrm{J_2}} + 3\alpha'(\kappa)\sigma_m = 0 \tag{7b}$$

$$\kappa = \int d\bar{\varepsilon}^P \tag{7c}$$

$$\left(d\bar{\varepsilon}^P\right)^2 = 2\left\{\left(d\varepsilon_x^P\right)^2 + \left(d\varepsilon_y^P\right)^2 + \left(d\varepsilon_z^P\right)^2\right\} + \left(d\gamma_{xy}^P\right)^2 + \left(d\gamma_{yz}^P\right)^2 + \left(d\gamma_{xz}^P\right)^2 \tag{7d}$$

where, σ_m is mean stress, $\mathrm{J_2}$ is the second invariant of the deviatoric stresses. In the case of Mohr-Coulomb model, $\mathbf{g(\theta)}$ is given below:

$$\mathbf{g(\theta) = \frac{3 - \sin\phi_{mob}}{2\sqrt{3}\cos\theta - 2\sin\theta\sin\phi_{mob}}} \tag{8a}$$

$$\sin 3\theta = -\frac{3\sqrt{3}\mathrm{J_3}}{2\mathrm{J_2^{1.5}}} \tag{8b}$$

where, ϕ_{mob} is the mobilized frictional angle, $\mathrm{J_3}$ is the third invariant of the deviatoric stresses. The friction and dilatancy are mobilized from peak value to residual, using the function, $\alpha(\kappa)$ and $\alpha'(\kappa)$, respectively, as follows:

$$\alpha(\kappa) = \alpha_p + \frac{\alpha_1\kappa}{B + \kappa} \tag{9a}$$

$$\alpha_1 = -(\alpha_p - \alpha_r) \tag{9b}$$

$$\alpha_p = \frac{2\sin\phi_p}{\sqrt{3(3 - \sin\phi_p)}} \tag{9c}$$

$$\alpha_r = \frac{2\sin\phi_r}{\sqrt{3(3 - \sin\phi_r)}} \tag{9d}$$

$$\alpha'(\kappa) = \alpha'_p \left(1 - \frac{\kappa}{C + \kappa}\right) \quad \textbf{(10a)}$$

$$\alpha'_p = \frac{\tan \psi_0}{\sqrt{9 + 12 \tan^2 \psi_0}} \quad \textbf{(10b)}$$

where, ϕ_p is peak friction angle, ϕ_r is residual friction angle, ψ_0 is angle of dilatancy at peak, and B & C are constants (Walters & Thomas).

The elastic moduli are estimated from modified equation proposed by Hardin & Black (1968) and are given the following equations in the case of clean sand:

$$G_{max} = G_0 \frac{(2.17 - e_0)^2}{(1 + e_0)} \left(\frac{\sigma_m}{P_a}\right)^{0}.5\ P_a\ (P_a = 98\ kPa) \quad \textbf{(11a)}$$

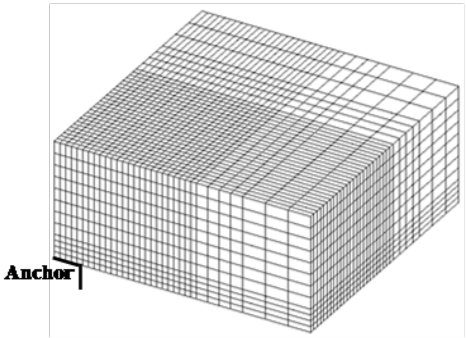
$$K = \frac{2(1 + \nu)}{3(1 - 2\nu)} G_{max} \quad \textbf{(11b)}$$

where, ν is Poisson's ratio, e_0 is the initial void ratio , G_0 is the initial-shear-modulus constant and P_a is atmospheric pressure.

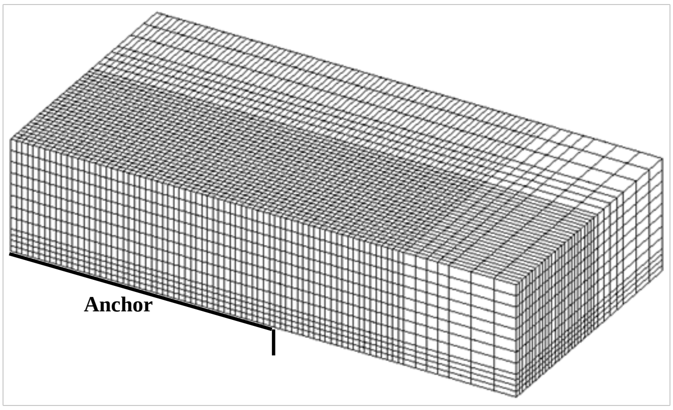
The calibration part for the numerical model discussed above was done in Tanaka & Sakai (1993), and the material parameters are: dry density, $\gamma_d = 16.5\ kN/m^3$, $G_0 = 150$, $\nu = 0.3$, $\phi_p = 45°$, $\phi_r = 35°$, $\psi_0 = 20°$, $B = C = 0.5$. In addition, 3 mm wide shear band is adopted, which equals to 20 times of mean diameter (D_{50}) of the Toyoura sand, Yoshida et al. (1993).

As the 3D analysis consists more elements, and, therefore, more nodes and integration points, more computer resources and time is required, compared to 2D analyses. Taking the benefits of two vertical planes of symmetry acting through the rectangular anchor foundation's center, one quarter of the domain is discretised into finite elements, some meshes are shown in the **Fig. 2**. Following the recommendation of Bray (1990), the mesh extends, sufficiently, 2*H* from the edges of the anchor, to ignore the boundary effects, and *H* above the anchor. The discretisation, using first order hexahedral elements, is finer in the central zone (zone above the anchor and extending to *H* from the anchor edges) than those at the edges, where limited deformation was expected. Ten 3D models with H/B=2, B=50mm, and L/B=1, 2, 3, 4, 5, 6, 7, 8, 10, and 20, and one 2D plane strain model (L/B=0) were created to investigate the shape effect on the uplift resistance and displacement. For the longer anchors, more elements were used in the longitudinal direction to maintain uniform element size across the all models. For example, the square anchor with L/B=1 (**Fig. 2a**) consists of 12288 elements, and the rectangular anchor with L/B=10 (**Fig. 2b**) consists of 48768 elements, which is, approximately, four times as many as the square anchor. Zero-displacement boundary conditions were applied to prevent the out of plane displacements of the central vertical symmetrical boundaries, and the base of the mesh were fixed in all three coordinate directions, except the anchor plate area. The differential quasi-elastic displacements were applied along the anchor boundary in small consecutive increments till to the failure, and the corresponding nodal forces were averaged over the displacement control nodes to find the uplift load.

To analyze the plane strain case of rectangular anchor problems (which is analogous to trapdoor problem), the same density of mesh as that on the face of the symmetrical plane of the three-dimensional mesh was employed. To construct the mesh, first order quadrilateral elements were used, and the equivalent boundary conditions, soil conditions, and analysis procedure were maintained.



(a)



(b)

Figure 2: Finite element meshes: (a) Square anchor (*L/B*=1) & (b) Rectangular anchor (*L/B*=10)

4. RESULTS AND DISCUSSIONS

Following the scaling rules of Lindsay (1980), the displacement (δ) is normalized as displacement factor, δ/B, and the uplift load (P) as pullout resistance factor, N_p ($=P/\gamma_d HBL$). The curves in the **Fig. 3** depict the relationships between the pullout resistance and displacement factor of the square and rectangular anchor foundations with *L/B*=1, 2, 3, 4, 5, 6, 7 and 8 (*B*=50 mm & *H/B*=2). The all curves show three distinct phases: the initial phase with sharp increase in pullout resistance with the anchor displacement, followed by a softening nature of decreasing the pullout resistance with displacement, and, finally, the pullout resistance remains unchanged with the further uplifting of the anchor, defining the failure phase. The average slope of the initial phase of the curves show that it is stiffer in the anchors with lower *L/B* ratio. The rate of softening of after the peak load is higher in the anchors with lower *L/B*. The residual loads are more or less similar irrespective of *L/B*. The characteristic progressive failure mechanism for the anchor with *L/B*=1 is shown in **Fig. 4**.

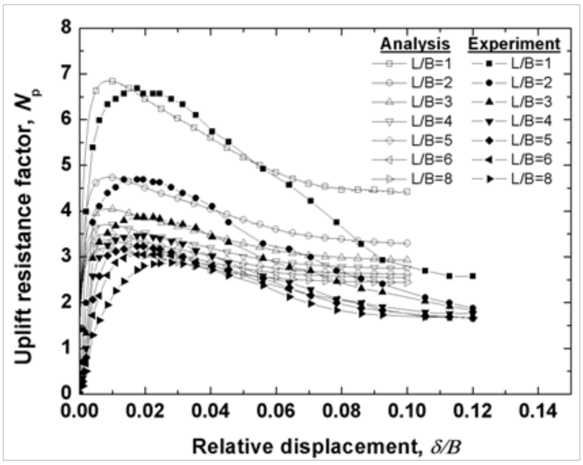
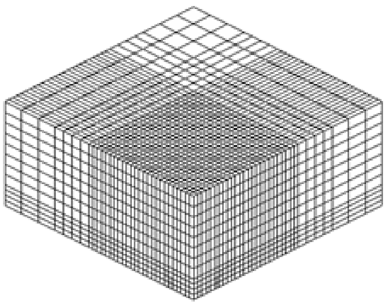
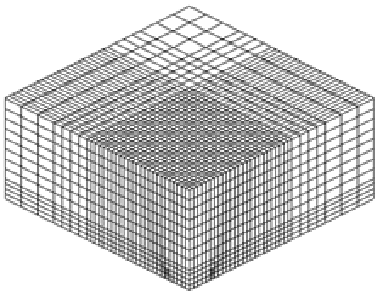


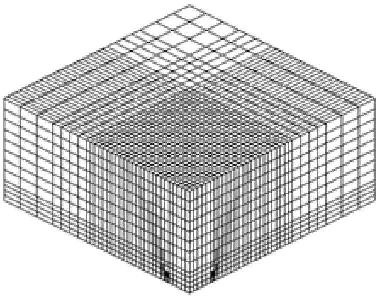
Figure 3: Load-displacement relationship.



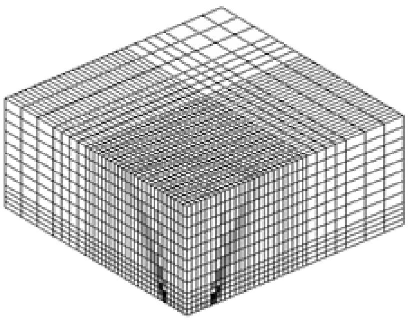
(a) δ/B=0.004



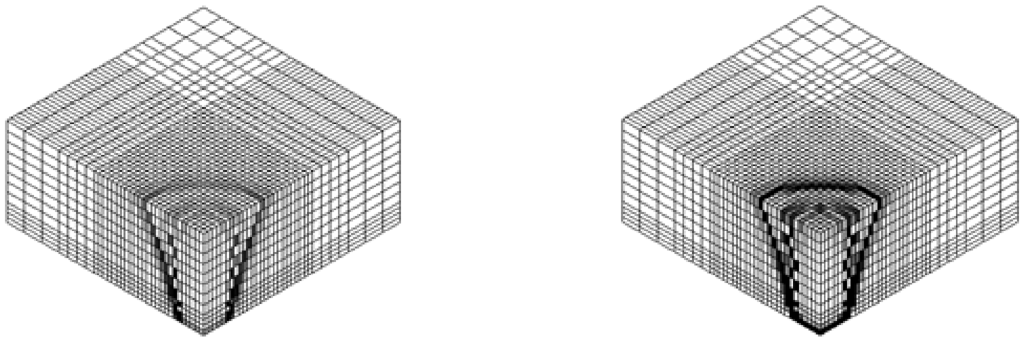
(b) δ/B =0.008



(c) δ/B =0.012



(d) δ/B =0.016



(e) δ/B =0.032 (f) δ/B =0.08

Figure 4: Characteristic progressive failure mechanism for the anchor with L/B=1 (the darkest region indicates the maximum shear strain greater than 10%)

The very important design factor, peak resistance factor (N_{pu}), found by analyses are quite similar as those in experiments, plotted in **Fig. 5**. The peak resistance factor decreases with the increase in *L/B* ratio, and, after *L/B*=10, it becomes nearly constant or gradually decreasing. That means, the failure mechanism is changing from 3D to 2D plane strain ($L/B=\infty$), and it is well known that the confinement in plane strain case is more than that in triaxial case. For the lower value of *L/B* or square anchor, the mechanism is far away from plane strain, showing the noticeable larger value of peak resistance factor than that of strip anchor. It proofs the necessity of 3D design considerations for such anchor foundations. Even for *L/B*=20, the peak resistance factor is less than that of plane strain anchor. It is to note here that, during the derivation of the previously proposed empirical shape factors, the anchor was assumed to be strip when *L/B*>8 (Dickin, Ovesen & Stromann), *L/B*>6.7 (Frydman & Shamam) or *L/B*>10 (Murrary & Geddes). So, they all underestimated the shape factors.

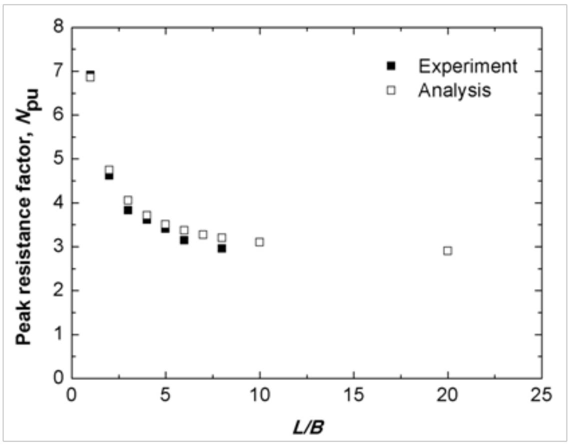


Figure 5: Peak resistance factor as a function of L/B ratio.

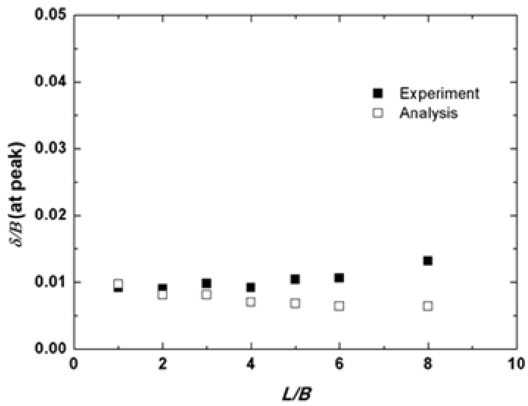


Figure 6: Relative displacement at peak as a function of L/B ratio.

The displacement factors at the peak loads are plotted as a function of *L/B* ratio in the **Fig. 6**. It shows that the displacement at the peak is, gradually, decreasing with the increase of *L/B* ratio in the experiments, but, which is somewhat opposite in the analyses. The displacement factors at the peak loads from the analyses are underestimated, and the difference increases with *L/B* ratio. The reason is may be due to the no hardening in the pre-peak region and simple softening law in the post peak region adopted in the constitutive model. In this regard, the simple constitutive model can be upgraded to a rigorous one, introducing the strain hardening-softening model like Sakai & Tanaka (2007) for the more accurate predictions.

The failure of the shallow anchors, considered in this study, are associated with the formation of shear bands around the edges of the anchor and almost a rigid upward movement a block of soil directly above the anchor, which causes a heave of the soil on the ground surface. A characteristic photograph of the ground surface at failure is shown in the F**ig. 7**. **Fig. 8** and **Fig. 9** shows the 3D and plan view of the failure mechanisms, respectively, with the distribution of the maximum shear strain (the darkest region indicates the shear strain larger than 10%). These figures show that the four corners of the boundary of the heaved ground surface is a circular arc, the result is almost matching the observations in the experiments with the analyses. In addition, the direction of the outermost shear zone development inside the ground is almost 65° with the horizontal.

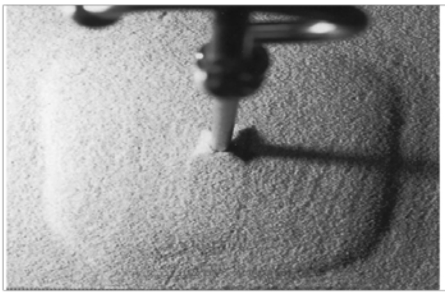
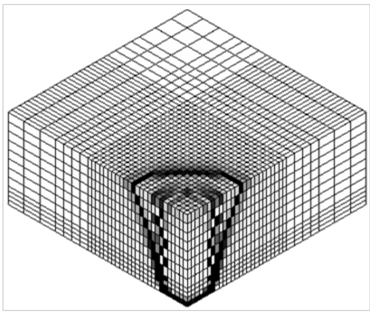
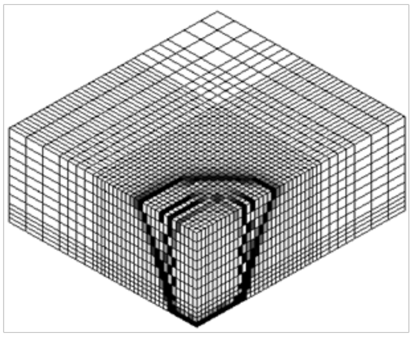


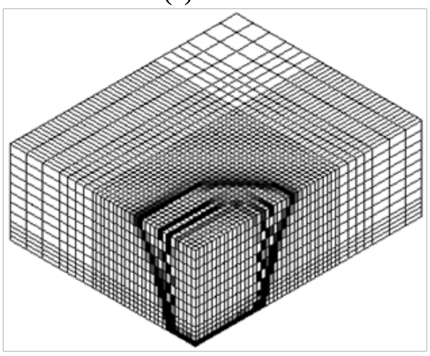
Figure 7: Characteristic photograph of failure mechanism on ground surface



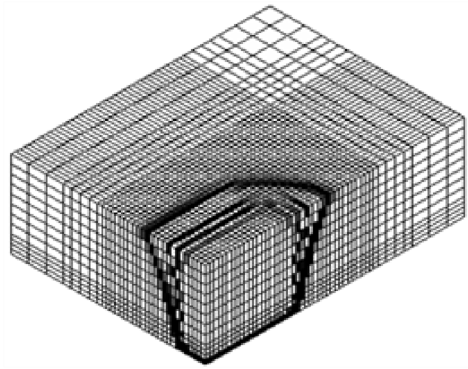
(a) *L/B*=1



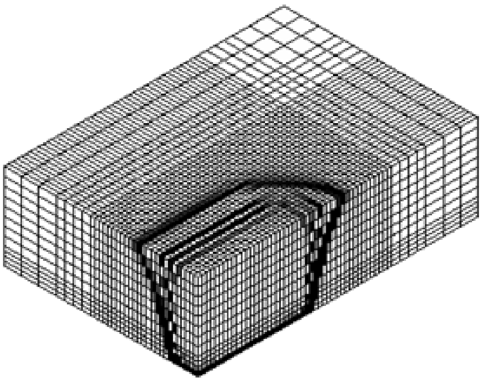
(b) *L/B*=2

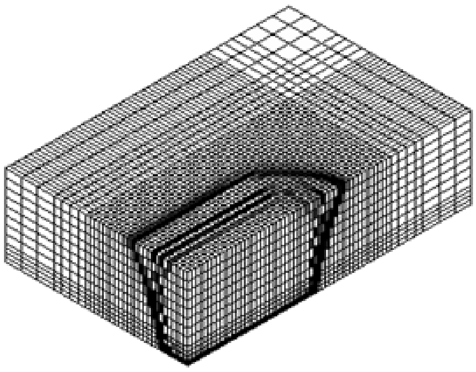


(c) *L/B*=3

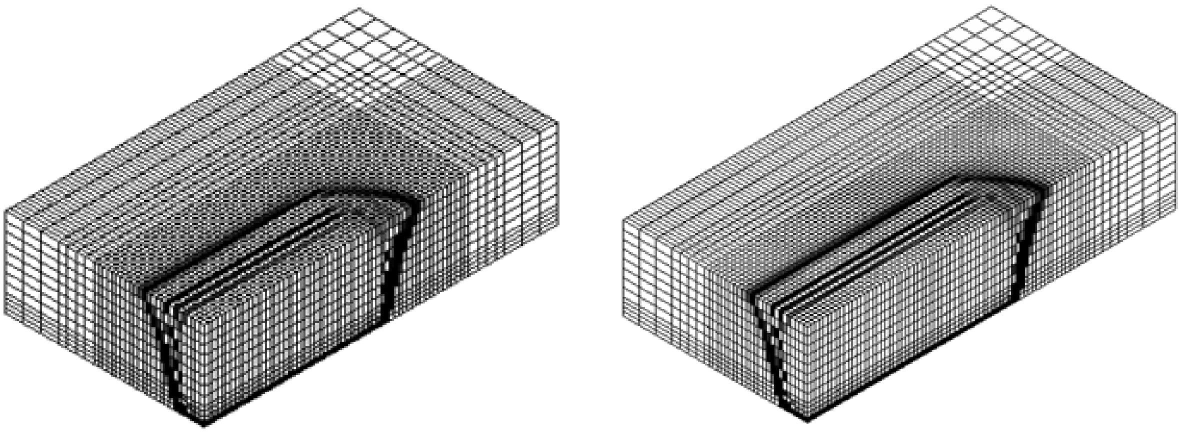


(d) *L/B*=4



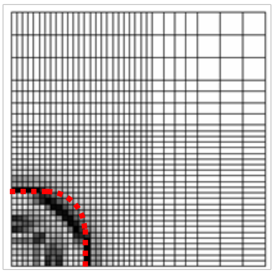


(e) *L*/*B*=5 (f) *L*/*B*=6

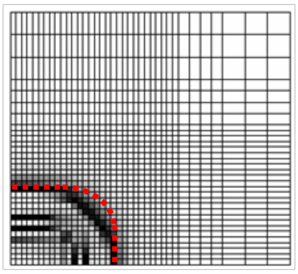


(g) *L*/*B*=8 (h) *L*/*B*=10

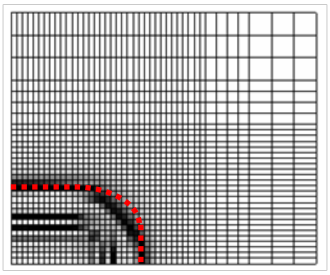
Figure 8: 3D view of the failure mechanisms (not to scale).



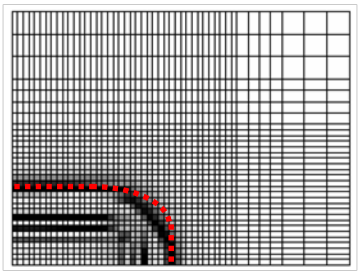
L/*B*=1



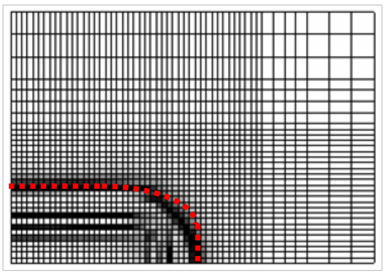
L/*B*=2



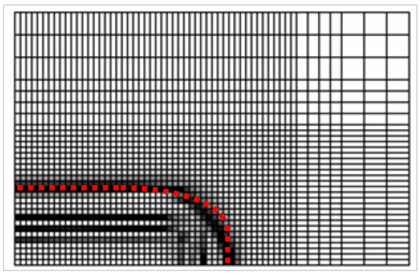
L/*B*=3



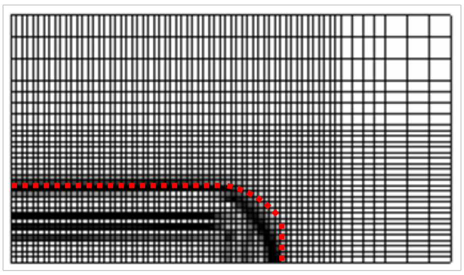
L/*B*=4



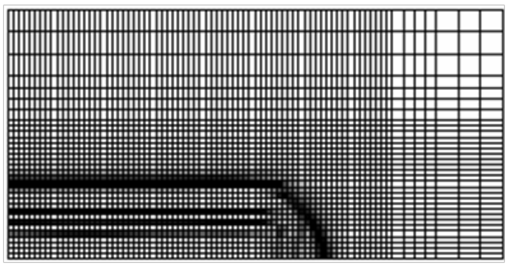
L/*B*=5



L/*B*=6



L/*B*=8



L/*B*=10

Figure 9: Plan view of the failure mechanisms (dotted lines are experimental boundary of outcropped failure surface)

5. CONCLUSIONS

In this research work, a numerical model is validated for the shape effect of the square and rectangular anchor foundations, comparing the numerical results with the experiments in terms of the normalized uplift resistance-displacement relationships, the peak uplift resistance factors, displacement factors at the peak load and the extensions of the failure mechanisms from the edges of the anchor foundation. From the results, the following conclusions are drawn:

(1) the numerical model satisfactorily predict the load-displacement relationship of the pullout experiments.

(2) the numerical model can accurately reproduce the peak loads and the peak load decreases with the increase in *L/B*. For the lower values of *L/B* or square anchor foundation, the peak resistance is noticeable larger than that of strip anchor.

(3) the numerical model has predicted the average inclination of the outermost failure surface as that in the experiment, accurately, which is about 65° with the horizontal.

(4) however, the simplicity of the proposed numerical model with the constitutive model: no-hardening before peak and simple strain softening law after peak consequences the underestimation of the displacement at the peak resistance, recommending more sophisticated hardening-softening law.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development
(ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

PREDICTION OF LOAD CARRYING CAPACITY OF RAMMED AGGREGATE PIER USING FEM

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ABSTRACT

Finite Element Method (FEM) can be a good technique to simulate the situation and hence, for necessary predictions. So the PLAXIS, a FEM software is used here as a numerical tool to predict the experimental results reported by Hossain. It is observed that the predicted result satisfactorily follows the observed values with reasonable degree of accuracy. Thus, this study proves that the FE model can be used with quite satisfactorily for the prediction of the behavior of RAPs reinforced soft ground.

Keywords: FEM, RAP, bearing pressure.

1. INTRODUCTION

In this paper, a developed calibrated numerical model is used to predict the bearing capacity of soft ground improved by Rammed Aggregate Piers (RAPs). The sub-soils in Khulna region mainly consists of soft fine-grained soil up to great depth with a layer of very compressible soil. So it is very essential to improve this soft soil to construct various types of civil infrastructures. In KUET campus, the RAPs in three different categories were constructed to improve the soil with locally fabricated equipments and construction materials. Load tests of full size isolated square footing were conducted on both the natural soil and improved ground to determine the load carrying capacity. The interaction between footings, RAPs and surrounding soil is a complex problem indeed. Finite Element Method (FEM) can be a good technique to simulate the situation and hence, for necessary predictions. So the PLAXIS, a FEM software is used here as a numerical tool to predict the experimental results reported by Hossain (2007). It is observed that the predicted result satisfactorily follows the observed values with reasonable degree of accuracy. Thus, this study proves that the FE model can be used with quite satisfactorily for the prediction of the behavior of RAPs reinforced soft ground.

2. MATERIALS AND METHOD

2.1 Rammed Aggregate Pier

Rammed Aggregate Pier (RAP) has become a common ground improvement technique for improving the marginal sites. RAP methods have used successfully other country for some ground improvement projects (Alamgir 1996). The performance of this technique is required to investigate further in details in local condition. The RAP system uses to reinforce reasonably good to poor soils, including soft to stiff clay and silt, loose to dense, organic silt, peat, variable and uncontrolled fill. RAP soil reinforcement is used for the support of transportation structures including Mechanically Stabilized Earth retaining walls and large embankment fills. The installation of stiff RAP elements provides a significant increase in the composite stiffness of otherwise soft

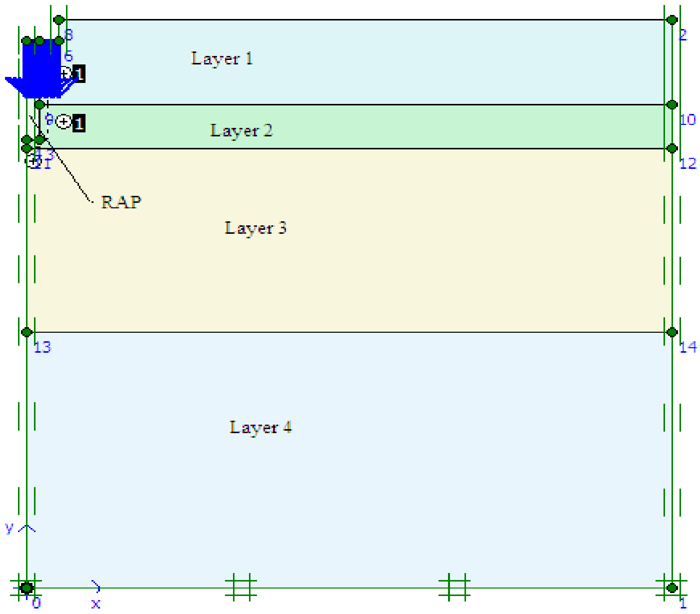

Fig. 1: Geometry

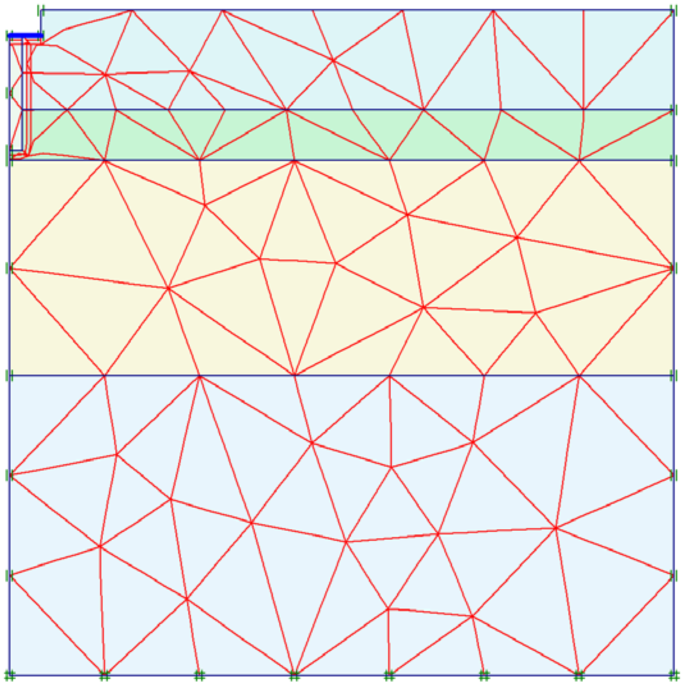

Fig. 2: FEM mesh

and compressible foundation soils. RAP Construction using open-graded stone affords radial drainage to the elements. The result of RAP installation is a significant decrease in both settlement magnitude and duration with the RAP-reinforced zone. The RAPs were installed successfully at KUET campus to investigate the improvement of a poor soil conditions having a significant layer of organic soils (Hossain 2007 and Hossain & Alamgir 2010). The geometry (PLAXIS model), FEM mesh are shown in Figs. 1 and 2.

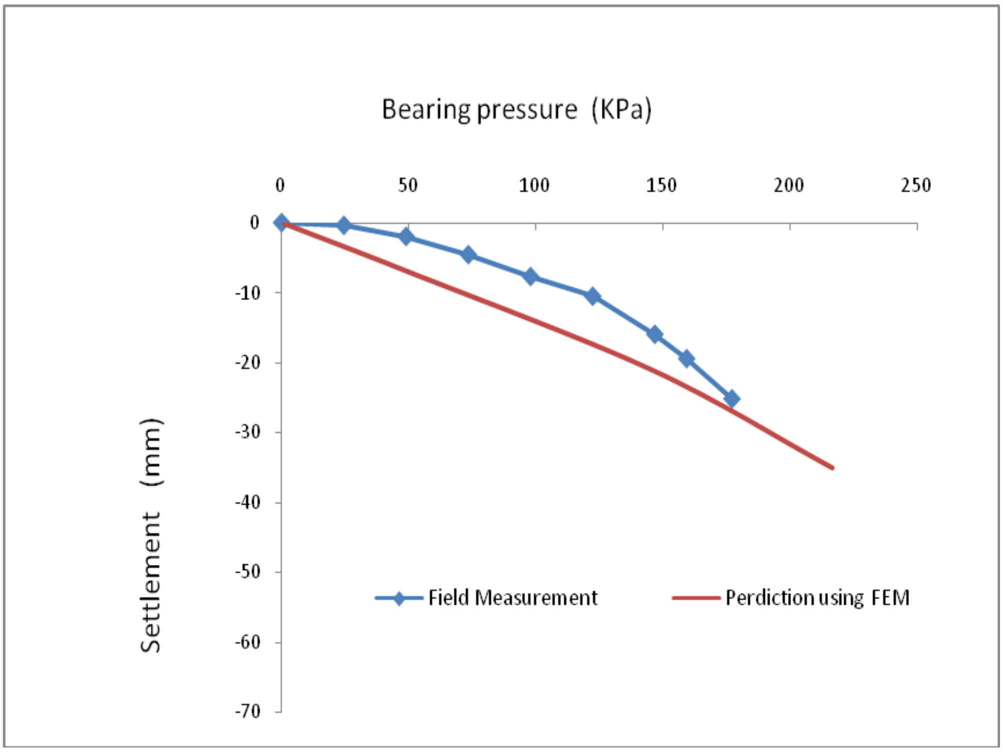

Fig. 3: a) Load-Settlement Curve

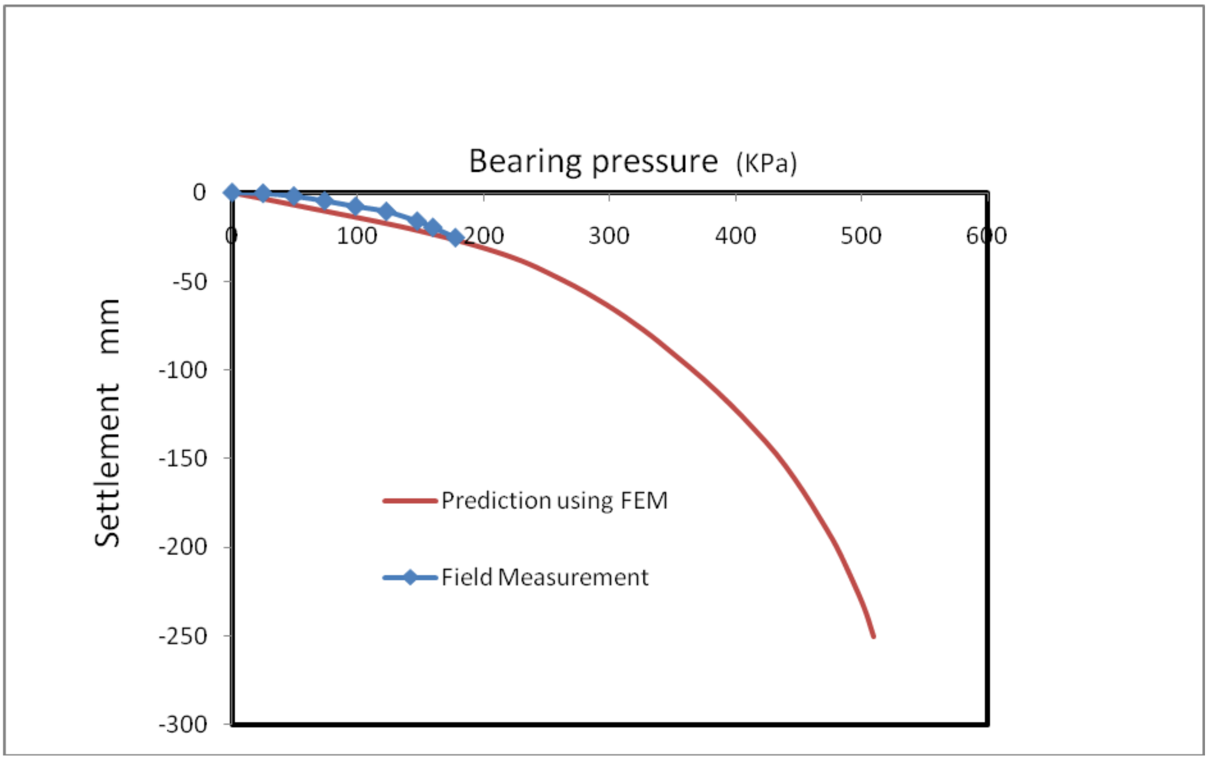

Fig. 3: b) Load-Settlement Curve

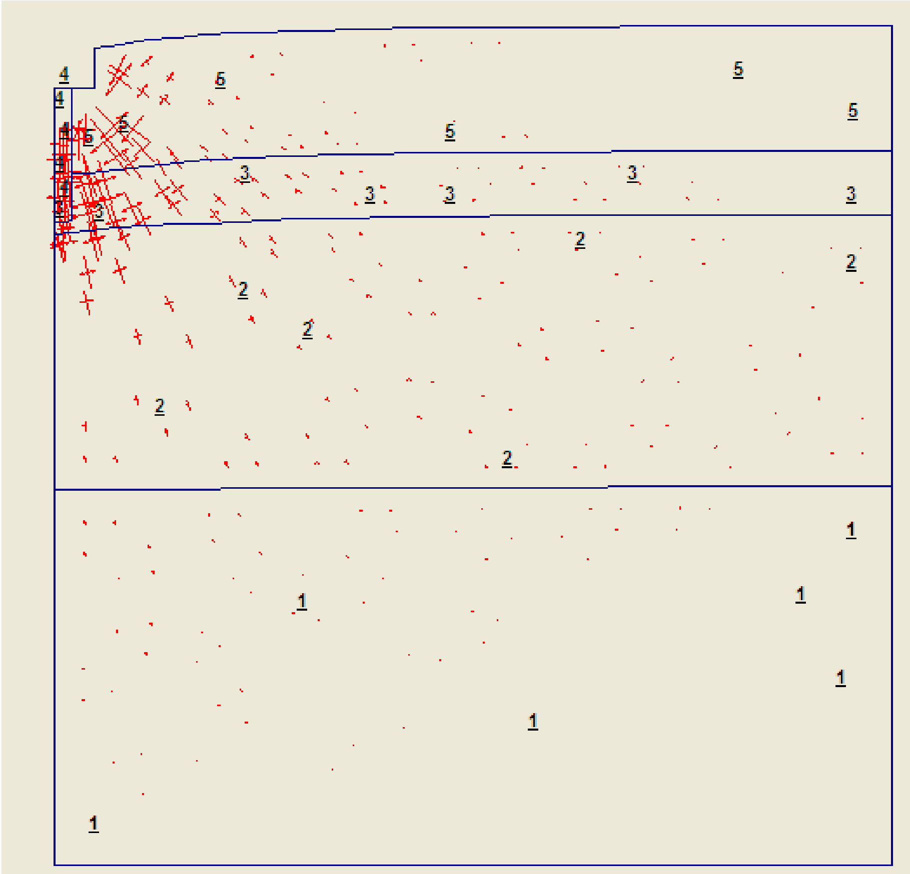

Fig. 4: Total Stress

2.2 Numerical model

PLAXIS Version 8 is a finite element package intended for the two-dimensional analysis of deformation and stability in geotechnical engineering. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time dependent and anisotropic behaviour of soils and/or rock. In addition, since soil is a multi-phase material, special procedures are required to deal with hydrostatic and nonhydrostatic pore pressures in the soil. Although the modelling of the soil itself is an important issue, many tunnel projects involve the modelling of structures and the interaction between the structures and the soil. Load-Settlement curve are shown in Figs. 3.a) and 3.b).

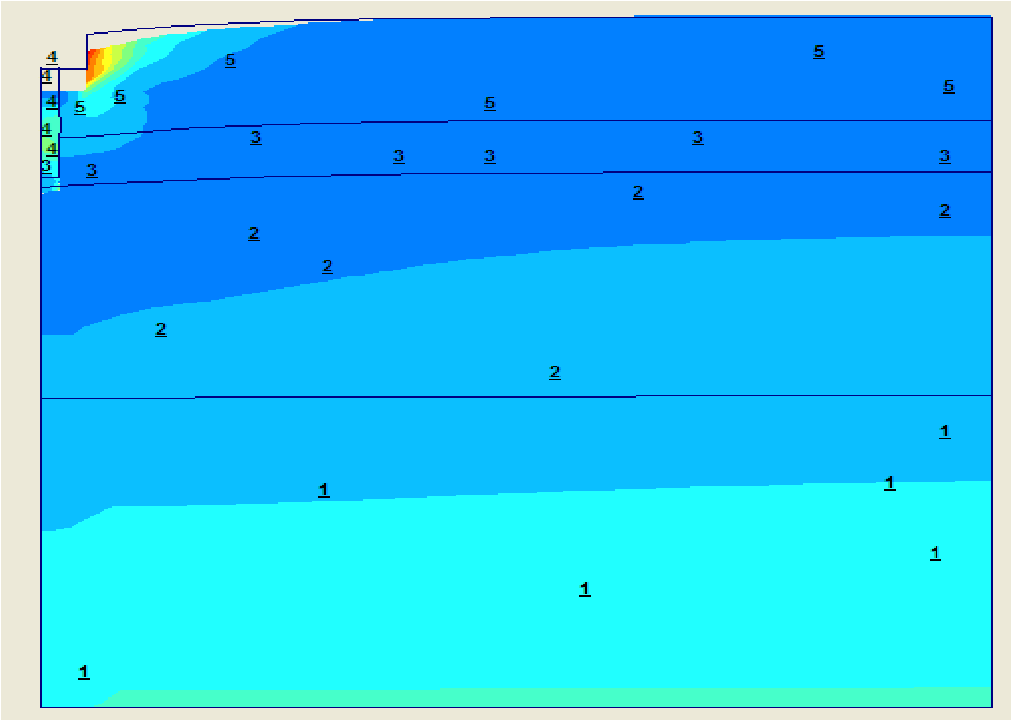

Fig. 5: Vertical Effect stress

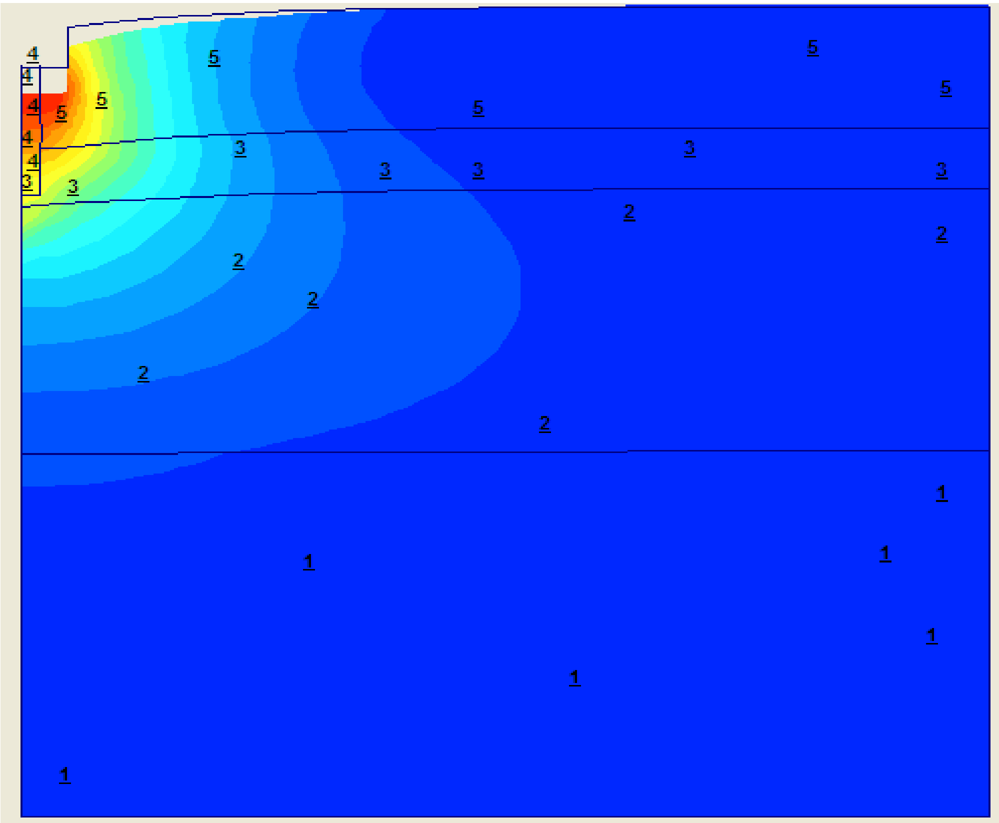

Fig. 6: Total Displacement

PLAXIS is equipped with the features to deal with various aspects of complex geotechnical structures. Mohr-Coulomb failure criteria is robust and simple. It is based on soil parameters that are well-known in engineering practice. Not all non-linear features of soil behaviour (hardaning and softening) are included in this model, however. The Mohr-Coulomb model may be used to compute realistic support pressures for tunnel faces, ultimate loads for footings, etc. It may also be used to calculate a safety factor using a 'φ-c reduction' approach. In this study, an elastro-perfectly plastic FE medel with Mohr-Coulomb failure criteria has been used to predict the bearing capacity of the soft soil improved by RAPs.

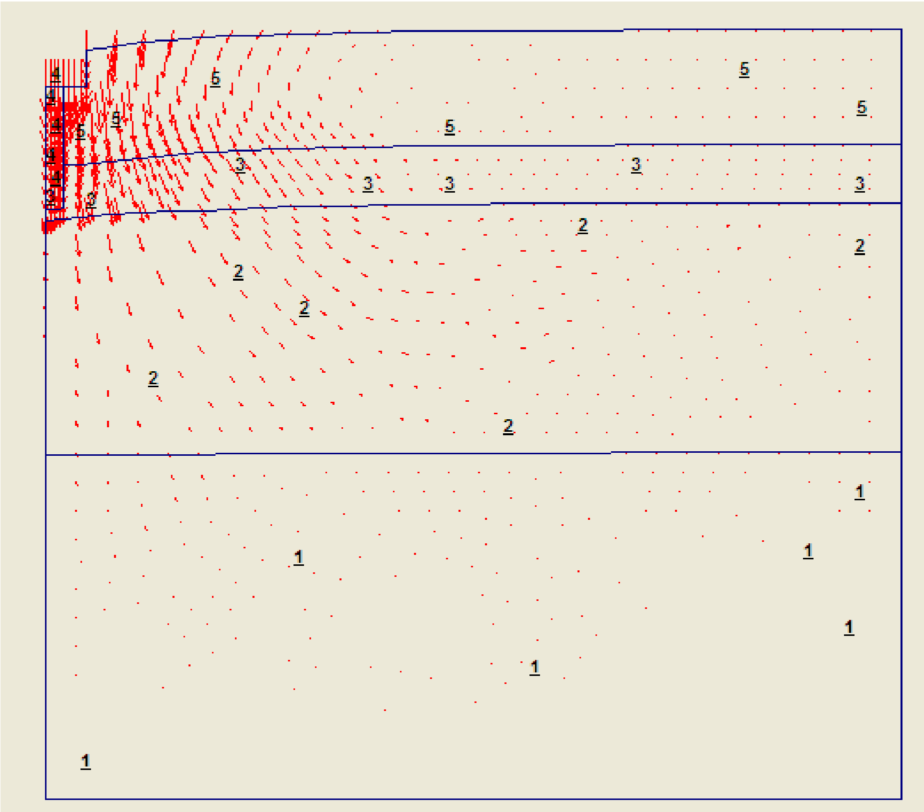

Fig. 7: Total Displacement

Table 1: Material parameters used in PLAXIS

Layer	Soil type	Ko	λ_{unsat} KN/m^3	λ_{sat} KN/m^3	E KN/m^2	ν	C KN/m^2	Φ degree	ψ degree
Layer 1	Silty Sand	1.00	17.00	20.00	10,000	0.30	15.00	30	10
Layer 2	Clay	1.00	16.90	18.90	2,000	0.30	30.00	0	0
Layer 3	Organic clay	1.00	12.63	15.00	2,000	0.30	31.00	0	0
Layer 4	Silty clay	1.00	16.69	18.69	5,000	0.30	13.00	0	0
Sand Column	RAP	1.00	17.00	20.00	20,000	0.30	10.00	45	15

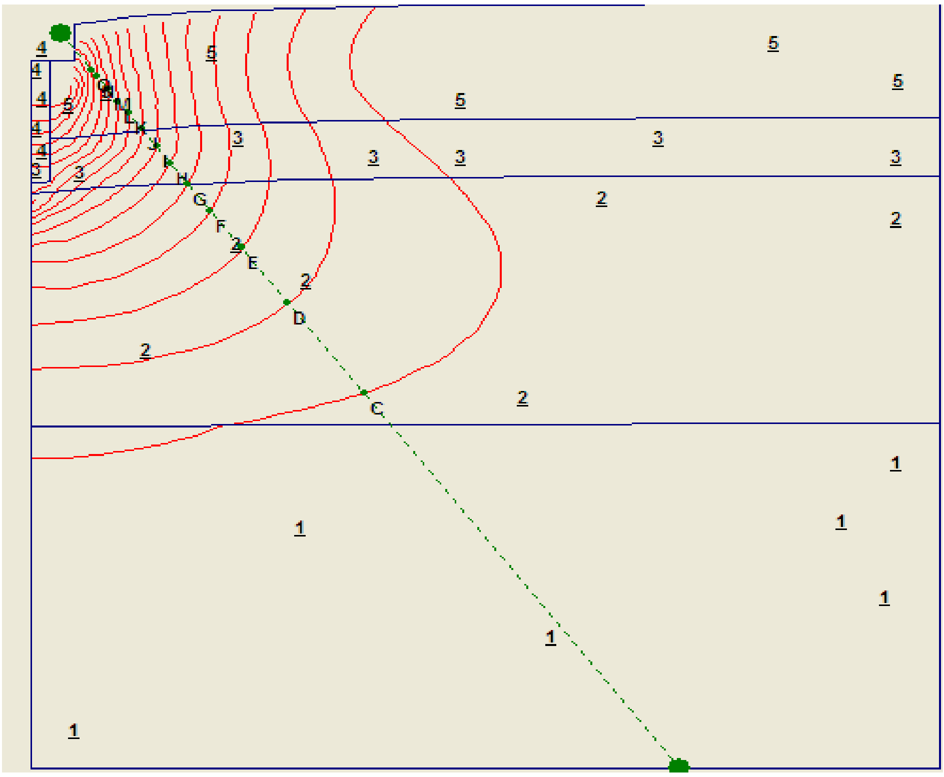

Fig. 8: Total Displacement (contour)

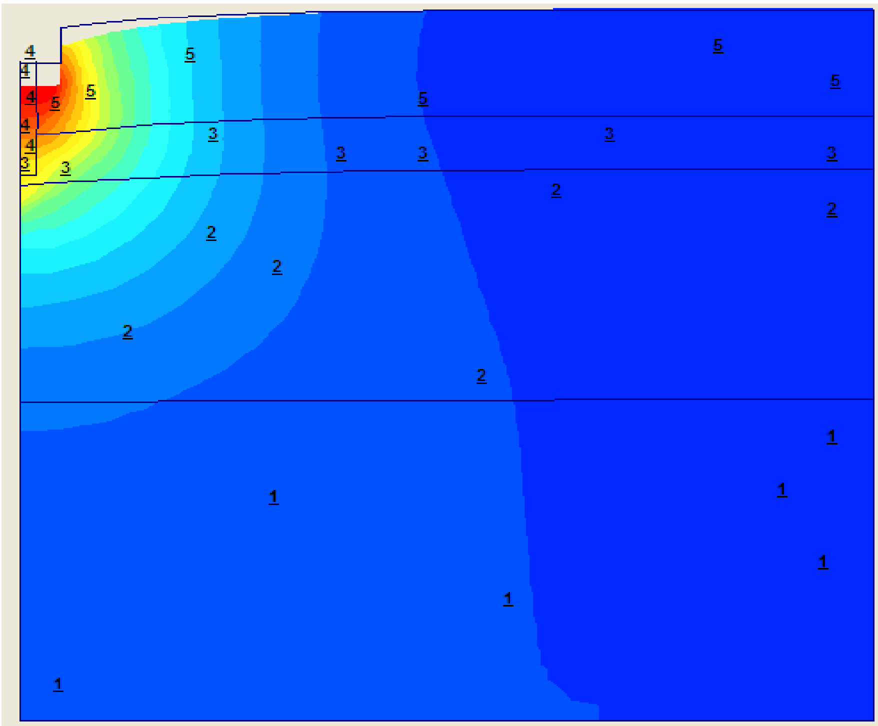

Fig. 9: Vertical displacement

3. RESULT AND DISCUSSION

The required material parameters of the soil layers and the RAP are presented in Table 1.. The load carrying capacity of the RAP as obtained in the field as well as numarical prediction shown in Fig. 4 to 13. It is observed that PLAXIS can be used with reasonable degree of accuracy to predic the capacity of RAP installed in a soft ground with layering property.

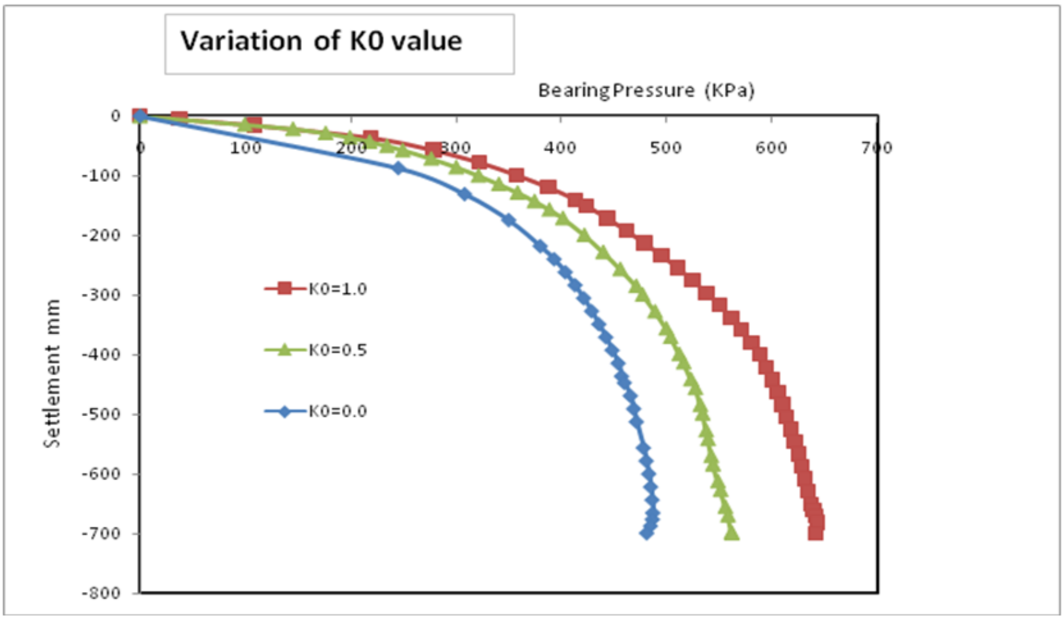

Fig. 10: Parametric study effect of initial earth pressure

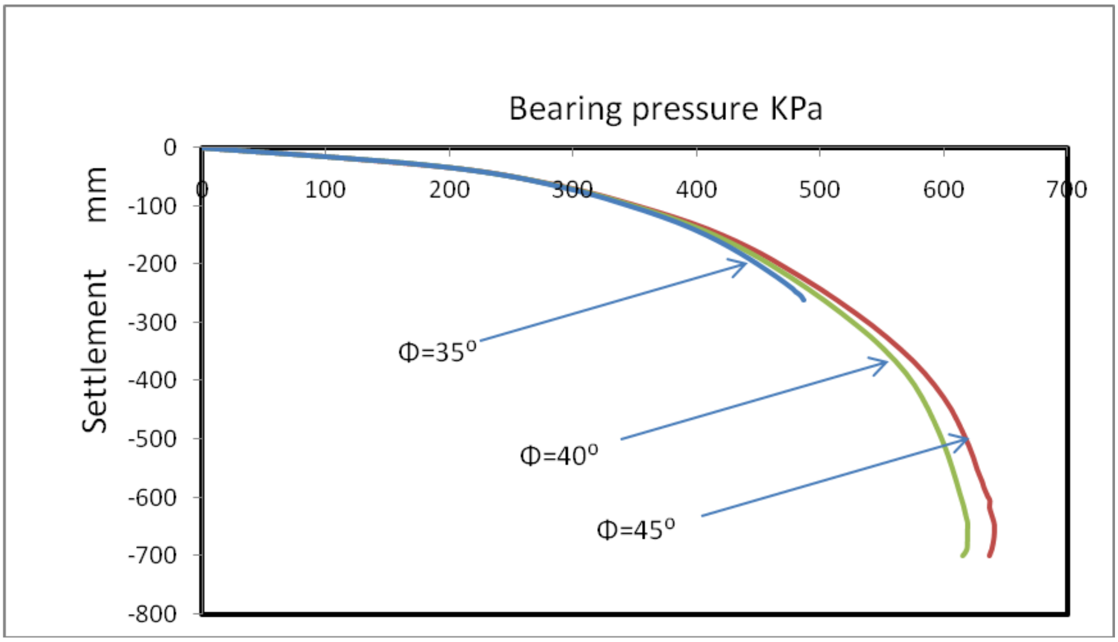

Fig. 11: Effect of the variation φ value of RAP

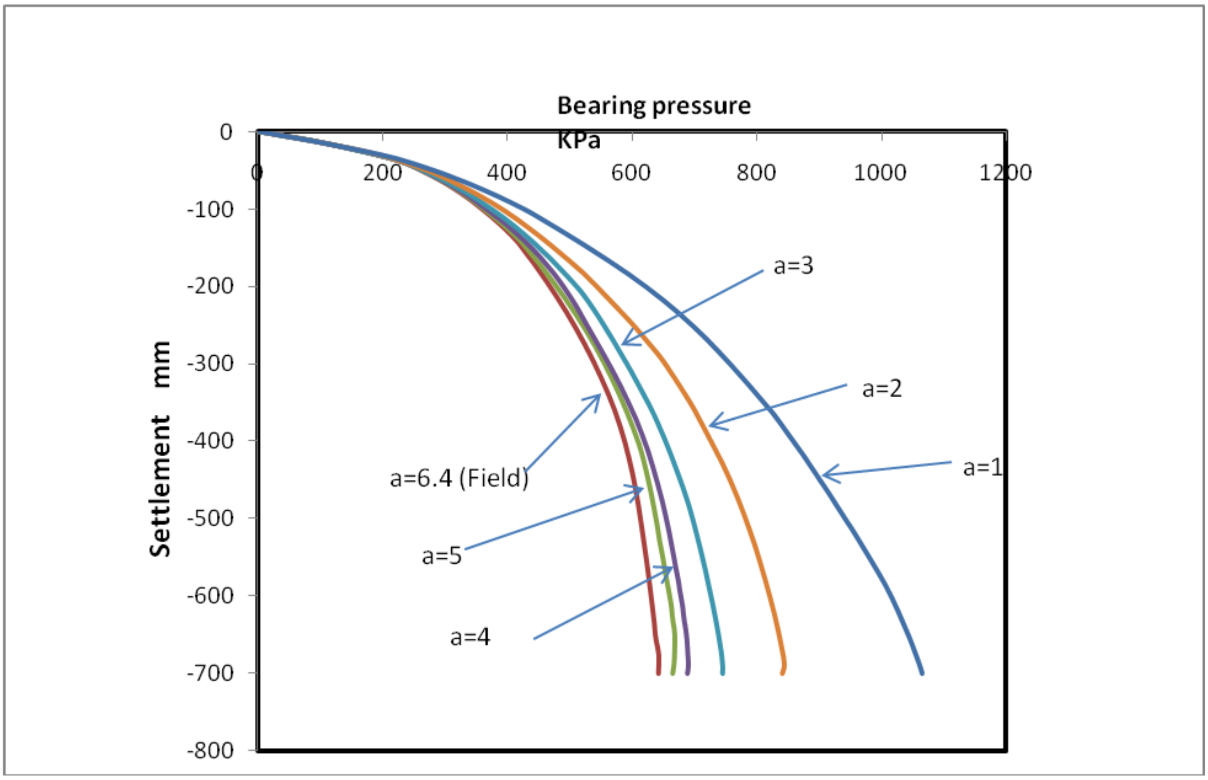

Fig. 12: RAP and test plate area ratio

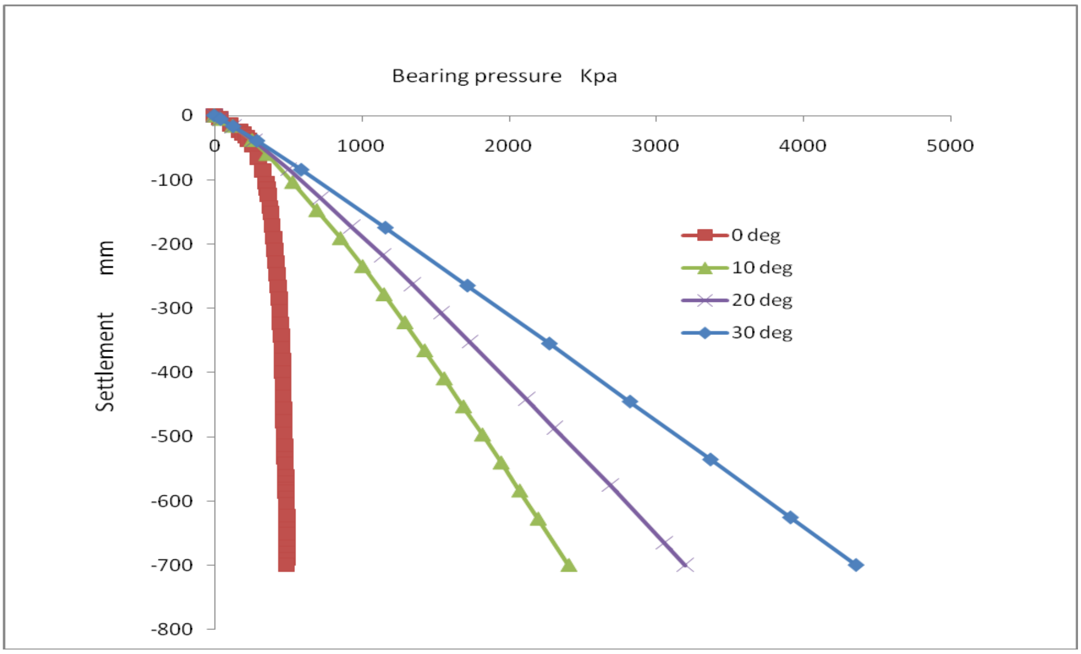

Fig. 13: Dilatiation angle of soil ψ

Using this valideted FE model, an extensive parametric study has been completed to find the effect of 1) Area ratio, 2) properties interface, 3) shear strenght parameters of surrounding soils, 4) rigidity of foundation, 5) shear strength properties of RAP and 6) dilatiation angle of soil and RAP material on the bearing capacity.

4. CONCLUSIONS

Based on this study the following conclusions can be made:

(i) RAP is a proven technique to improve poor soil conditions.

(ii) RAP improved ground can be analyzed and hence predicted with reasonable degree of accuracy using the Finite Element method.

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THE EFFECT OF PRE-CONSOLIDATION PRESSURE AND ORGANIC CONTENT ON THE SHEAR STRENGTH AND COMPRESSIBILITY PARAMETERS OF RECONSTITUTED SOIL

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ABSTRACT

This study illustrates the findings of an experimental investigations into the effect of pre-consolidation pressure and organic content on the shear strength and compressibility parameters of reconstituted soil prepared in the laboratory. To thses attempts, The disturbed soil sample was collected from two selected locations at Khulna region whose organic contents of 8 and 65%.. The reconstituted soil samples were prepared to mix at various proportions of inorganic and organic soil at the water content equal to 1.25 times of liquid limits thus resulting in reconstituted soil having organic content of 30, 40 and 50%. The usual procedure of preparation of soil slurry, deposition in a mold and application of surcharge were used to reconstitute sample. The mold diameter was 152mm and height 222mm and applied ultimate surcharge was about 60, 80 and 100 untill the end of primary consolidation. In the laboratory, ASTM (2004) standard test were performed on the reconstituted samples to evaluate the strength and compressibility properties. Here, it can be depicted that pre-consolidation pressure and organic content significantly influence the shear strength and compressibility parameters of reconstituted soil.

Keywords: *Reconstituted soil, organic content, pre-consolidation pressure, strength and compressibility, khulna city.*

1. INTRODUCTION

Khulna is situated in the south-western part of country near the Sundarbans, the largest mangrove forest of the world. The sub-soil of this region consists of fine grained soils with a considerable part of decomposed and semi-decomposed organic matter (Rafizul et al. 2006). In Khulna region, the soft soil deposit extends up to a considerable depth, as a result the recent alluvial deposits with organic composition creates problem to Geotechnical Engineers in designing economic foundation to construct the required infrastructure (Alamgir et al., 2001). Generally the organic soil depositions in Khulna region exist in most of the places in between the depth of 10 to 25 ft below the existing ground surface. Moreover, the soil is also erratic in nature both in vertical and horizontal directions (Rafizul et al. 2009). Fully decomposed organic soil has very low bearing capacity, extensive amount of settlement of structure and always leads to adopt a costly foundation for the construction of infrastructures. Sometimes the valuable structures are collapsed due to excessive total and differential settlement while constructed in Khulna region without proper foundation. The behavior of organic soil subjected to maximum overburden pressure in the past can be obtained at varying pre-consolidation pressure which has considerable effect on engineering parameters (Rafizul et al. 2007). So, it is obligatory to perform a comprehensive study on the reconstituted soil, prepared in the laboratory based on the procedure followed by Burland (1990) to obtain the wide range of organic content under different pre-consolidation pressure. Moreover, develop some correlations among the strength and compressibility properties in relation to the variation of pre-consolidation pressure and organic content of reconstituted soil. However, to depict the validity of the developed model then compared with the results available in the literature for the same cases.

2. LABORATORY INVESTIGATIONS

In this study, disturbed soil samples were collected from two selected locations of Khulna region, one from Beel Dakatia, 2km away from KUET campus, Khulna at a depth of about 10 feet and another from KUET campus at a depth of about 5 feet from the existing ground surface. The detailed methodology followed for this laboratory investigation can be obtained in Rafizul et al. (2007) and presented in Figure 1. In the laboratory, the physical and index properties of soil samples used to prepare reconstituted soil were determined by ASTM (2004) methods and presented in Table 1.

Table1: Physical and Index properties of soil samples to prepare reconstituted soil

Property name	Locations	
	Beel Dakatia	KUET campus
Organic content ,OC (%)	65	8
Moisture content, w (%)	240	39
Liquid limit, w_L (%)	81	49
Plastic limit, w_p (%)	64	33
Compression index, C_c	1.24	0.386
Initial void ratio, e_o	1.80	0.88
Unconfined compressive strength, q_u (kPa)	19	66
Bulk unit weight, γ (kN/m^3)	9	17
Speific gravity, G_s	1.63	2.57
Soil particle (4.75 -0.076 mm)	65	8
Soil particle (0.076- 0.002 mm)	27	36
Soil particle (<0.002 mm)	8	56

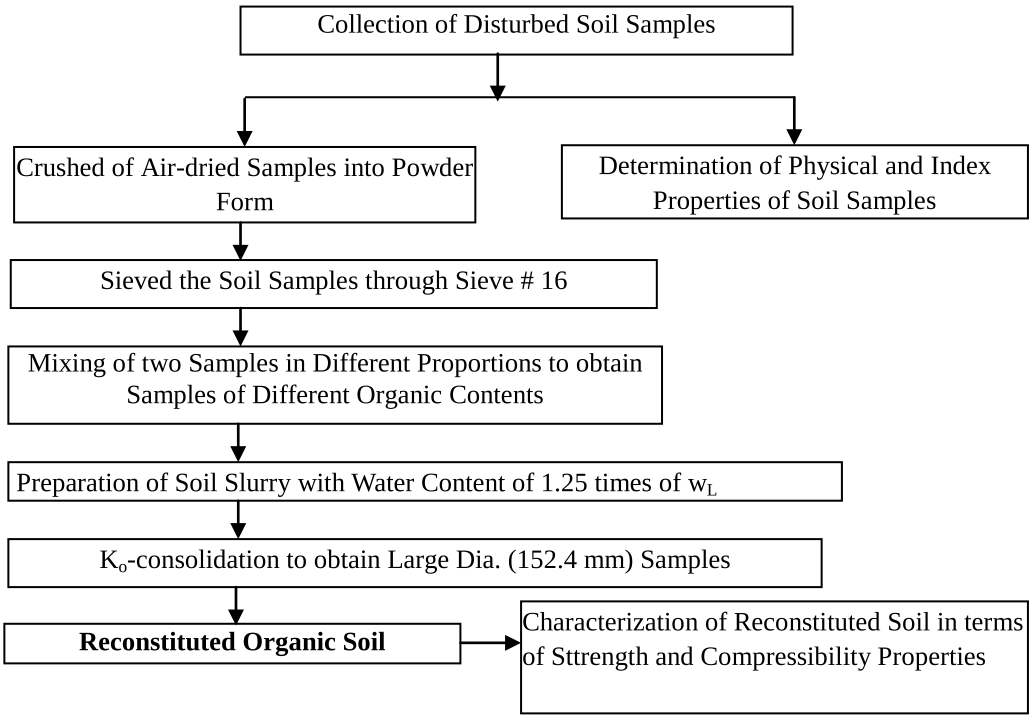

Figure 1: Flow chart of laboratory investigation

2.1 Preparation of Soil Slurry

In this study, the samples were first air-dried and then powdered. The powered samples were then sieved through No 16 sieve and the samples were then mixed with a water content equal to 1.25 times of liquid limits which was found as sufficient to yield uniform and homogeneous slurry. Generally No. 40 sieve is used for making reconstituted inorganic soils by various researchers, such as Bashar (2002), however, in this study a standard sieve of larger opening size i.e. No. 16 sieve was used to obtain a better representative reconstituted organic soils. The Detailed of soil slurry and test specimen preparation can be obtained in Rafizul et al. (2007).

2.2 Consolidation of Slurry

The slurry was consolidated to form a uniform reconstituted soil cake in a cylindrical consolidation mold of 152mm diameter and 222mm height. Such types of slurry were kept in three molds to consolidate it under the consolidation pressure of 60, 80 and 100 kN/m^2. This pressure of 60, 80 and 100kN/m^2 was maintained until the end of primary consolidation. The required axial load of 60, 80 and 100kN/m^2 was gradually applied to the sample using a loading frame initially the slurry was allowed to consolidate by the self-weight and the weight of the porous metal discs for about 24 hours. Then a small pressure of 3kN/m^2 was applied to the sample for the next 24 hours. After the completion of consolidation, a soil cake of about 114 to 127mm length and 152mm diameter was obtained from the reconstitute soils.

2.3 Selection of Overburden Pressure

Earlier the minimum pre-consolidation pressure of 276 kN/m^2 that make the clay soil just stiff enough & latter as the skill in testing it was found of 150 kN/m^2. (kirkpatrick and khan 1984). Singh (1992) suggested that soil containing high organic matter shows large volume changes on loading & expulsion of water, low shear strength & low dry density. In addition, the reconstituted organic soil is fully decomposed with normally loaded state and shows highly compressible phenomena. So, in this study, the reconstituted soils were prepared in k_o-consolidation cell by a consolidation pressures, σ_c of 60, 80 and 100 kN/m^2.

3. RESULTS AND DISCUSSIONS

This article deals with the presentation and discussions on the observed shear strength and compressibility properties at varying applied pre-consolidation pressure (σ_c) and organic content (OC) through a series of laboratory tests. Based on the changing pattern of developed equations, the effect of σ_c and OC on the engineering properties of reconstituted soil are highlighted and discussed in followings.

3.1 The Effect of Pre-consolidation Pressure on the Mixing Organic Content in Soil Slurry

In this study for laboratory investigation, it has been intended to prepare reconstituted soil at the organic contents of about 8, 30, 40, 50 and 65% by mixing the soil samples collected from KUET campus and Beel Dakatia. However, the soil samples of organic content of 8% and 65% were obtained as the samples from KUET campus and Beel Dakatia, respectively, without any mixing. Although slurry was prepared to achieve such organic contents, but after applying σ_C of 60, 80 and 100 kPa and burning it in the laboratory for 5.5 hours at temperature 550^{0}C, the measured organic contents were rounded as 5, 6, 7, 11, 13, 17, 20, 21, 23, 28, 32, 36, 39, 40 and 45% against to that of predetermined organic contents as stated earlier. The reconstituted organic soil at each designed organic content were prepared in the laboratory based on the procedure followed by Burland (1990). The Figure 2 represents the variation of measured organic content in the reconstituted samples with mixing organic content in soil slurry at varying σ_C.

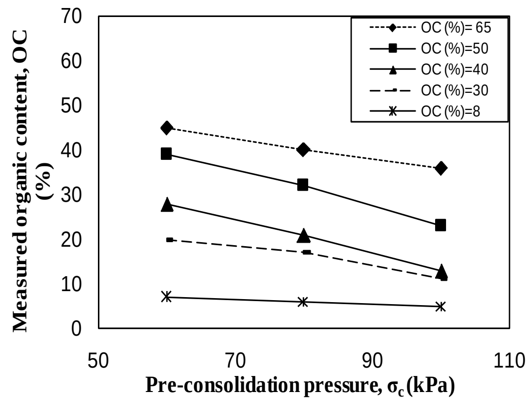

Figure 2: Effect of pre-consolidation pressure on the mixing organic content in soil slurry

3.2 Effect of Pre-consolidation on the Shear Strength Parameter of Reconstituted Soil

The effect of σ_C ranging from 60 to 100 kPa on the shear strength parameters in terms of undrained shear strength (s_u), cohesion (c) and angle of internal friction (Φ) on reconstituted soil are presented in Table 2 and hence discussed here. The variation of s_u with the variation of σ_C is evident in Figure 3. This figure reveals that s_u increases with the increasing of σ_c from 60 to 100 kPa of reconstituted soil. Moreover, the s_u is a function of organic content for a natural soil and soil peat mixtures and also the interconnected bonding of soil particles i.e. the cementing bonding (Franklin et al. 1973). In organic soils, the particles interconnected contracts have established within organic matters. As the organic matter bears low strength and high compressibility, the strength properties of soil reduced with increase of organic content. In

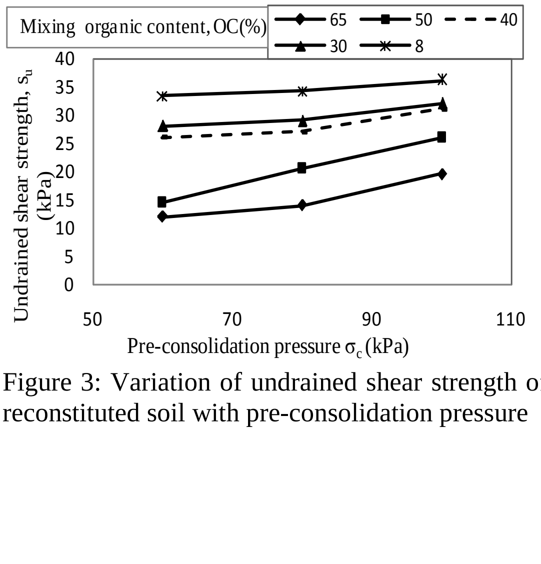

Figure 3: Variation of undrained shear strength of reconstituted soil with pre-consolidation pressure

contrast, based on Figure 4 it is clear that the cohesion has increased significantly with the increase of pre-consolidation pressure from 60 to 100 kPa of reconstituted soils when mixing organic content is constant. Moreover, the angle of internal friction has increased with the increase of pre-consolidation pressure from 60 to 100 kPa of the reconstituted soils shown in Figure 5.

Table 2: Shear strength parameter and pre-consolidation pressure in the reconstituted soil

Mixing organic content in slurry, OC (%)	Pre-consolidation pressure , σ_c(kPa)	Shear strength parameter		
		Undrained shear strength, s_u (kPa)	Cohesion, c (kPa)	Angle of internal friction, Φ (°)
65	100	19.5	18.5	22
	80	14	16	16
	60	12	15	15
50	100	26	20	25
	80	20.5	19	24
	60	14.5	18	21
40	100	31	22	28
	80	27	19	25
	60	26	20	25
30	100	32	23	30
	80	29	20	29
	60	28	21	27
8	100	36.1	26	32
	80	34.27	25	31
	60	33.38	24	30

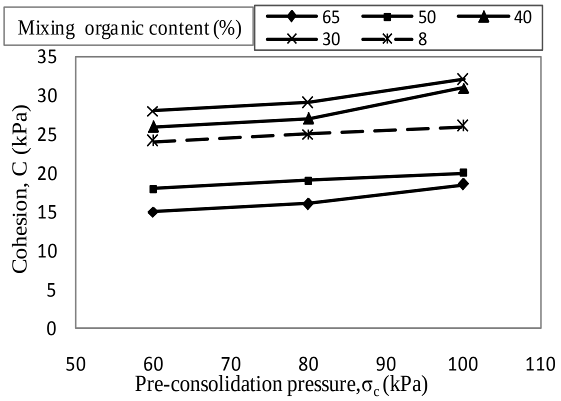

Figure 4: Variation of cohesion with pre-consolidation pressure

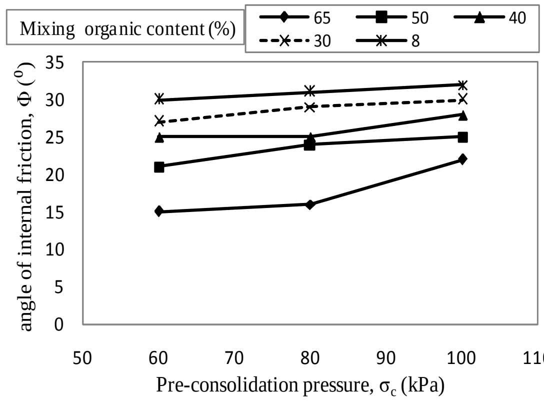

Figure 5: Variation of angle of internal friction with pre-consolidation pressure

3.3 Effect of Pre-consolidation on the Compressibility Parameter of Reconstituted Soil

The variation of compressibility parameters in terms of initial void ration (e_o) and compression index (C_c) with the variation of σ_c 60 to100 kPa are presented in Table 3 and hence discussed here. However, the variation of e_o in relation to the variation of σ_c of reconstituted soils is shown in Figure 6. The Figure 6 depicts that e_o decreases with the increasing of σ_c. In organic soil the void space is more and filled up by air or/and water. Oades (1989) described that physical properties are changed significantly with organic contents. Moreover, From the Figure 7 it is clear that Cc has decreased significantly with the increase of σ_c of reconstituted soils. The main purpose of consolidation tests is to obtain the data that may be used in predicting the rate and the amount of settlement of structures. Conducting a series of K_o-consolidation tests in the laboratory the compressibility parameters in terms of e_o, C_c and C_v as well as primary consolidation period were obtained at varying organic content of reconstituted soils.

Table 3: Compressibility parameter and pre-consolidation pressure in the reconstituted soil

Mixing organic content, OC (%)	Pre-consolidation pressure, σ_c (kPa)	Compressibility parameter	
		Initial void ratio, e_o	Compression Index, c_c
65	100	1.47	0.586
	80	1.51	0.634
	60	1.73	0.827
50	100	1.4	0.516
	80	1.44	0.569
	60	1.49	0.63
40	100	1.3	0.428
	80	1.38	0.469
	60	1.428	0.548
30	100	1.11	0.421
	80	1.34	0.446
	60	1.37	0.454
8	100	0.827	0.249
	80	0.827	0.335
	60	0.91	0.377

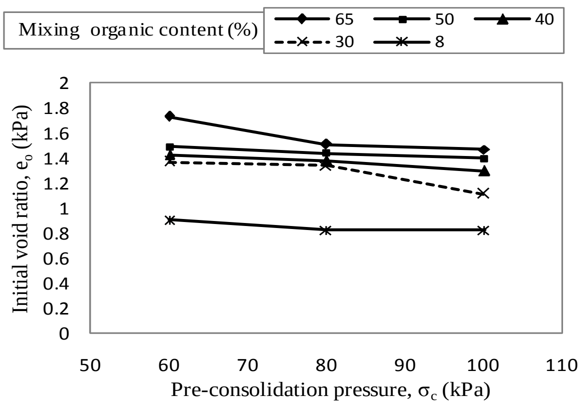

Figure 6: Variation of e_o with pre-consolidation pressure.

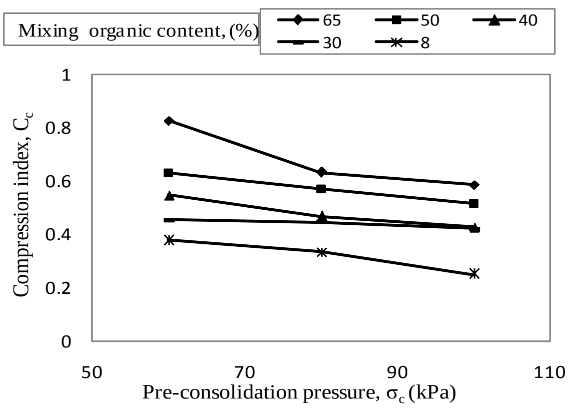

Figure 7: Variation of compression index with pre-consolidation pressure

3.4 Effect of Organic Content on the Shear Strength Parameter of Reconstituted Soil

The stress- strain behavior have been established for the organic contents of about 5 to 45% of reconstituted soil. From this stress-strain behavior the undrained shear strength (s_u) of reconstituted soils for various organic contents were evaluated. The finding are presented and discussed in the following sections.

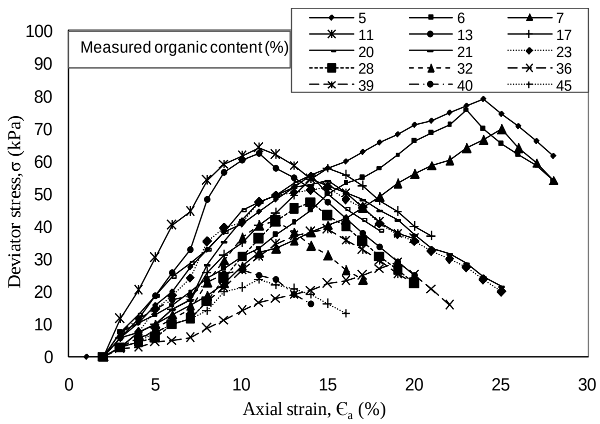

Figure 8: Deviator stress versus axial strain of reconstituted organic soil.

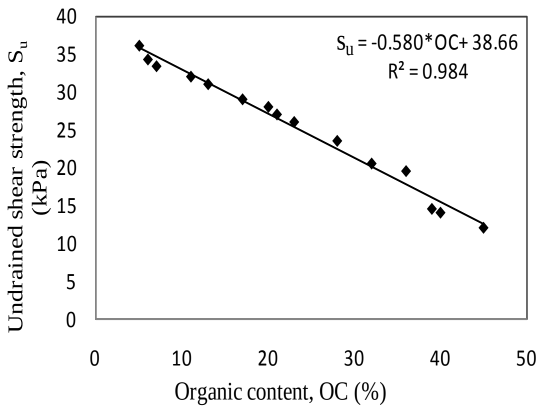

Figure 9: Variation of undrained shear strength in relation to the increasing of organic content

3.4.1 Stress-strain behaviour

The evaluated stress-strain behavior of reconstituted soil at varying organic content is presented in the form of deviator stress, $\Delta\sigma'$ (kPa) versus axial strain, ϵ_a (%) diagram in Figure 8. The Figure 8 reveals that the stress was increased with the increase of axial strain and showing almost similar behavior for the reconstituted soils at any organic contents. The figure also depicts that the strength decreases with the increase of organic contents.

Table 4: Shear strength parameter and organic content in the reconstituted organic soil

Organic content, OC (%)	Unconfined compressive strength, q_u (kPa)	Undrained shear strength, s_u (kPa)	Cohesion, c (kPa)	Angle of internal friction, ϕ (°)
5	72.2	36.1	26	32
6	68.54	34.27	25	31
7	58.76	29.38	24	30
11	64	32	23	30
13	62	31	22	28
17	58	29	20	30
20	56	28	21	27
21	54	27	19	25
23	52	26	18	24
28	47	23.5	20	25
32	41	20.5	19	24
36	39	19.5	18.5	23
39	29	14.5	18	21
40	28	14	16	16
45	24	12	15	14

3.4.2 Shear strength parameter

The variation of s_u, c and ϕ in relation to the variation of organic content in reconstituted soil are presented in Table 4 and hence discussed here. The s_u of reconstituted soils varies with the increase of organic content as shown in Table 4 and also in Figure 9. From the figure it can be seen that s_u has decreased significantly from 36.1 to 12 kPa for the increase of organic content from 5 to 45% in the reconstituted soils. Here, it is important to note that a correlation between S_u and OC can be expressed by the Equation 1

$$s_u = -0.580*OC + 38.66 \text{ ....(1)}$$

whose regression coefficient, R^2=0.984 that means the measured organiic content against the mixing organic content in slurry was found closer to each other and their evaluate s_u is satisfactory.This relation is clearly shown in Figure 9. Moreover, from the Figure 10 it can be seen that the cohesion has decreased significantly from 26 to 15 kPa for the increase of organic content from 5 to 45% in the reconstituted soils and from here it is interesting to notice that a correlation between c and OC is expressed by the Equation 2.

$$c = -0.223*OC + 25.54 \text{...............(2)}$$

In contrast, the ϕ varies with the increase of organic content shown in Figure 11. This figure shows that there is a linear decrease of ϕ with the increase of organic content. From the figure it can be seen that ϕ has decreased significantly from 32 to 15^o for the increase of organic content from 5 to 45% in reconstituted soil and from here it can also be observed that a correlation between φ and OC is expressed by the Equation 3.

$$\Phi = -0.370*OC + 33.79 \text{............(3)}$$

Whose regression coefficien, R^2=0.914. This relation is clearly shown in Figure 11

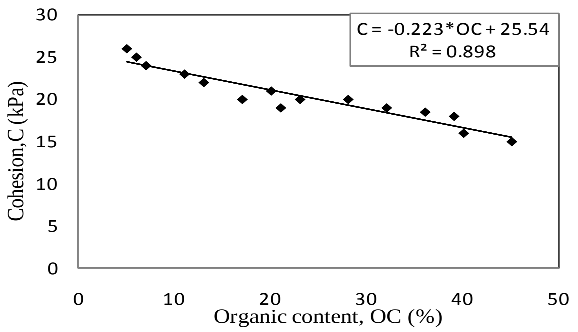

Figure 10: Variation of cohesion in relation to the increasing of organic content

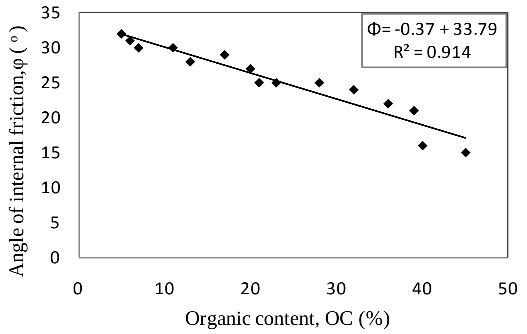

Figure 11: Variation of angle of internal friction with organic content of reconstituted soil

3.5 Effect of Organic Content on the Compressibility properties of Reconstituted Soil

The compression index is obtained from the void ratio (e) versus effective pressure, $\sigma^{/}$ (kg/cm^2) diagram. Conducting a series of K_o-consolidation tests in the laboratory the compressibility parameters (C_c and C_v) are obtained at varying organic content in reconstituted soils and hence discussed in the following sections.

3.5.1 Initial void ratio

The variation of e_o of reconstituted soil with the increase of organic content is presented in Figure 12. The Figure 12 illustrates that e_o has increased from 0.827 to 1.73 for the increase of organic content from 5 to 45%. In organic soil the void space is more and filled up by air or/and water. Oades (1989) described that physical properties are changed significantly with organic contents. From here it is significant to note that, a correlation between e_0 and OC can be expressed by the Equation 4.

$$e_o = 0.0180*OC + 0.884.............(4)$$

3.5.2 Compression index

The variation of C_c with that of organic content is shown Figure 13 and it can be signified that C_c has increased significantly from 0.249 to 0.827 with the increase of organic contents from 5 to 45%. To predict the amount of settlement, knowledge on C_c of soil must be understood to solve the soil engineering problems (Lambe 1969). However, the amount of settlement of a soil mass depends on its composition and its pore spaces. It can be seen from here that, a correlation between c_c and OC is expressed by the equation 5.

$$c_c = 0.010*OC + 0.267.............. (5)$$

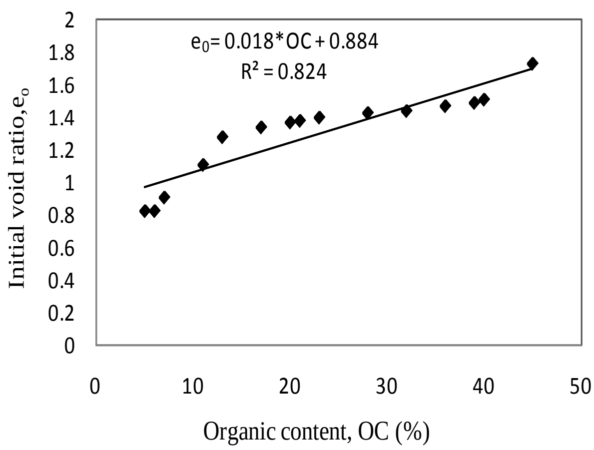

Figure 12: Variation of initial void ratio in relation to the increasing of organic content

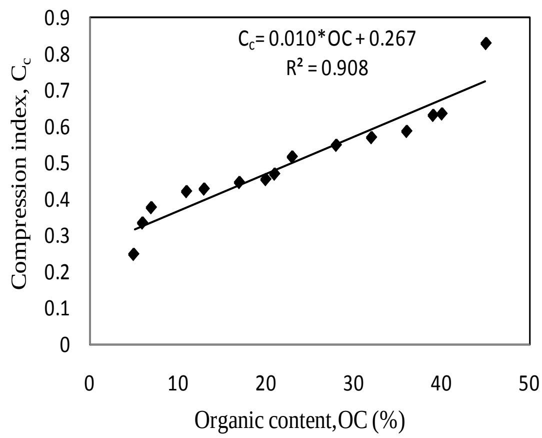

Figure 13: Variation of Compression index in relation to the increasing of organic content

3.5.3 Coefficient of consolidation

The variation of C_v with the increase of organic contents is presented in Table 5 and also in Figure 14. The figure illustrates that there is an increasing trend of C_v with the increase of organic contents. It can be seen from the figure that at a particular applied pressure, say 100kPa, the value of C_v has changed from 0.0076 to 0.0285cm^2/sec for the increase of organic contents from 5 to 45%. Similar increasing trend of C_v are also observed for other applied pressure ranging from 25 to 800kPa. Here, it can be concluded that the values of C_v was found as insignificant for low organic content and then significant for high organic content. The value of C_v depends on organic matter, permeability, void ratio, C_c and applied pressure (Lambe 1969). Moreover, due to induced organic matter in the soil mass, the C_v has changed significantly, with respect to the applied pressure.In contrast, the variation of C_v with the increase of applied pressure for different organic contents as depicted in Figure 15. The figure reveals that there was an increasing trend of C_v with the increase of applied pressure at varying organic content. For the organic content of 20%, the value of C_v increases from 0.0095 to 0.0351cm^2/sec for the increase of applied pressure from 25 to 800kPa. Similar degree of increment was also observed for the other organic content.

Table 5: Coefficient of consolidation with organic contents under applied pressure on reconstituted soil

Organic Content OC (%)	Coefficient of consolidation, C_v (cm^2/sec) at varying applied pressure, σ (kPa)					
	25	50	100	200	400	800
5	0.0051	0.0063	0.0076	0.0088	0.0104	0.0114
6	0.0056	0.0065	0.0091	0.0099	0.0114	0.0127
7	0.0062	0.0071	0.0109	0.0114	0.012	0.0152
11	0.0074	0.0099	0.0117	0.0142	0.0207	0.0268
13	0.0079	0.0114	0.0127	0.0152	0.0228	0.0285
17	0.0091	0.0117	0.0134	0.0169	0.0233	0.0326
20	0.0095	0.012	0.0146	0.0175	0.0243	0.0351
21	0.0097	0.0127	0.0152	0.019	0.0268	0.038
23	0.0099	0.0152	0.019	0.0228	0.0285	0.0456
28	0.0109	0.0148	0.0175	0.0233	0.03	0.057
32	0.012	0.0142	0.019	0.0253	0.038	0.0756
36	0.0127	0.0168	0.0215	0.026	0.039	0.076
39	0.015	0.0175	0.0228	0.0268	0.0396	0.0844
40	0.0152	0.019	0.0253	0.032	0.045	0.0912
45	0.0189	0.024	0.0285	0.033	0.0456	0.104

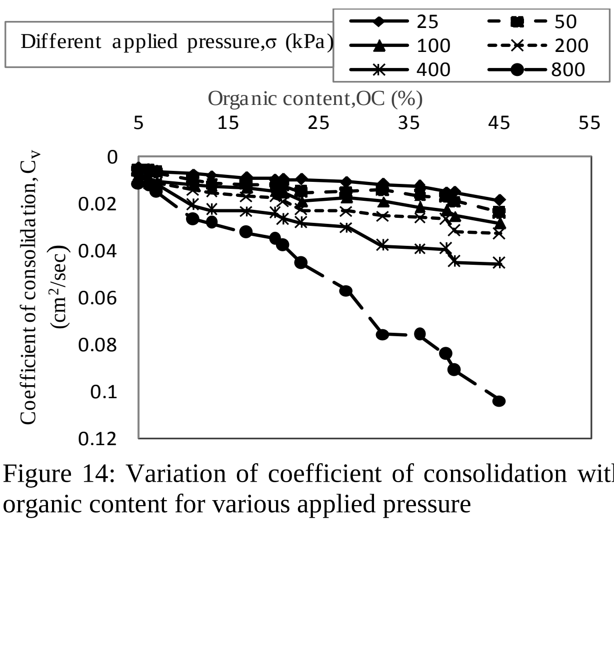

Figure 14: Variation of coefficient of consolidation with organic content for various applied pressure

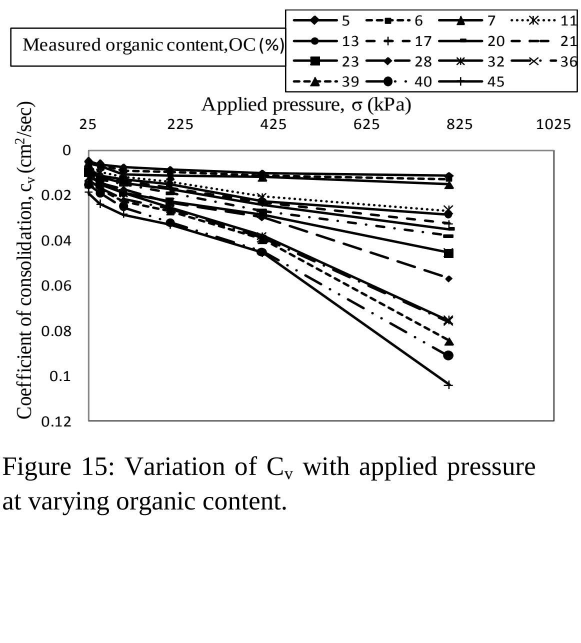

Figure 15: Variation of C_v with applied pressure at varying organic content.

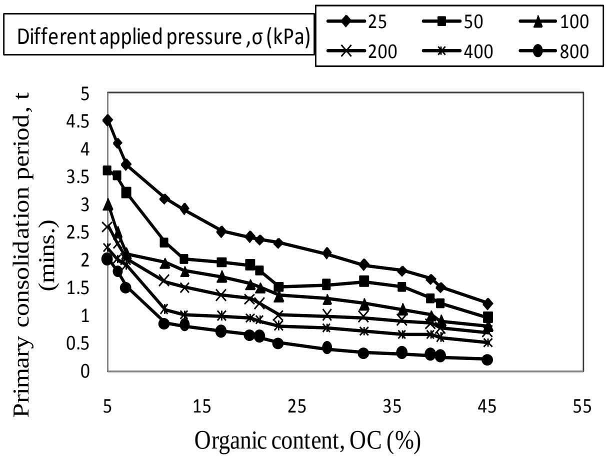

Figure 16: Variation of primary consolidation period with organic content at varying applied pressure

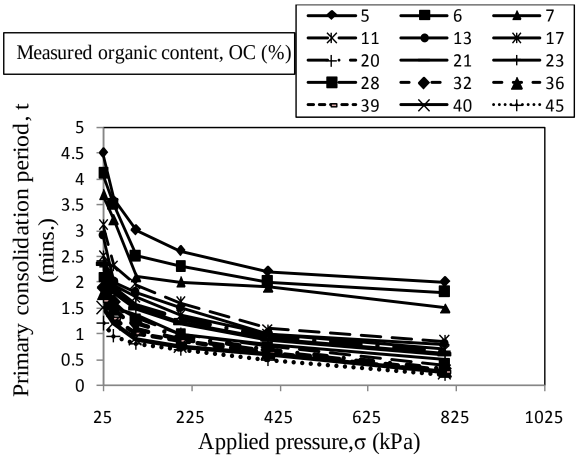

Figure 17: Variation of primary consolidation period at varying applied pressure

3.5.4 Primary consolidation period

The decreasing trend of t for the increase of organic contents in reconstituted soil is presented in Figure 16. For the applied pressure 100kPa, the value of t decreases from 3 to 0.8 mins for the increase of organic contents from 5 to 45%. Similar degree of decrement was also observed for the other applied pressure. The figure also depicts that the values of t, was found insignificant at high pressure and then it was significant for low pressure. The organic soils generally posses low shear strength and high compressibility and the time required for primary consolidation has varied significantly with the change of applied pressure and organic content (Rafizul et al. 2010). In contrast, the values of t, of reconstituted soil with the increase of applied pressure as presented in Figure 17. The figure depicts that there was a trend of non-linear variation of the decrease of t, with the increase of applied pressure in all percentages of organic contents. For the organic contents of 20%, the value of t, decreases from 2.40 to 0.65 mins. for the increase of applied pressure from 25 to 800kPa. Similar degree of decrement is also observed for the other organic contents. From the figure it can be concluded that the values of t, was found significant at low organic content and then insignificant for the high organic content.

Table 6: Coefficient of compressibility with organic content at varying applied pressure

Organic content, OC (%)	Coefficicent of compressibility, a_v (cm^2/kg) at varying applied pressure σ_c (kPa)					
	25	50	100	200	400	800
5	0.16	0.142	0.084	0.059	0.037	0.015
6	0.167	0.146	0.085	0.071	0.039	0.019
7	0.189	0.155	0.098	0.09	0.046	0.02
11	0.362	0.19	0.116	0.106	0.054	0.025
13	0.362	0.197	0.151	0.111	0.056	0.028
17	0.397	0.205	0.152	0.112	0.058	0.029
20	0.406	0.222	0.16	0.118	0.057	0.031
21	0.421	0.29	0.176	0.125	0.058	0.032
23	0.435	0.365	0.182	0.121	0.066	0.031
28	0.446	0.38	0.186	0.125	0.063	0.032
32	0.467	0.417	0.211	0.125	0.066	0.032
36	0.474	0.435	0.215	0.135	0.07	0.034
39	0.588	0.435	0.217	0.146	0.071	0.038
40	0.641	0.479	0.272	0.163	0.08	0.044
45	0.952	0.692	0.272	0.174	0.094	0.047

3.5.5 Coefficient of compressibility

The results about the variation of coefficient of compressibility (a_v) with the variation of organic content at varying applied pressure are presented in Table 6 and also in Figure 18. The figure shows that for the applied pressure 100kPa, the value of a_v increases from 0.084 to 0.272cm^2/kg for the increase of organic contents from 5 to 45%. Similar degree of increment is also observed for the other applied pressure.

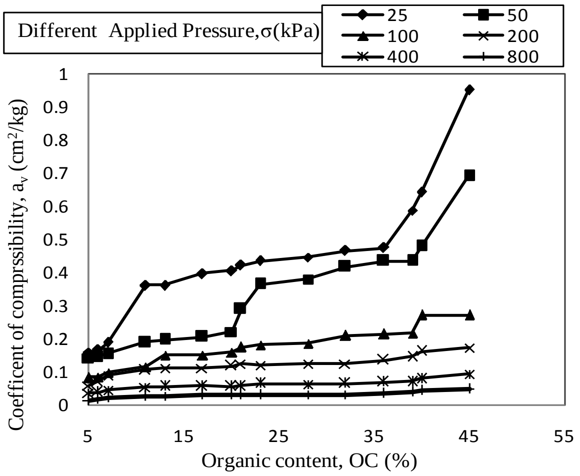

Figure 18: Variation of coefficient of compressibility with organic content at varying applied pressure

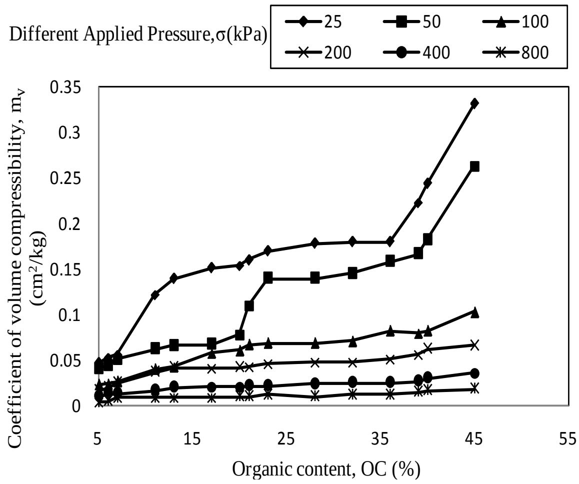

Figure 19: Variation of coefficient of volume compressibility with organic content at varying applied pressure

3.5.6 Coefficient of volume compressibility

The variation of coefficient of volume compressibility (m_v) of reconstituted soils with the increase of organic contents is presented in Figure 19. The Figure illustrates that at a particular applied pressure, say 100kPa, the value of m_v has changed from 0.023 to 0.103cm^2/kg for the increase of organic contents from 5 to 45%. Similar increasing trend of m_v are also observed for other applied pressure ranging from 25 to 800kPa.

4. VERIFICATION AND COMPARISON

Any developed correlation or empirical equation is focused to verification and comparison with those published results available in the literature for the similar cases. This section consists of the laboratory test results on the reconstituted soils to observe the variation of different shear strength and compressibility parameters with the change of organic contents in the reconstituted soils.

4.1 Shear Strength Parameter

In this section, it is projected to verify and compare the founded empirical equations between the shear strength parameters and percentage of organic contents of reconstituted soils. The verification and comparison of s_u of reconstituted soil can be expressed by an emperical Equation 1 and presented in Figure 20. In the present study, the s_u decreases with the increase of organic content. The results reported by Franklin et al (1973) and Rafizul et al. (2006) are compared with these evaluated results and it is revealed that s_u also decreases with the increase of organic content. The decreasing trend of s_u with the increase of organic contents is comparable for all the cases. Due to the factors on which the magnitude s_u depends the results obtained from present study and that of reported by Franklin et al (1973) differs from each other significantly. From the figure, it can be seen that for Franklin et al (1973) the value of s_u for the reconstituted organic soils, decreases significantly from 158.5 to 97.50 kPa respectively, with the increase of organic contents from 5 to 28%. For Rafizul et al., (2006) value of s_u decreases significantly from 35.36 to 20.50 kPa respectively, with the increase of organic contents from 6 to 42%, it is also observed that the decreasing rate of s_u is 36.1 to12 kPa for the present study with the increase of organic contents from 5 to 45%.

4.2 Compressibility Parameters

Here, it is intended to verify and compare the founded empirical equations between the compressibility parameters and percentage of organic contents of reconstituted soils and hence discussed in followings.

4.2.1 Initial void ratio

Figure 21 represents the comparison of predicted results of e_o in terms of emperical Equation 4 with the increase of organic contents are compared with those reported by Coutinho and Lacerda (1987) and Rafizul et al. (2006). The e_o as reported by Coutinho and Lacerda (1987) increases with the increase of organic contents, and it is also observed for the present study, however the rate of increase is different. For example, from the figure, for Coutinho and Lacerda (1987) it can be seen that the value of e_o, increases significantly from 1.0 to 8.5, with the increase of organic contents from 0 to 60%. For Rafizul et al.(2006) the value of e_o, increases from 0.646 to 1.98, with the increase of organic contents from 5to 42% and for the present study with the increase of organic contents from 5 to 45%, while the increasing rate of e_o is from 0.827 to 1.73.

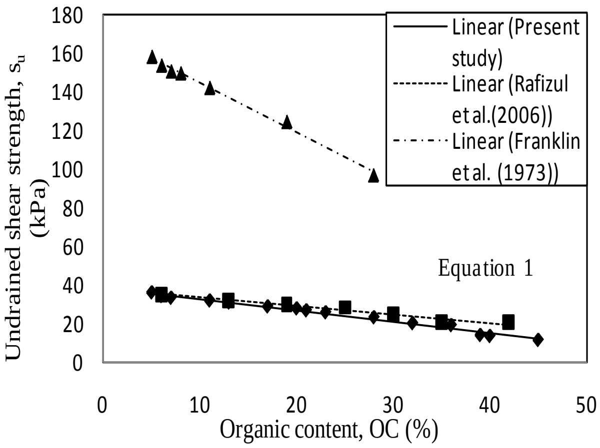

Figure 20: Comparison of s_u obtained from the proposed empirical equation in the present study with other sources.

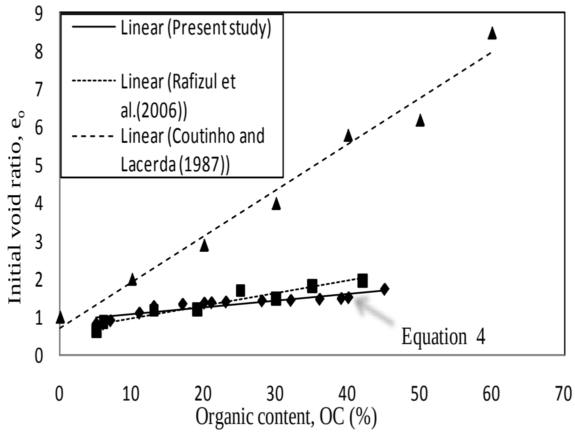

Figure 21: Comparison of e_o obtained from the proposed empirical equation in the present study with other sources.

4.2.2 Verification and comparisons of compression index

The verification and comparison of the proposed emperical Equation 5 in terms of compression index (C_c) as presented in Figure 22 are with those of reported by Coutinho and Lacerda (1987) and Rafizul et al.(2006) for the same cases of reconstitited soil. In the present study, the C_c increases from 0.249 to 0.827 with the increase of organic content of 5 to 45%. The finding is in good agreement with the general performance of C_c versus organic contents. The results reported by Rafizul et al.(2006) give relation in the same pattern and, the results reported by Coutinho and Lacerda (1987) are higher than that of predicted by the present study. The increasing trend of C_c for the increase of organic contents is similar for both the cases.

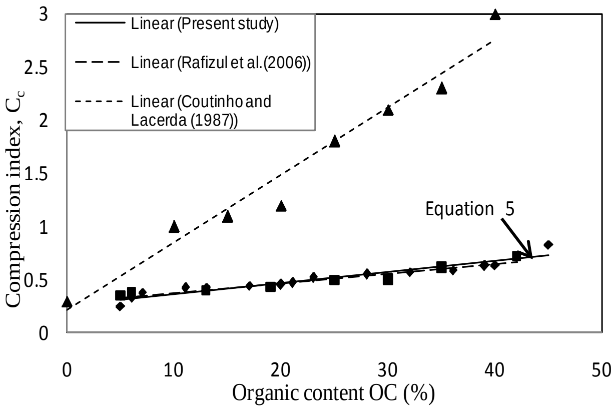

Figure 22: Comparison of C_c from the proposed empirical equation in the present study with other sources.

5. CONCLUSIONS

From observed results it can be depicted that organic content in all the reconstituted soil samples varies with the application of different pre-consolidation pressure. Results showed that with the increase of pre-consolidation pressure the measured organic content has decreased against the mixing organic content in slurry. The effect of pre-consolidation pressure which is the major factor for the variation of strength and compressibility properties is investigated by the author. Moreover, the undrained shear strength has decreased with the increase of organic content and the stress-strain diagram shows almost similar behaviour in all the reconstituted soils. In contrast, the compression index has increased with the increase of organic content. Moreover, the coefficient of consolidation has increased for all applied loading conditions as well as with the increase of organic content. Finally, it can be concluded that shear strength and compressibility properties with the variation of organic contents can be expressed by a series of empirical equations (where symbols bear their usual meanings) with reasonable degree of accuracy and judgment. However, the properties of organic soil might be evaluated knowing the simple soil parameters with the help of these proposed empirical equations.

s_u = -0.580*OC + 38.66 ....(1)

c = -0.223*OC + 25.54...............(2)

Φ = -0.370*OC + 33.79.............(3)

e_o = 0.0180*OC + 0.884.............(4)

c_c = 0.010*OC + 0.267..............(5)

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STABILIZATION OF SOFT CLAY SOIL USING RICE HUSK ASH

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ABSTRACT

This paper depicts, the problem with expansive soils can be minimized by improving the properties of soil using rice husk ash (RHA) as an admixture. The potential of RHA is to reduction of swelling and excessive settlement of soil under footing of building sub-grade of road. To improve the physical properties such as strength, workability and compressibility characteristics as well as stabilizing of soil different percentage of RHA content were mixed with the collected clayey soil. Using ASTM standard performance of the soil-RHA was investigated with respect to compaction characteristics, consistency characteristics, free swell index, shear strength and unconfined compressive strength UCS) tests. The test result shows that there is a decrease in the maximum dry density (MDD) and increase in optimum moisture content (OMC) with increase in RHA content. Stress strain behavior of unconfined compressive strength showed that the failure stress and strains increased with the addition of RHA. The optimum compressive strength was obtained at 10% RHA content. The shear strength parameters of the soil can be improved by increasing the RHA content.

Keywords: *RHA, consistency, shear strength parameters, UCS , Swelling and consolidations*

1. INTRODUCTION

Geotechnically, soil improvement could either be by modification or stabilization, or both. Soil modification is the addition of a modifier (cement, lime, etc) to a soil to change its index properties, while soil stabilization is the treatment of soils to enable their strength and durability to be improved such that they become totally suitable for construction beyond their original classification.(Alhassan, M., 2008). Clays exhibit generally undesirable engineering properties. They tend to have low shear strengths and to lose shear strength further upon wetting or other physical disturbances (Mitchell, 1986). Partially saturated clayey soils having high plasticity are very sensitive to variations in water content and show excessive volume changes. Such soils, when they increase in volume because of an increase in their water contents, are classified as expansive soils. This highly plastic soil creates as cracking and break-up of pavements, railways, highway embankments, roadways, building foundations, channel and reservoir linings, irrigation systems, water lines, sewer lines etc (Gromko, 1974, Mowafy, 1985 and Kehew, 1995). Rice husk is an agricultural waste obtained from rice milling. About 108 tons of rice husks are generated annually in the world (Alhassan, 2008). In Bangladesh, about 39.3 million ton of rice is produced annually (Mustafi, 2005) which generate about 9.83 million ton of RH after milling of the paddy is used as animal food as well as fuel in rural area. RHA which is generated from the burning of Rice husk is considered as waste material and usually dumped backside of the kitchen of the village people in Bangladesh. Rice husk ash has high quantity of silica with small quantities of oxide (Agarwal, 1989, Kumar, 1993) having high specific surface that is very suitable for activating the reaction of soil and act as a binding material like cement. Thus RHA can be used as a cost-effective additive particularly in regions having high production capacity like Bangladesh. The silica content in RHA depends on the burning temperature. The technique which is used for burning RH to produce amorphous silica is suitable for pozzolana cement production, have been developed (Kumar, 1993, Ahmed, et al.1993).

The objective of this paper is to use soft soil as a construction material using RHA which are generally treated as waste materials in our country. The soils which are treating with RHA were collected near Khan Jahan Ali Hall at Khulna University of Engineering & Technology (KUET) in Khulna city, Bangladesh. The soil reports collected from the laboratory of this university were studied to obtain the general characteristics of soil in this area. These reports show that it has high content clayey silt of very soft to soft consistency up to 20 ft. from the ground surface level. High water content, compressibility and low workability of these soils often caused difficulties in the civil engineering construction projects. The soil which is used for the construction pavement or sub-base should have some specification of geotechnical properties for obtaining required strength against tensile stresses and strains variety. In this study, RHA was used for the improvement of workability, compressibility and compaction characteristics as well as the physical properties of highly plastic clayey soil. In the earlier, cement and lime are the two main materials used for stabilizing soils which is now costly in price due to the sharp increase in the cost of energy since 1970s (Neville, 2000). RHA is the most cost-effective locally available materials act as a binding agent like cement which increases some geotechnical properties as well as stabilization of soil as an alternative option of cement and lime.

2. MATERIALS AND METHODS

2.1 Materials

The soil samples, generally greyish colered silty clay, were collected from Khanjahan Ali Hall at Khulna University of Engineering & Technology(KUET) in Khulna, Bangladesh. This samples were dried with oven at 105^0C. The oven dry soils were first ground into smaller size by hand and machine. The grinding samples were screening throgh 4.75 mm apearture sieve before sending for testing. According to the AASHTO classifiation systems, the soil is characterized as A-7-6 and acoording to the Unified soil classification sytems, samples are naming as CL(Clay with low plasticity). The particle size distribution of original soil as show in figure 1.

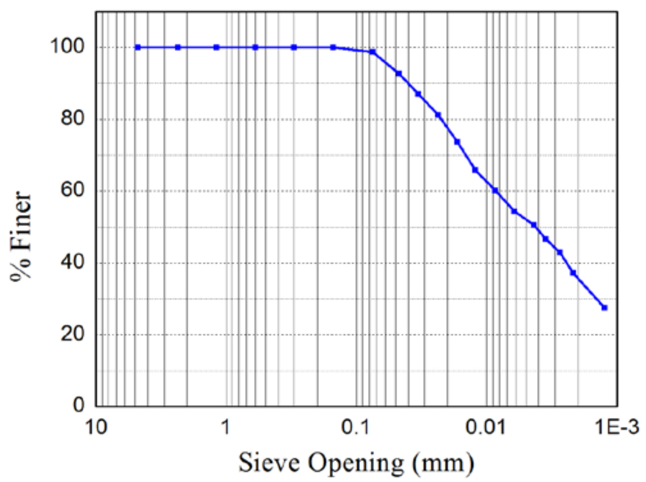


Figure 1: Particle size distribution of original soil

In this study, RHA were used as stabilizing materials which were prepared by simply burning rice husk (RH). These RH were collected from the locally available mills in Khulna city. The temperatures of whole burning process were recorded by five thermocouples connecting with a data logger. The produced RHA contains about 93% silica which is a key factor for improving the properties of soil.

2.2 Preparation of Testing Samples

The whole tests were performed in five different schemes as shown in Table 1. In each case, 4000 gm soil were mixed with five different percentage(5%,7.5 %,10 %,12.5 %) of RHA content. The collected soil samples and ash contents were oven-dried at 105^0C overnight to remove moisture and repress microbial activity.Then the oven dried samples were mixed throughly by hand in a large tray in a dry state.

Table 1: Combination scheme of soil samples

Samples ID	Samples Description		
	Soil (gm)	RHA (gm)	% Ash content
Original soil	4000	--	0
R1	4000	200	5
R2	4000	300	7.5
R3	4000	400	10
R4	4000	500	12.5

2.3 Testing Procedure

To determine the workability, compressibility, strength and compaction characteristics of treated and original soil samples several tests were performed in this study. Before conducting test, firstly the ash treated samples were mixing about ten minutes with water by hand. After that, the mixtures were put into polyethylene bags and mixing was continued by shaking and overturning the bag to squeeze out the air into the soil void. In the similar way, different amount of water were added to the different soil samples and mixing as before described to obtain the optimum moisture content and maximum dry density. A series of standard proctor tests on non-treated and RHA treated soils (5, 7.5, 10% and 12.5% RHA content) were conducted according to ASTM D 698.

Specific gravity of original and ash treated soil samples were determined according to ASTM D854-10. Consistency of the original and treated soil samples were determined by Atterberg limit test(ASTM D-4318-10).The effect of shrinkage of fine grained soils are of considerable significance to cause serious damage to small building and highway pavements. Free swell index of soil samples were determined as per as IS: 2720(Part-40). Ten grams of soil samples were put into two 100ml glass cylinder and the remaining 90ml of the cylinder was filled with distilled water and kerosene respectively. After keeping it overnight the volume of soil in each cylinder was measured and from this volume free swell index of soil was determined from the equation shown in below.

Free swell index, %= $$\frac{Volume\,of\ in\,distilled\ water - Volume\,of\ in\,distilled\ \mathrm{ker}\,osene}{Volume\,of\ in\,distilled\ \mathrm{ker}\,osene}\,x100$$

All the test specimens were prepared at the optimum moisture content before performed the remaining selected properties. After adding water the test specimens were prepared with the standard Proctor compaction test (ASTM D 698). The specimens were demolded after completion of compaction and samples of different size were prepared as per requirement for performing the selected test in uncured condition.

The uncured unconfined compressive strength of the specimens was determined according to ASTM D-2166-98 and cylindrical specimens of 36-mm-diameter and 71-mm-lengthwere used for this test. Drained shear strength parameters(c & φ) were determined by direct shear test (ASTM D3080-3) of the compacted uncured soil specimens. To perform these tests, three samples of 60mm diameter and 25 mm height were prepared from the compacted samples. Settlement characteristics of soils were determined by performing Laboratory consolidation test (ASTM D-2435) and samples of 63.5 mm diameter and 25 mm height were prepared for performing these test.

3. RESULTS AND DISCUSSIONS

3.1 Compaction Characteristics

The compaction characteristics of soil mainly depend on both grain size distribution and specific gravities of the soil. The variation of optimum moisture content and maximum dry density of RHA treated and untreated soil is shown in figure 2. This figure represents the maximum dry density of soil decreases gradually with an increase of. This is due to comparatively low specific gravity value 2.25 of RHA than that of soil which is 2.65 and the initial simultaneous flocculation and agglomeration of clay particles caused by cation exchange.

On the other hand, the optimum moisture content of soil increases with an increase RHA because fines of RHA are more than soil. As a result fines of the treated soil as well as surface area of particles in treated soil increases which have an affinity with water. The RHA content also decrease the quantity of free silt and clay fraction and forming coarser materials which occupy larger spaces for retaining water. The increse of water content also attributed by the pozzalanic reaction of RHA with the soil.

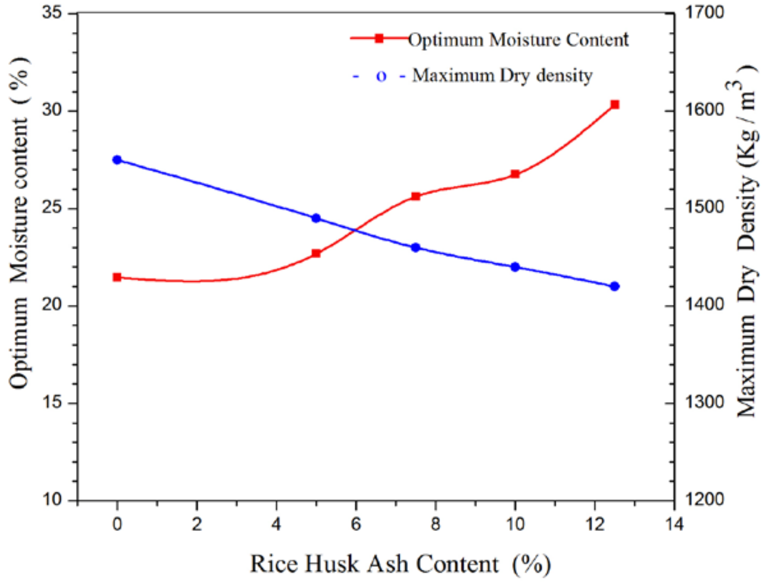


Figure 2: Variation of maximum dry density and optimum moisture content with rice husk ash content.

3.2 Specific gravity

The specific gravity of soil is decreasing due to the addition of RHA as shown in Figure 3 because of the low specific gravity of RHA as compared with soil.

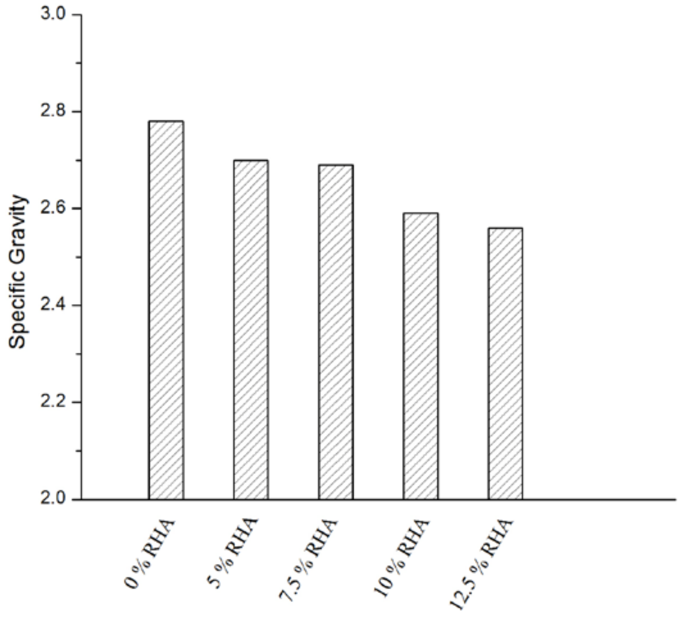


Figure 3: Variation of specific gravityof soil with rice husk ash content.

3.3 Consistency

Consistency is expressed interms of liquid limit, plastic limit and shrinkage limit are extensively used by engineers to discriminate between the different kinds of soil within a broad category.the value of liquid limit, plastic limit and shrinkage limit are measured from Atterberg limit test. The Atterberg limits are a basic measure of the nature of a fine-grained soil. The test result of atterberg limt is shown in Table 2.

Table 2: Atterberg limit test result

RHA(%)	Liquid Limit(%)	Plastic Limit(%)	Plasticity Index(%)	Shrinkage Limit(%)
0	46	22	24	22
5	50	27	23	27
7.5	52	31	21	28
10	55	34	21	31
12.5	56	36	20	34

The variation of liquid limit with the percentage of RHA is shown in Figure 4. This figure illustrate that liquid limit of soil increses gradually from 46% to 50% with the increses of RHA content up to 12.5. This improvement attributed that more water is required for the RHA treated soil to make it fluid because of the pozalonic characteristics of RHA . Similar variation was obtained for plastic limit that the value of plastic limit increases graduallyfrom 22% to 36% with the increases of RHA up to 12% due to the pozalonic charactericts of RHA as shown in Figure 5. This improvement of plastic limit implies that RHA treated soil required more water to change it plastic state to semisolid state. This may also improve the shear strength characteristics of soil.

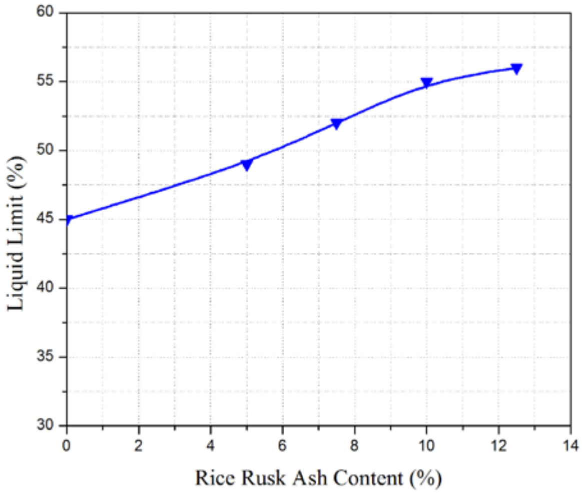


Figure 4: Variation of LL with RHA

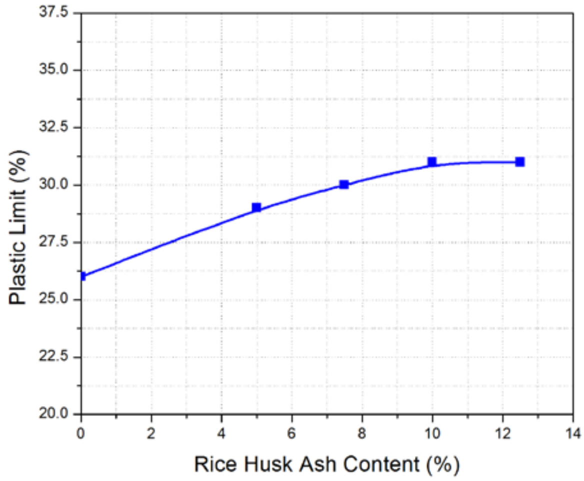


Figure 5: Variation of PL with RHA

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. The variation of shrinkage limit is shown in Figure 6. Figure 6 illustrate that as the addition of RHA the value of shrinkage limit increases from 22% to 34%. It is clear from that result that the RHA treated soil absorb more water to change it semi-solid state to solid state.

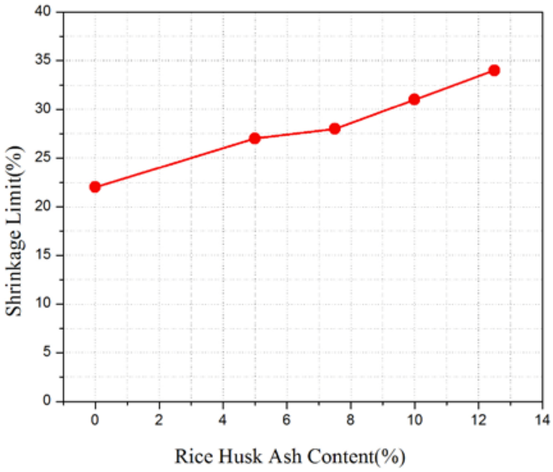


Figure 6: Variation of Shrinkage Limit(SL) and Shrinkage Ratio(SR) with RHA

3.4 Free swell index

The variation of free swell index with increasing RHA content is shown in Figure 7. The free swell index gradually decreased from 20% to 17.5% for up to 10% RHA content. After the addition of 2.5% more RHA content with the soil, it felt down pointedly from 17.5% to 10%. So it is clear from the above discussion that swelling of soil as well as the possibility of crack formation on foundation can be minimized with the addition of RHA.

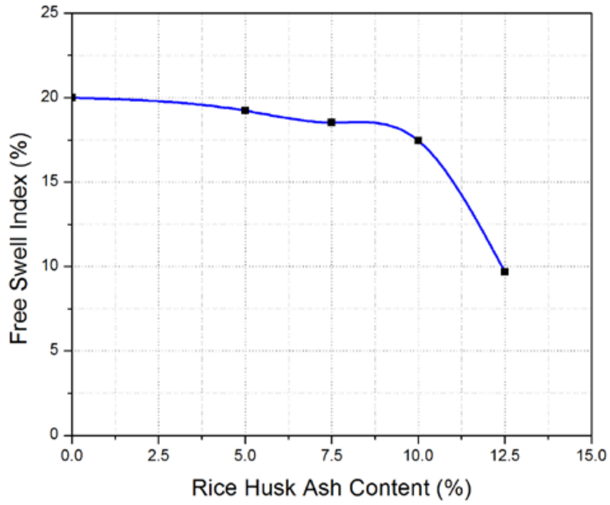


Figure 7: Variation of Free swell index with RHA

3.5 Unconfined Compressive Strength

The test result of unconfined compressive strength is shown in Figure 8. This figure illustrates the stress-strain behavior of original and RHA treated soil under vertical load. Initially the stress is gradually increases with the increase of strain. After attaining the peak stress, it decreases with the increase of strain for all the combination of RHA and soil. Approximately all the specimen shows shear failure after observing the failure plane of specimens. The variation of unconfined compressive strength for soil at different percentages of RHA is shown in Figure 9. There is a rapid increase of unconfined compressive strength from 0.06 MPa to 0.172 MPa with the addition of only 5% RHA.

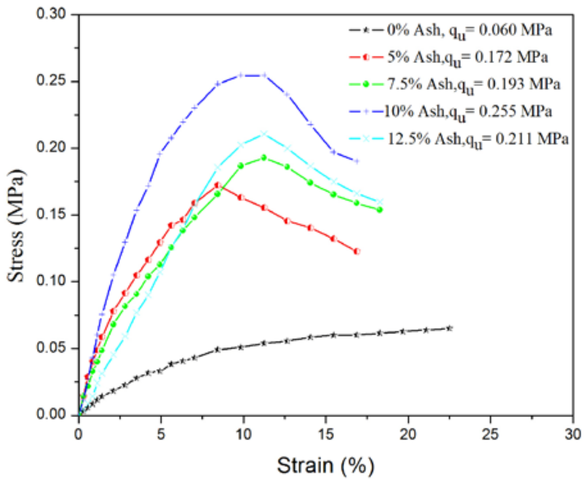


Figure 8: UCS of original and RHA treated soil

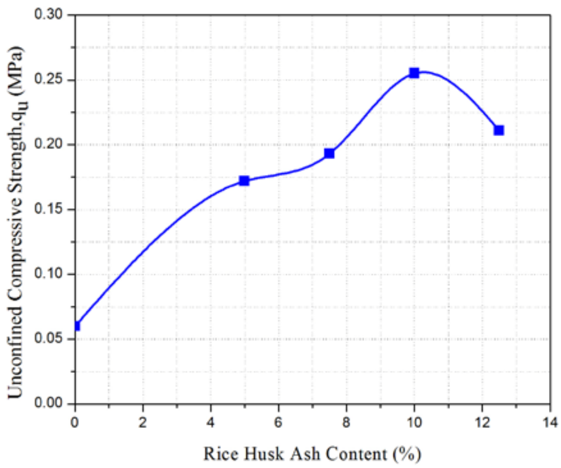


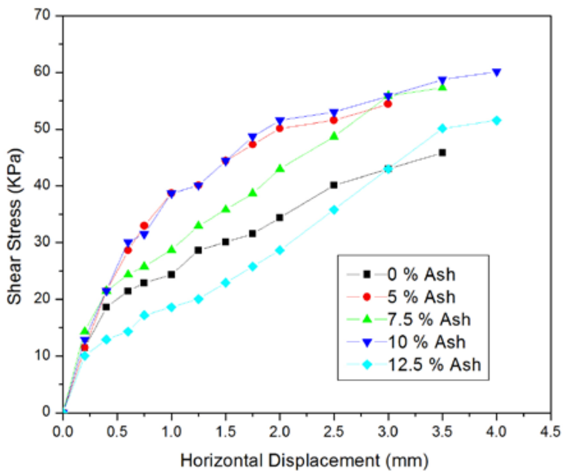
Figure 9: Variation of UCS with RHA

The optimum value of unconfined compressive strength 0.255 MPa were obtained for 10% of RHA content. After that, the value of unconfined compressive strength decreased from 0.255 MPa to 0.211 MPa for 12.5% of

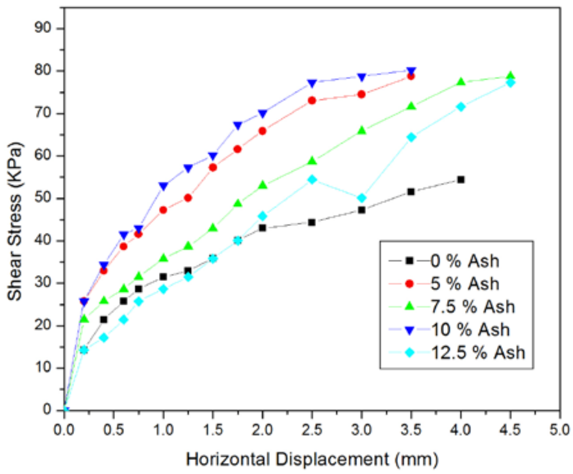
RHA content. So it can be accomplished that the maximum unconfined compressive strength were obtained at 10% of RHA and after that the value of unconfined compressive strength decreased with the addition of RHA. The following mechanism also explains that there is an positive impact of RHA on uncofined confined compressive strength.The reason for this improvement is due to the pozzolanic reactions of RHA with soil. This results in agglomeration in large size particles and causes the increase in compressive strength

3.6 Shear Strength

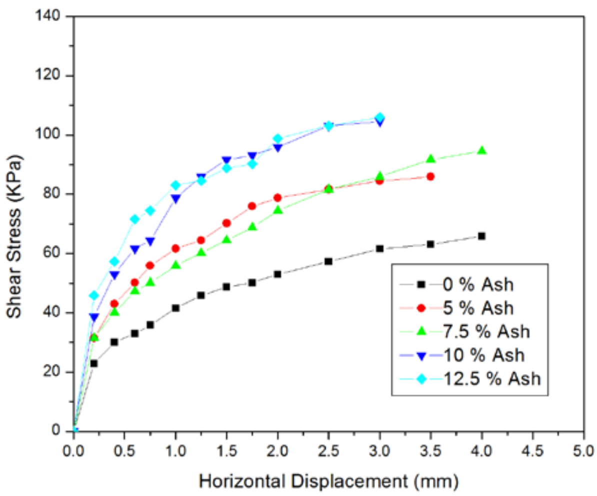
Generally soils are weak in shear force and the failure of structure on soil is due to the shear failure of soil. The variation of shear stress with horizontal displacements for the applied normal stress of 34.696 KPa, 69.392 KPa and 104.087 KPa is shown in Figure 10. Theses curves illustrate that the value of shear stress increases with the shear displacement for all the applied normal stress until failure occurs This Figure also illustrate that the optimum shear resistance obtained in case of 10 % RHA treated soil for all the three applied normal stress.



(a) For 34.696 KPa normal stress



(b) For 69.392 KPa normal stress



(c) For 104.087 KPa normal stress

Figure 10: Variation of shear stress with horizontal displacement

The shear strength parameters(c&φ) also determine from these curve. After determining the shear strength parameters , it is plotted against RHA as shown in Figure 11. According to the illustration, there is a rapid increase in cohesion(c) for the first 5 % of RHA content ant it decreased mildly for the further amount of RHA content upto 10%. There is a dramatic decrease in cohesion(c) for the 12.5% RHA content and it decreased from 37.24KPa to 23.87Kpa. On the other hand the value of angle of internal friction(φ) increases gradually with the increases of RHA content. For the first 5% of RHA content there is rapid increase in angle of internal friction(φ) from about 16^0 to 25^0. There is a gradual increase in angle of internal friction(φ) for next percentage of RHA content. This improvement of angle of internal friction(φ) implies that the silica content in RHA act as a binder which agglomerate the particles into a larger one and the soil changes from clay to silt.

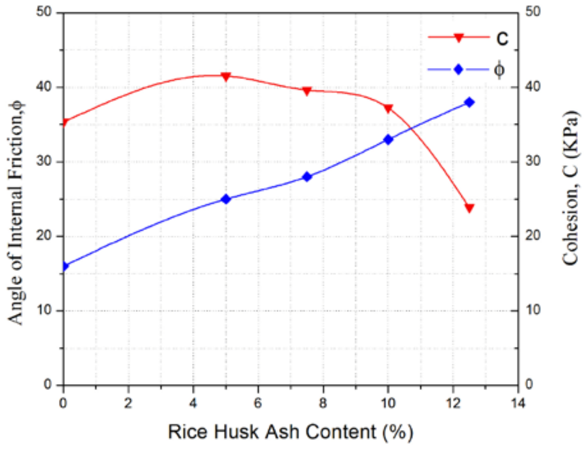


Figure 11: Variation of shear strength parameters(c&φ) with RHA

3.7 Consolidation Characteristics

In this study one dimensional consolidation test were performed to determine the consolidation characteristics of original and RHA treated soil. After the test result, a graph is plotted for different void ratio against vertical stress in loading and unloading condition as shown in Figure 12. This figure illustrate that the void ratio increases with the increases of stress in loading condition and after unloading the value of void ratio increases. The settlement of soils is directly related to the compression index. The variation of compression index and initial void ratio with RHA content are shown in Figure 13. Firstly, this plot shows that the compression index (C_c) is decreasing gradually from 0.368 to 0.328 with increasing RHA for up to 7.5%. The value of C_c decreased well under from 0.328 to 0.248 for the addition of 10% RHA with the soil and after that it is slightly increased from 0.248 to 0.258 for 12.5% RHA content. This decrease in compression index implies that there could be a result of increased formation of pozzolanic products within the pore spaces of soil from physicochemical changes (Osinubi et al. 2006) which leads to a reduction in compression index. When the rice husk ash content exceeds the quantity required for the soil-ash reaction, they will be filled between the voids of the soil. A more compact state of the soil is probably attained. On the other hand the value of initial void ratio (e_o) increased gradually from 1.305 to 1.378 with increasing RHA for up to 7.5%. Then the value of e_o decreased well under from 1.378 to 1.315 for the addition of 10% RHA and after that, it decreased slightly from 1.315 to 1.314 for 12.5% RHA content.

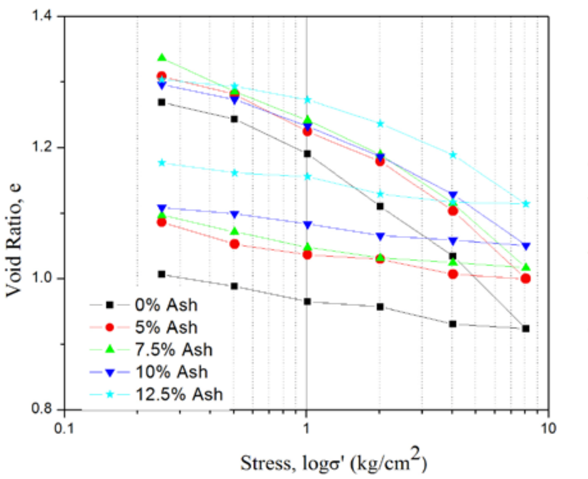


Figure 12: Plot of void ratio,e versus effective stress, σ

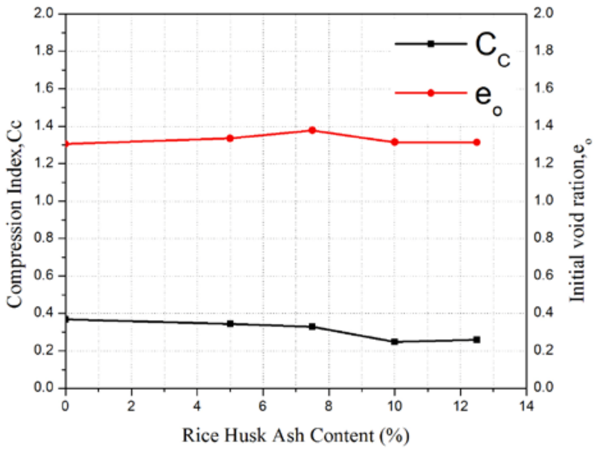


Figure 13: Variation of Initial void ratio(e_o) and Compression index(c_c) with RHA

4. CONCLUSIONS

In this investigation, the optimum RHA content for stabilization of soft clay soil were determined with respect to some geotechnical properties such as compaction characteristics, consistency, swelling, lightness, shear strength and compressive strength. The test result shows that the maximum densities of soil were decreases with the increase of RHA content because of the lower specific gravity of RHA. Whereas the value of maximum moisture content shows positive relation with the addition of RHA content because more water is required for the hydration of the pozzalonic RHA. Moreover, the value of liquid limit and plastic limit also increased with the increasing percentage of RHA whereas the value of plasticity index shows different characteristics. Increasing the amount of RHA cause a decrease in shrinkage limit as well increase in shrinkage ratio which improves the shear strength characteristics of soil. The pozzalonic behavior of RHA makes the RHA treated soil coarser than original soil samples due to the agglomerations of RHA and soil particles. This improvement changes the naming of soil from clay to silt. The free swell index test result shows a negative relationship with RHA as it decreased with the increase of RHA content which reduced the possibility of crack formation on the surface of foundation. The unconfined compressive strength test result shows a positive relation with RHA up to 10 % of RHA. After that the unconfined compressive strength decreased for further amount of RHA content. The optimum compressive strength was obtained for 10% of RHA content. The optimum shear strength of soil was obtained for 10% of RHA. The cohesion of soil shows an increasing order for first 5 % of RHA and after that this value decreases with the addition of RHA whereas the angle of internal friction shows a positive relationship with RHA content. From the consolidation test result, it can be concluded that the amount of settlement due to consolidation can be minimized by mixing RHA with soil. The test result shows that the values of compression index decreased with the increases of RHA and the initial void ration shows positive relation with RHA. Finally this paper depicts that the soil can be made lighter which leads to decrease in dry density and increase in moisture content and reduced free swelling and consolidation settlement due to the addition of RHA with the soil. Besides that the unconfined compressive strength and shear strength of soil can be maximized at the addition of 10% RHA content with the cohesive soil. So it can be finally concluded that RHA can be used as a stabilizer of soft soil and 10% RHA treated shows the optimum value.

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Proceedings of the 1st International Conference on Civil Engineering for Sustainable Development (ICCESD-2012), 23~24 March 2012, KUET, Khulna, Bangladesh (ISBN: 978-984-33-4247-8)

FINITE ELEMENT SIMULATION OF THE NORMAL FAULT RUPTURES, SAND DEPOSIT AND RAFT FOUNDATION INTERACTION

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ABSTRACT

An incomplete understanding of the failure mechanisms in fault rupture propagation has led to inconsistent and insufficient regulations in building codes. In the present study, a sophisticated numerical model is calibrated and validated in order to clarify a complex problem involving the interaction of fault ruptures, medium dense Fontainebleau sand deposits, and existing structures across the fault plane. Calibration is performed using direct shear test data. Repeatable centrifuge models of dip-slip normal faults with a dip angle of 60° in the free field condition and light and heavy rigid strong raft foundations are used for the validation. The present numerical model satisfactorily simulates the centrifuge models. Rigid rafts divert the shear bands so as to bypass the rafts, rather than rip them apart. The rafts tilt on the foundation soil during fault rupture. The raft-tilting increases as the raft bearing pressure on the soil decreases.

Keywords: *Sand deposits, Normal fault, Raft foundation, Centrifuge, Shear band, FEM.*

1. INTRODUCTION

The co-seismic movements on the fault surface in the dip direction cause permanent soil deformations in the overlaying soil strata. In a relatively dense soil, the strains may be localized in narrow shear bands that propagate upward in the soil. If the soil is less dense, soil deformations during fault rupture may be associated with extensive shear bands in the overlaying soil. Terzaghi (1943) classified these failure patterns as general and local failures, and Bray et al. (1994) reported that these failure patterns occurred in brittle and ductile ground materials, respectively. As a secondary effect, the resulting permanent soil displacements in the overlaying soil cause additional stresses in the geotechnical structures in and above the soil layer. The resulting strains or displacements of the structure may violate the serviceability requirements of the structure, or worse, cause structural collapse. The published guidelines by US governmental agencies (Cole and Lade, 1984) or other building design codes include inconsistent setback criteria to restrict the distances between the facilities and existing active or potentially active faults. Modifying the current design codes for these facilities in the seismically active regions requires a better understanding of the complex problem of the interaction of fault ruptures, soil deposits, and existing structures. However, the main goals of previously published studies were to identify the failure pattern in the alluvium (Bray et al., 1994; Cole and Lade, 1984; Roth et al., 1981, Lade et al., 1984) and the height of fault rupture in the model ground (Bray et al., 1994) as well as the general criterion for surface faulting (Scott, and Schoustra, 1974) in the free field condition. However, a number of field case studies (Bray et al., 1994; Taylor et al., 1985; Anastasopoulos, 2005) were conducted to investigate the failure mechanisms in the soil during a fault rupture event and the effects of the resulting ground movements and deformations on exiting structures across or near the fault rupture plane.

Soil behavior after failure has been shown to play a major role in problems related to shear band formation and propagation, and this mechanism can be investigated using an accurate numerical model. Applying the finite element method (FEM) in combination with the elastic-perfectly plastic constitutive soil model with Mohr-Coulomb failure criterion, Scott and Schoustra (1974). obtained results that contradict both reality and experiments. In contrast, Bray (1990) and Bray et al. (1994 and 1994), also using the FEM with a hyperbolic non-linear elastic constitutive law, achieved satisfactory agreement with small-scale model test results. Walters and Thomas (1982) performed a sandbox experiment and FEM simulations and found that the non-associated flow rule and proper strain softening were essential in the localization of rupture. Other researchers (Roth et al., 1982; Nakai et al., 1995; White et al., 1994) applied the finite difference method with the elasto-plastic constitutive model, Mohr-Coulomb failure criterion, and strain softening. In their simulations, rupture propagated through the sand and broke the ground surface with only a fraction of the displacement observed in experiments. However, the previous numerical models used for the analysis of fault rupture propagation through overlaying soil beds were not accurate and robust in the modeling of material hardening/softening or in handling the mesh size effect (due to softening) and the confining pressure effect, which indicates the necessity for an accurate numerical model and its validation.

In the present study, a numerical model using an elasto-plastic framework with a non-associated flow rule, a strain-hardening/softening law, and the shear band effect is calibrated and validated for the study of a complex target problem involving the interaction of normal fault ruptures, sand deposits, and rigid raft foundations. The calibration of the numerical model is performed using direct shear model test data. Centrifuge models, which can produce prototype stress level in the model ground of dip-slip normal fault events with a dip angle of 60° in the free field condition, and light and heavy rigid strong raft foundations are used to validate the numerical model, by comparing the failure mechanisms from experimental and numerical results. Such an integrated study will help to ensure that the proposed numerical model is applicable in the real-world design and analysis of such complex problems.

2. TESTING PROCEDURES

The beam centrifuge of the University of Dundee was used in the experiment on fault rupture propagation through a Fontainebleau sand deposit (D_{50} = 0.24 mm, C_u = 1.33, G_s = 2.59, e_{max} = 0.833, e_{min} = 0.55, and fines content = 0%). The internal dimensions of the strongbox are 800 mm × 500 mm × 500 mm (**Fig. 1**). The strongbox has front and back transparent Perspex plates through which the models were monitored during the tests. Two hydraulic cylinders were used to push the right part up or down to simulate reverse and normal faulting. A central guide (*G*) and three wedges (A_1-A_3) were used to guide the imposed displacement at the desired dip angle (60°). Sand was pluviated in the strongbox in 20-30-mm-thick layers up to 217 mm. A line of dyed sand was laid on top of each layer in order to clearly visualize the shear bands. The density measuring cans were placed to verify the sand unit weights near the edges, at the bottom, and in the middle of the model ground. A series of digital images were taken for the displaced model ground after each stepwise fault dip slip of approximately 0.5 to 1.5 mm until the total machine allowable maximum vertical component of base dislocation, h_{max}, was reached, as shown in **Table 1**. The displacement vectors and shear strains in the model ground were analyzed using the deformation measurement system (Geo-PIV program of White et al. [15]). In addition, linearly variable differential transformers (LVDTs) were used to monitor the vertical settlements of the model ground surface and the vertical component of the base dislocation, *h*. The parameters of the physical model used in the present study are defined in **Fig. 2**. After preparing the model, the strongbox was mounted on the centrifuge, and the centrifuge was spun to 115g. The dimensions and parameters of the prototype used in the experiments are listed in **Table 1**. Three tests were conducted in the repeatable centrifuge test setup in order to investigate the failure mechanism in the sand layer during the movement on an underlying normal fault. One test (Test12_R2) was conducted in the free field condition in order to confirm the outcropping location of the fault rupture on the ground surface, and other two tests (Tests 14_R and 15) were conducted in order to study the interaction between the fault rupture and existing rigid rafts for light and heavy structures, centered above the scarp position confirmed in the free field test. The nature and dimensions of the superstructures were not considered. Two model raft footings were used (shown by *R* in **Fig. 1**). Each had dimensions of 87 mm × 500 mm × 10 mm. Sheets of sand paper (No. 100) were glued to the bottom surfaces of the models in order to create a rough-base condition. In Tests 14_R and 15, the footings were made of steel and aluminum, respectively, in order to obtain foundation bearing pressures (*q*, **Fig. 2**) of 91 and 37 kPa, respectively. Details on the experiments of the present study can be found elsewhere (El Nahas et al., 2006).

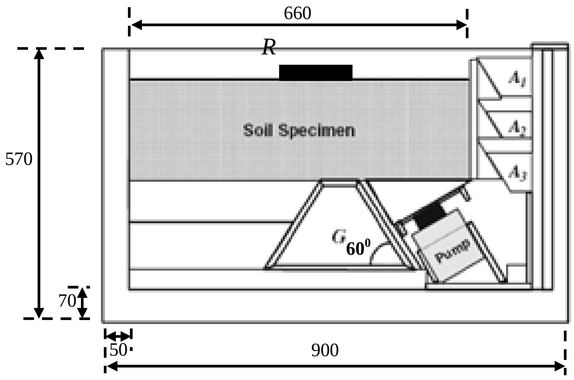


Fig.1 Dimensions of the experimental apparatus installed in the Dundee University centrifuge (all dimensions are in mm & figure is not to scale).

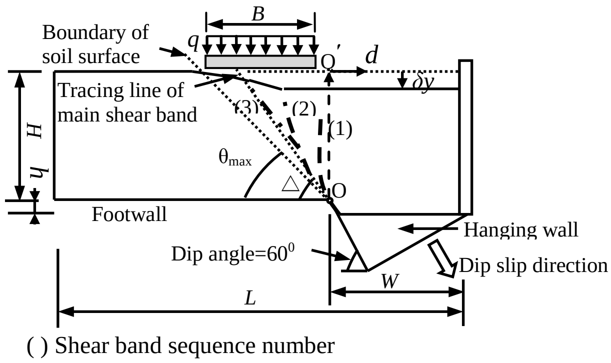


Fig.2 Definitions of the physical model with raft (figure is not to scale).

Table 1 Prototype dimensions and parameters for the centrifuge models.

Test name	D_r (%)	H^* (m)	L^* (m)	W^* (m)	h_{max} (m)	q^* (kPa)
Test12_R2	60.2	24.6	75.9	24.15	3.15	0**
Test15	59.2	24.9	75.9	24.15	3.15	37***
Test14_R	62.5	24.6	75.9	24.15	2.89	91***

* these terms are defined in **Fig. 2**.
, * fault rupture in free field and foundation centered at the scarp position confirmed in the free field test, respectively.

3. NUMERICAL MODELING

The FE model of the present study uses an elasto-plastic framework with a non-associated flow rule and a strain-hardening/softening law. An explicit dynamic relaxation method devised with a return mapping stress updating algorithm (Tanaka and Kawamoto, 1988; Tanaka and Sakai, 1993) is used for the fast solution of nonlinear equations. The standard FE solutions of the strain-softening material are strongly dependent on mesh size. Several techniques have been proposed to resolve the mesh-dependent pathology of FE solutions. For example, Pietruszczak and Mroz (1981) proposed the concept of using a softening modulus scaled by the element size. Following this method, shear banding is introduced in the numerical model by a strain localization parameter, S, in the additive decomposition of total strain increments ($d\boldsymbol{\varepsilon}$) into elastic ($d\boldsymbol{\varepsilon}^{\mathbf{e}}$) and plastic ($d\boldsymbol{\varepsilon}^{\mathbf{p}}$) parts, as follows:

$$d\boldsymbol{\varepsilon} = d\boldsymbol{\varepsilon}^{\mathbf{e}} + S d\boldsymbol{\varepsilon}^{\mathbf{p}} \tag{1}$$

where $S = F_b/F_e$ (F_b is the area of the shear band in each element, and F_e is the total area of an element). Here, F_b is decided based on the shear band thickness (*SB*). By ignoring the effects of the orientation of the shear band in each finite element, an approximated form of S is used in the present study:

$S = \frac{SB}{\sqrt{F_e}}.$ (2)

The shear band thickness is known to be approximately 16 to 30 times the mean particle diameter (D_{50}) (Vardulakis et al., 1981; Yoshida et al., 1993). In the present study, it is assumed that the deformation of a given soil element under uniform boundary stress conditions is homogeneous in the pre-peak regime and that the strain localization in a shear band starts suddenly at the peak stress state.

The yield function (f) corresponding to the Mohr-Coulomb model and the plastic potential function (Ψ), which is represented geometrically by the Drucker-Prager model, are given as follows:

$$f = -3\alpha(\kappa)\sigma_m + \frac{\sqrt{J_2}}{g(\theta)} = 0 \quad (3)$$

$$\Psi = -3\alpha'(\kappa)\sigma_m + \sqrt{J_2} = 0 \quad (4)$$

where

$$\kappa = \int d\bar{\varepsilon}_p \quad (5)$$

$$\left(d\bar{\varepsilon}_p\right)^2 = 2\left\{\left(de_{xp}\right)^2 + \left(de_{yp}\right)^2 + \left(de_{zp}\right)^2\right\} + \left(d\gamma_{xyp}\right)^2 \quad (6)$$

and σ_m is the mean stress (positive in compression), J_2 is the second invariant of deviatoric stresses, θ is the Lode angle, κ is an internal variable, and de_{xp}, de_{yp}, de_{zp}, and $d\gamma_{xyp}$ are the incremental deviatoric plastic strains along the *x*, *y*, and *z* axes. In the case of the Mohr-Coulomb model, the Lode angle function, $g(\theta)$, is given as $g(\theta) = \frac{3 - \sin\phi_{mob}}{2\sqrt{3}\cos\theta - 2\sin\theta\sin\phi_{mob}}$ (7a)

where

$$\theta = \frac{1}{3}\cos^{-1}\left[\frac{3\sqrt{3}}{2}\frac{J_3}{J_2^{3/2}}\right] \quad (7b)$$

and J_3 is the third invariant of the deviatoric stresses. The mobilized friction angle, ϕ_{mob}, is given as

$$\phi_{mob} = \sin^{-1}\left\{\frac{3\sqrt{3}\alpha(\kappa)}{2 + \sqrt{3}\alpha(\kappa)}\right\} \quad (8a)$$

where $\alpha(\kappa)$ are the frictional hardening and softening functions and expressed as

$$\alpha(\kappa) = \left\{\frac{2\sqrt{\kappa\varepsilon_f}}{\kappa + \varepsilon_f}\right\}^m \alpha_p \text{(hardening-regime: } \kappa \le \varepsilon_f\text{)} \quad (8b)$$

$$\alpha(\kappa) = \alpha_r + (\alpha_p - \alpha_r)\exp\left\{-\left(\frac{\kappa - \varepsilon_f}{\varepsilon_r}\right)^2\right\} \text{(softening-regime: } \kappa > \varepsilon_f\text{)} \quad (8c)$$

where ε_f, ε_r, and *m* are hardening/softening material parameters. The parameters of α_p and α_r are estimated using the following equations:

$$\alpha_p = \frac{2\sin\phi_p}{\sqrt{3}(3 - \sin\phi_p)} \quad (9a)$$

$$\alpha_r = \frac{2\sin\phi_r}{\sqrt{3}(3 - \sin\phi_r)} \quad (9b)$$

where ϕ_p and ϕ_r are the peak and residual friction angles, respectively, which can be obtained from appropriate model tests in the laboratory. However, in order to consider the stress level effect, the peak friction angle is estimated from the empirical equation of Bolton (1986):

$$I_r = D_r\{10 - \ln(\sigma_m)\} - 1 \quad (10a)$$

$$\phi_p = 3I_r + \phi_r. \quad (10b)$$

This equation agrees well with the direct shear test results at different confining pressures for medium dense Fontainebleau sand (**Fig. 3**). The results of this test are discussed in detail in the following section.
The plastic potential function $\alpha'(\kappa)$ is defined for the plane strain conditions as

$$\alpha'(\kappa) = \frac{\tan\psi}{\sqrt{9+12\tan^2\psi}} \quad (11a)$$

The dilatancy angle, ψ, is estimated from the modified Rowe's stress-dilatancy relationship:

$$\sin\psi = \frac{\sin\phi_{mob} - \sin\phi'_r}{1-\sin\phi_{mob}\sin\phi'_r} \quad (11b)$$

where

$$\phi'_r = \phi_r\left[1-\beta\exp\left\{-\left(\frac{\kappa}{\varepsilon_d}\right)^2\right\}\right] \quad (11c)$$

and ε_d and β are the stress-dilatancy material parameters.

The elastic moduli are estimated using the modified equation proposed by Hardin and Black (1968) and, in the case of clean sand, are given by the following equations:

$$G_{\max} = G_0\frac{(2.17-e_0)^2}{1+e_0}\left(\frac{\sigma_m}{P_a}\right)^{0.5} P_a(P_a = 98\text{kPa}) \quad (12a)$$

$$K = \frac{2(1+\nu)}{3(1-2\nu)}G_{\max} \quad (12b)$$

where G_0 is the initial-shear-modulus constant, e_0 is the initial void ratio, P_a is atmospheric pressure, and ν is Poisson's ratio.

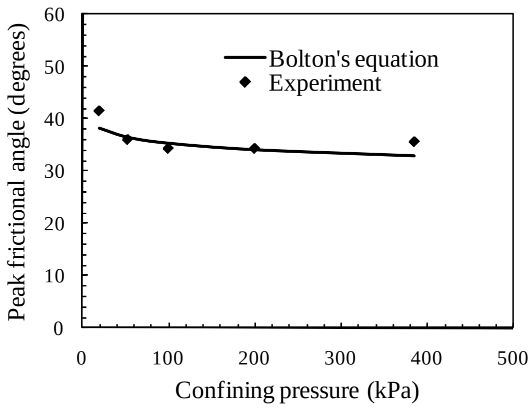


Fig.3 Effect of confining pressure on peak frictional angle.

4. CALIBRATION

In order to use the numerical model to analyze fault problems, it is necessary to calibrate the material parameters for hardening/softening (ε_f, ε_r, and *m*) and the stress-dilatancy relationship (ε_d and β) of the incorporated sophisticated constitutive model. For this purpose, the model experimental data obtained from the direct shear tests are used, because this data closely mimics the shearing mechanism in the faults. The Fontainebleau sand (D_r = 63±4%) was pluviated in the shear box. Direct shear tests were then conducted at a quasi-static displacement rate of 0.01 mm/s. The length and thickness of the specimen were 60 and 30.4 mm, respectively. The test procedures followed the test standards BS1377-7 (1990). The FE mesh used to simulate the direct shear box test is shown in **Fig. 4**. The analysis was performed for the plane strain case with the following boundary conditions. The nodes along the bottom of "A" were fixed. Nodes along the upper box sides were given a prescribed displacement in the horizontal direction. Element row "C" represents the gap between the upper and lower boxes. The upper box was free to move relative to the lower box. The solid top element row "B" was the loading plate for transmitting the vertical load. The loading plate was free to rotate and to move in the vertical direction. The material of the elements for the loading plate and the side walls was linear elastic (Young's modulus: 2.1×10^4 MPa, Poisson's ratio: 0.3). Interfaces were assumed to be a soil sample attached to all walls with a maximum frictional angle of 6° (Tatsuoka and Haibara, 1985). In order to clarify the effect of the material parameters of such a sophisticated constitutive model on the relationship between the average shear stress of the elements along the prescribed horizontal shear plane (**Fig. 4**) and the average vertical displacements of the loading plate with respect to the relative prescribed horizontal displacements in the direct shear tests, a detailed parametric study was conducted and some of the obtained results are shown in **Fig. 5(a)**. The hardening/softening material parameters ε_f, ε_r, and m influence the pre- and post-peak regimes of the relationship. The parameter ε_f is related to ε_r. The parameter m influences the hardening-regime, and ε_r influences the softening-regime. In addition, the stress-dilatancy material parameters (β and ε_d) control the mobilization of dilatancy and shear stress in the direct shear test box. As shown in **Fig. 5(b)**, there is close agreement between the numerical and experimental results for some of the direct shear tests with effective normal stresses (200 and 385 kPa) using the calibrated material parameters given in **Table 2**. Thus, the calibrated numerical model is valid for investigating the stress-level effect, the strain hardening-softening nature, and dilation in direct shear tests.

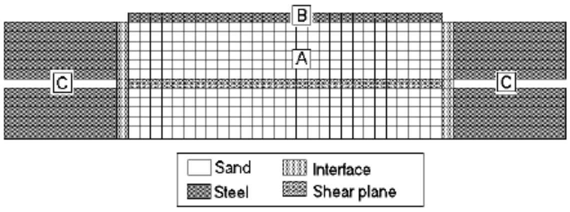
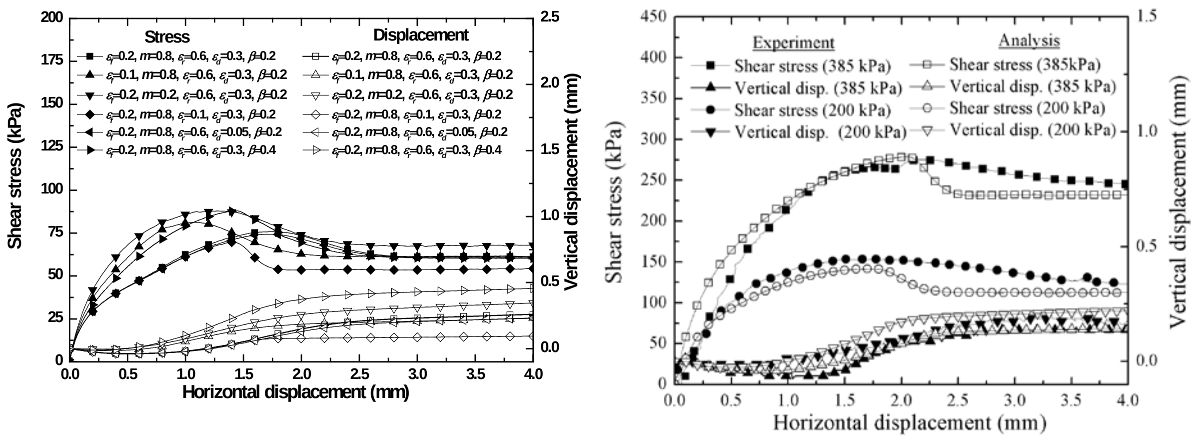


Fig.4 Finite element mesh and constituents for direct shear test box analysis ("A"=soil, "B"=loading plate, "C"=gap, figure is not to scale).



(a) (b)

Fig.5 (a) parametric study on direct sheat test box with normal effective stress of 100 kPa, and (b) comparison of experimental and numerical results (using calibrated material parameters) with normal effective stress of 200 and 385 kPa.

Table 2 Material parameters of the numerical model.

Unit weight (kN/m^3)	15.57
Initial void ratio (e_0)	0.64
Pressure coefficient at rest (K_0)	0.5
Initial-shear-modulus constant (G_0, kPa)	50
Residual friction angle (ϕ_r : °)	30.2
Poisson's ratio (ν)	0.3
Shear band thickness (SB: mm, model scale)	3.84
ε_f	0.2
ε_r	0.6
ε_d	0.3
m	0.8
β	0.2

5. FE TECHNIQUES AND MESH SIZE EFFECT

Following the recommendation of Bray (1990), the FE discretizations of the sandbox tests, as shown in **Fig. 6**, were performed such that the width of the FE model was equal to 4*H* (*H* is the uniform soil deposit thickness, **Fig. 2**) in order to minimize undesired boundary effects. The discretization was finer in the central part of the model than at the two edges, where limited deformation was expected. The differential quasielastic displacement was applied in small consecutive increments to the right side of the model (hanging wall). The elements were four-node quadrilateral Lagrange elements with reduced integration. Linear elastic elements with a Young's modulus of 2.1×10^4 MPa and a Poisson's ratio of 0.3 were used for the footing. However, following Kotake et al. (1999), no special interface elements were used along the interface between the rough footing base and the ground. Since the conventional "zero thickness" interface elements are relevant to the case of displacement discontinuity in the interface and the parameters of these elements are not so clear, particularly for modeling of the dilatancy characteristics of the thin elements adjacent to the contact. It was assumed that a shear band similar to bands that can develop inside the ground can also develop along the interface elements.

Before selecting a particular size of mesh for the analyses of the fault problems, it is necessary to investigate how much the proposed numerical model is susceptible to mesh size. In this regard, for the analyses of mesh size sensitivity, finite elements of 1 m × 1 m (width × height), 1.5 m × 1.5 m, and 2 m × 2 m in size were used in the central part of the mesh, and coarser elements of 2 m × 1 m, 3 m × 1.5 m, and 4 m × 2 m in size were used at the two edges. The sand (D_{50} = 0.24 mm) modeled in the centrifuge sand box corresponds to a prototype material with a mean particle size diameter equal to nD_{50}, where n is the scale factor. Therefore, the prototype shear band thicknesses, used for 115g (n = 115) model test analyses, was 115 × 16D_{50} (≈ 441.5 mm). All of the parameters used for the numerical analyses in the present study are given in **Table 2**. The results are compared with an arbitrary set experimental result (Test 12_R2), and no mesh size effect on the vertical displacements of the ground surface (**Fig. 7**) or the shear band orientation (**Fig. 8**) is observed. Therefore, FE meshes with FE = 2 m were used in the present study in order to reduce the computational time in analyzing the fault problems.

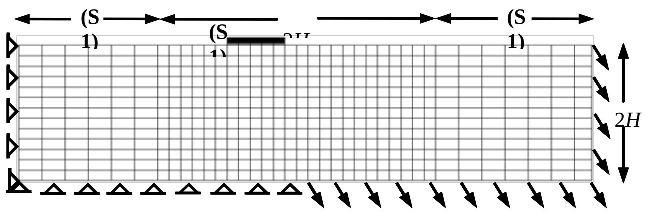


Fig.6 Finite element mesh and boundary conditions (R= raft and figure is not to scale).

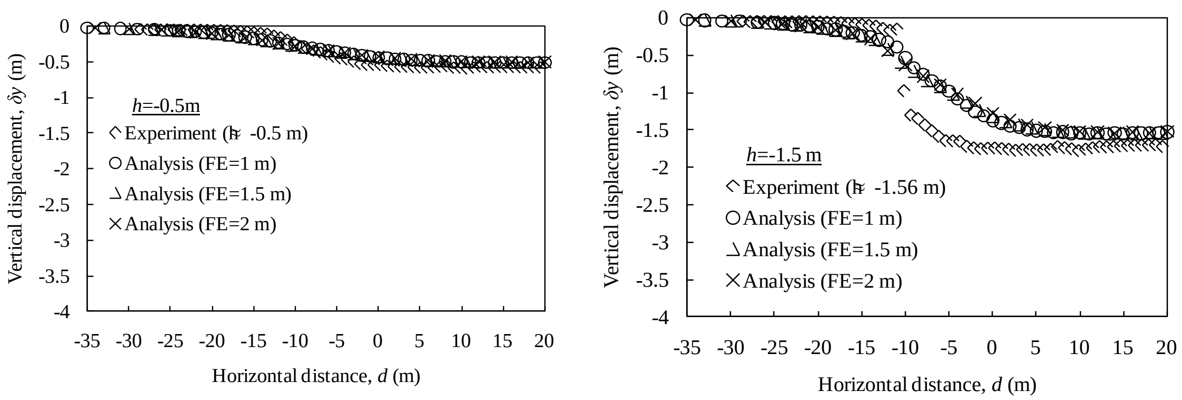


Fig.7 Analysis of sensitivity of mesh density (compared with Test 12_R2) for vertical displacements of ground surface.

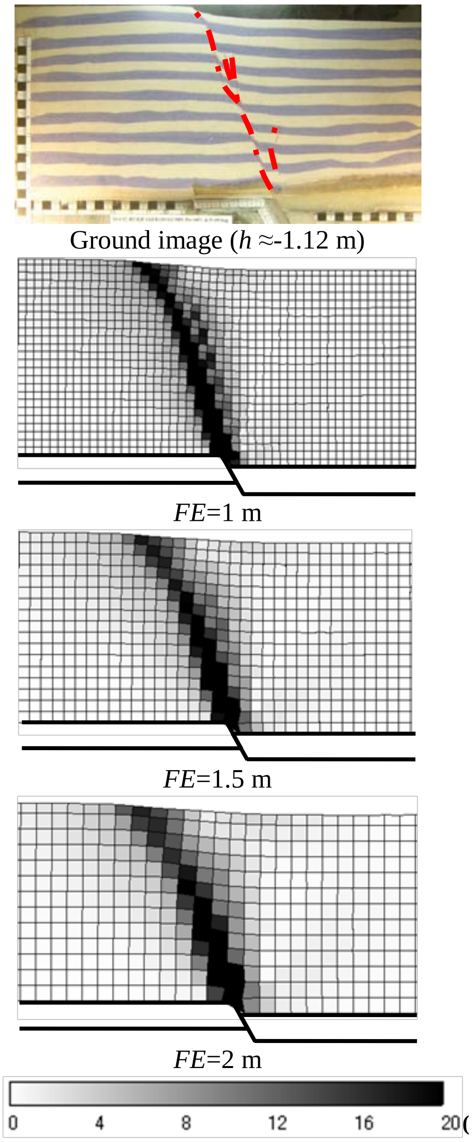


Fig.8 Comparison of experimental ground images (Test 12_R2) with deformed mesh and shear strain contour for FE sizes: 1m, 1.5 m and 2 m (figures are not to scale).

6. RESULTS AND DISCUSSION

Numerical predictions are compared with the experimental results in terms of vertical displacements on the ground surface, minimum vertical base dislocation for the rupture to reach the ground surface, the average dip angle propagated into the soil, and the horizontal extent of the deformed surface ground. It is expected that the rotations of the rigid foundations, which correspond to differential settlements across a foundation, are most likely to correlate with structural damage or lack of serviceability (e.g., Skempton and Macdonald, 1956; Charles and Skinner, 2004). This rotation is likely to lead to the redistribution of loads within a structure and may indicate a tendency for bending in a moderately flexible structure. Hence, foundation-rotation is also examined more closely in the present study.

6.1 Fault rupture in a free field (Test12_R2)

As shown in **Fig. 9**, the experimental vertical displacements on the ground for Test12_R2 ($h \approx$ -0.5 to -2 m) are closely predicted by the numerical analyses. The development of a failure mechanism in the model ground is shown in **Fig. 10**, indicating the sequence numbers of shear band formations. The shear bands were drawn by naked eye observation from the deformed colored sand layers. First, two shear bands (*S1* and *S2*) propagated upward from the fault rupture point with an increasing inclination to the horizontal, and, later, *S1* stopped propagating upward after h = -0.5 m and *S2* became inactive after h = -1.0 m. Thereafter, *S3* and *S4* started to rupture in the upper part of the model ground and extended upward moving toward the footwall. However, after a time *S3* stopped, and *S4* continued to reach the ground surface and became the slip surface of the downward moving soil mass. This extensional feature of shear band formation is implicitly predicted to some extent by the numerical analyses. The fault rupture propagated along the full depth of the model ground after reaching a value of *h* of approximately -1.1 m (experiment: -1.04 m). In the present study, for the calculation of the inclination of the main slip surface in the model ground to the horizontal (Δ, **Fig. 2**), the location of the fault rupture at the ground surface is identified by the point at which the absolute value of the second derivative of the vertical displacement along the horizontal direction is maximum (Yilmaz and Paolucci, 2007). The average dip angle, Δ, is 61° (experiment: 65°). The inclination of the line between the fault rupture point and the ends of the soil deformations at the ground surface at the end of the test (θ_{max}, **Fig. 2**) is 47° (experiment: 56.1°).

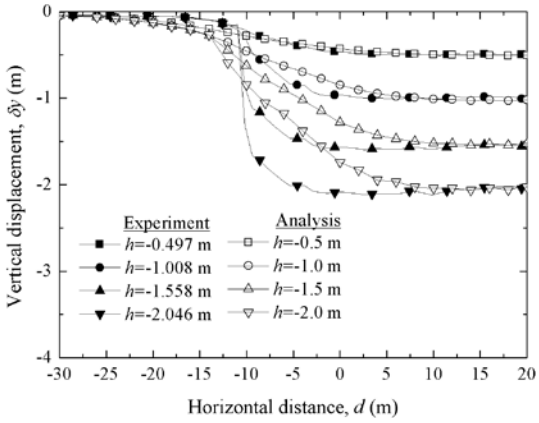
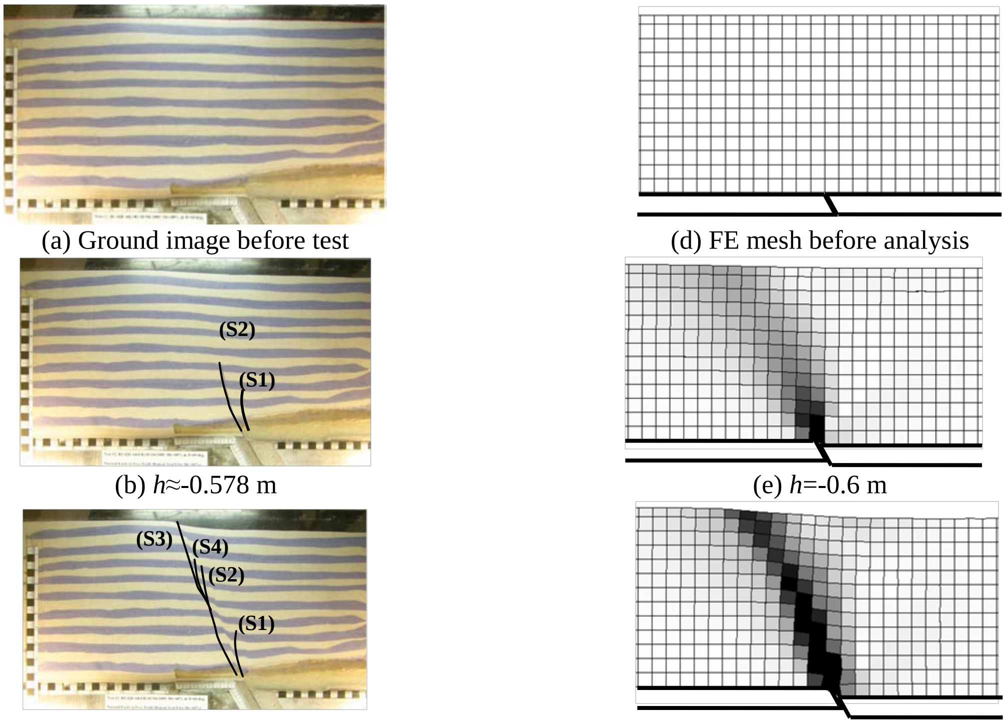


Fig.9 Comparison of experimental and numerical vertical displacements of ground surface for Test 12_R2.



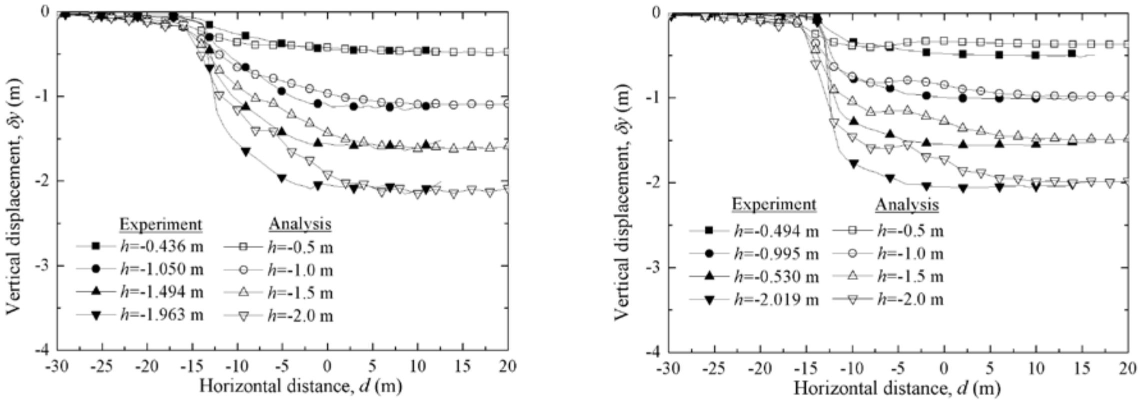
(a) Ground image before test
(d) FE mesh before analysis
(b) h≈-0.578 m
(e) h=-0.6 m
(c) h≈-1.278 m
(f) h=-1.3 m

Fig.10 Comparison of experimental ground images (a to c) and shear strain plots (darkest region indicates strain equal or more than 20%) on deformed FE mesh (d to f) for Test12_R2 in free field.

6.2 Interaction between fault rupture and rigid rafts (Tests 15 and 14_R)

The rafts (B = 10 m, **Fig. 2**) were centered at the scarp position confirmed in the free field test (Test12_R2) in Tests 15 and 14_R. As shown in **Fig. 11**, the profiles of the vertical displacements on the ground surface from experiment for base dislocations, h = -0.5 to -2.0 m, are satisfactorily predicted by the numerical model in Tests

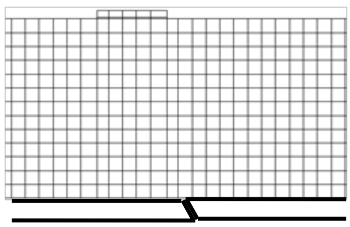
15 and 14_R with light (q = 37 kPa) and heavy (q = 91 kPa) rigid rafts, respectively. As shown in **Figs. 12** and **13**, some characteristic ground photographs taken at the start of the experiments and during the experiments are compared with the corresponding numerical deformed FE meshes with maximum average shear strain distribution in order to clarify the progressive nature of the failure mechanism. The rafts (light or heavy) do not affect the amount of base dislocation required for the main shear band to outcrop on the ground surface (see **Table 3**) because, in all cases, the shear bands cross the model ground after h = -1.1 m. However, the foundations divert the shear band so that the shear bands reach the ground surface outside of the foundations. In both cases, a shear band radiates downward from the corner of the raft and joins another shear band radiating upward from the fault point to generate the main shear band outcrops (see **Figs. 12** and **13**). This phenomenon is more clearly observed in faults with heavier foundations. The deviation of the main shear band may prevent damage to the structure. In several case studies (Murbach et al., 1999), the diversion of fault rupturing shear bands by strong rigid rafted structures was reported. Along with the diversion, the foundation is subject to rotation during faulting. As shown in **Fig. 14**, the variation of foundation rotation is a function of the interface frictional angle, h, and the footing pressure. The friction angle of 6° of the soil elements attached to the bottom of the footings closely predicts the footing rotations. The tilting markedly decreases after the main shear band outcrops. Due to the increase in footing pressure, the footing tilting decreases. The average propagated dip angles, Δs, are 57.8° (experiment: 63°) and 57.3° (experiment: 62.8°) with light and heavy rafts, respectively (see **Table 3**). The average propagated dip angles in the presence of foundations are less than those in the free field condition. The horizontal extent of the deformed surface ground (θ_{max}) are 48.8° (experiment: 54°) and 49.2° (experiment: 58.2°) with light and heavy raft foundations, respectively.



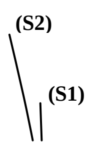
(a) (b)

Fig.11 Comparison of experimental and numerical vertical displacements of ground surface for (a) Test 15 with light raft and (b) Test 14_R with heavy raft.

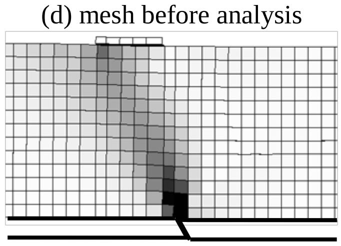
(a) Ground image before test



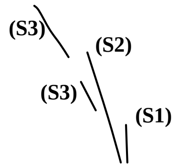
(d) mesh before analysis



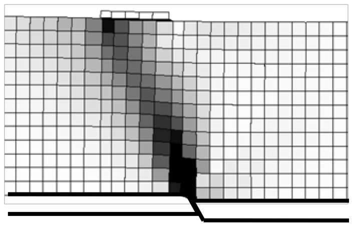
(b) h≈-0.736 m



(e) h=-0.73 m



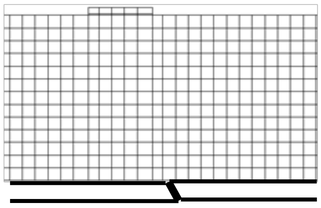
(c) h≈-2.139 m



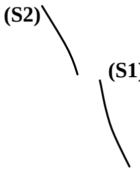
(f) h=-2.1 m

Fig.12 Comparison of experimental ground images (a to c) and shear strain plots (darkest region indicates strain equal or more than 20%) on deformed FE mesh (d to f) for Test 15 with light raft.

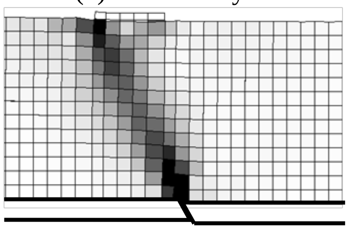
(a) Ground image before test



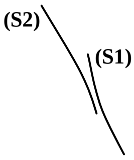
(d) before analysis



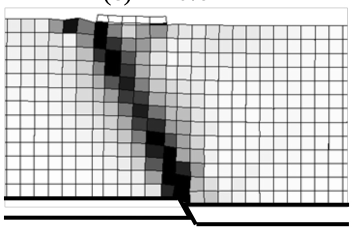
(b) h≈-0.675 m



(e) h=-0.67 m

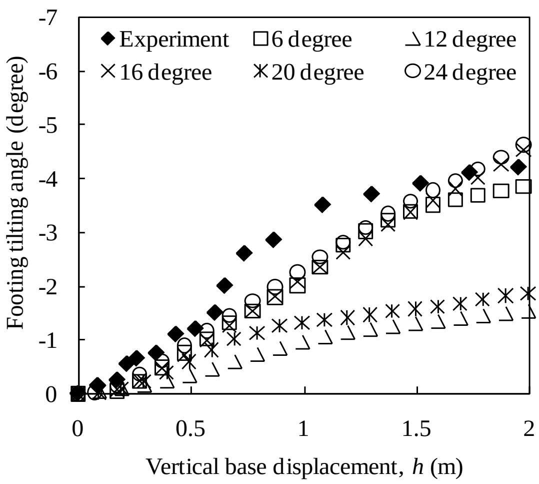


(c) h≈-1.105 m

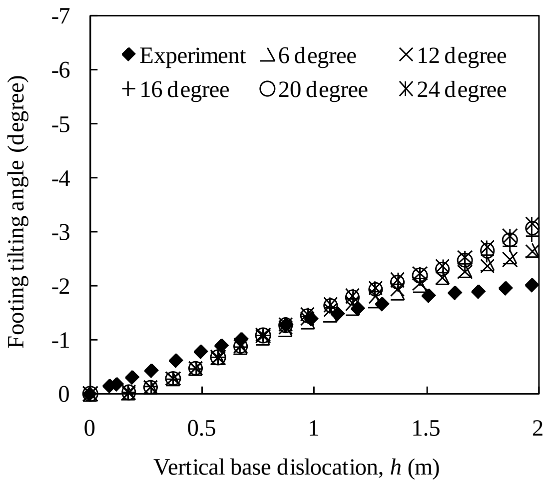


(f) h=-1.1 m

Fig.13 Comparison of experimental ground images (a to c) and shear strain plots (darkest region indicates strain equal or more than 20%) on deformed FE mesh (d to f) for Test14_R with heavy raft.



(a)



(b)

Fig.14 Comparison of experimental and numerical footing tilting variation as a function of base dislocation and interface friction: (a) Test 15 with light raft and (b) Test 14_R with heavy raft.

Table 3 Vertical base displacement, propagated dip angle and extension of soil deformation when fault rupture reached the ground surface.

Test name	*h* (m)		△ (°)		θ_{max} (°)	
	Exp.	FEM	Exp.	FEM	Exp.	FEM
Test12_R2	-1.04	-1.1	65	61	56.1	47
Test15	-1.0	-1.1	63	57.8	54	48.8
Test14_R	-0.98	-1.1	62.8	57.3	58.2	49.2

7. CONCLUSIONS

The normal fault rupture propagation in a sand layer and its interaction with existing rigid strong raft foundations in centrifuge model tests are successfully predicted by the calibrated FEM models. This has enabled the clarification of the shear band development and interaction with the rigid strong raft foundations in terms of the vertical displacements on the ground surface, the minimum vertical base dislocation for the main rupture to outcrop, the average propagated dip angle at failure, and the horizontal extent of the deformed ground surface. The results of the present study revealed the following:

(1) Light or heavy rafts do not affect the amount of fault displacement required to propagate the main shear band through the sand deposit.

(2) Rigid strong raft foundations are able to divert the fault rupture shear bands. However, these rafts suffer some tilting on the foundation soil. The tilting decreases markedly after the main shear band outcrops. The tilting of the raft foundation decreases as the raft foundation bearing pressure increases.

ACKNOWLEDGMENTS: The experimental portion of the present study was performed at the University of Dundee under the QUAKER research project, which is funded by the European Commission under contract number EVG1-CT-2002-00064. The authors would like to thank all of the members of the QUAKER research project at the University of Dundee.

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