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IDENTIFYING SAFE DRINKING WATER SOURCE FOR WATER SUPPLY IN LOHAGARA MUNICIPALITY, BANGLADESH

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ABSTRACT

Establishing sustainable water source is an indispensable component in any urban water supply scheme. The main objective of this study is to identify suitable water source for its potential utilization in urban water supply scheme in Bangladesh. Lohagara municipality of Narail district has been taken for the case study. Water demand for the municipality area is estimated up to the year 2040 based on the available population from census data. Mathematical models are developed to assess the potentials of surface water source and groundwater source against the existing and projected water demand. For identifying the potentiality of surface water source, the South-West Regional Model (SWRM) developed by Institute of Water Modelling (IWM), Bangladesh is applied. The flow simulation in the surface water model has been carried out by using MIKE 11 software. For developing groundwater model, the hydrogeological setting and aquifer demarcation in the municipality area is established by analyzing the individual lithology and developed hydrostratigraphical sections. The groundwater model has been constructed and simulated in the framework of an integrated MIKE-11 and MIKE-SHE software platform. The developed groundwater model covers 34 upazillas under south-west region of Bangladesh. From the simulation results and water quality data analysis, it appears that Lohagara municipality has high potentials of water supply from the surface water rivers and underground aquifers and both sources are sufficient to meet the water demand. Therefore, the present study concludes that existing surface water and groundwater sources can be successfully used for sustainable water supply in the Lohagara municipality under Narail district in Bangladesh.

Keywords: Aquifer, Groundwater model, Lohagara, MIKE-11, MIKE-SHE, Water Source

1. INTRODUCTION

Development activity in a country is currently associated with its water consumption. Globally, the demand of water grows as population and development activities increases. The condition is more complex particularly in the urban part of a country (Garcia et al., 2008). However, the increasing trend in water consumption in Bangladesh during the last years has been counteracted by a decrease in available water resources due to effect of climate change, mainly in summer. Urban population is increasing rapidly in the country as a result of natural urban growth and migration from rural areas. (Karim and Mohsin, 2009). According to BBS (2005), the current urban population is about 38 million and will be reached to about 74 million by 2035. This fact has put an obligation to establish an appropriate management policy that can ensure a continuity of both water resources as well as development activities. It is also highlighted in the National Water Polity of Bangladesh, which states that all sorts of required means and measures should be taken to manage water resources of the country in a comprehensive, integrated and equitable manner (NWP, 1999). In Bangladesh, municipal water supply systems mainly depend on surface water (SW) and groundwater (GW) sources to meet the water demand. Water quantity and quality limitations of the sources often impose economic constraints on system operation requiring additional treatment cost including more expensive alternative sources in the system (Shah and Khan, 2008). Thus, management of urban water supply systems becomes difficult if there is a shortfall in water availability from the sources constrained by water quantity or quality limitations. However, water management in an area should be based on the appropriate knowledge of the available SW and GW resources (Garcia et al., 2008). Das Gupta et al. (2005) reported that most of the SW sources in Bangladesh is usually polluted and requires treatment prior to consumption. Therefore, identifying safe drinking water source is a major challenging task in Bangladesh. Lohagara municipality is the only urban area of Lohagara upazila in Narail district. It was established on 2000 and presently, it is classified as a “C-category” municipality. Its population have been increasing since its inception. Although the relative importance of the municipality has ever been growing as a regional centre of trade and commerce, it has no piped water supply scheme at present. Therefore, a study has been undertaken to establish the water supply network coverage for the year 2040 with the help of mathematical modelling technique. The overall objective of the present study is to study and establish the appropriate water source options for supplying drinking water to the dwellers of Lohagara municipality under Narail district in Bangladesh.

2. ESTIMATION OF WATER DEMAND

A comprehensive water demand assessment is performed in this study for the baseline year and is projected up to the year 2040. The method of demand assessment includes both spatial and non-spatial information. GIS-based map of the study area is the spatial data and non-spatial data input includes the demographic characteristics. GIS maps are prepared based on the topographic survey data conducted by IWM in 2010. However, demographic information are collected from the population census reports of Bangladesh Bureau of Statistics (BBS) for the years 1981, 1991 and 2001. For the baseline year 2010, demographic data are obtained from the social impact assessment (SIA) survey. The projection of population in the municipality is done and then per capita demand for growing cities is assigned to the population according to the land use category in the municipality area. This method is repeated for every five years from 2010 to 2040 and the outcomes are presented in Table 1.

Table 1: Projected attributes of Lohagara municipality

Description of attributes	2010	2015	2020	2025	2030	2035	2040
Growth rate	2.731	2.705	2.679	2.654	2.628	2.603	2.579
Population	29,347	33,537	38,277	44,626	50,807	59,342	67,398
Water demand (m^3d^{-1})	752	1105	1588	2524	3775	5645	8053
Water demand (m^3d^{-1}) + backwash for WTP	792	1168	1684	2685	4028	6043	8650

3. ASSESSMENT OF SURFACE WATER SOURCE

3.1 Assessment of Surface Water Availability

Assessment of SW availability for water supply in Lohagara municipality is carried out for potential sources from the nearby Upper Nabaganga River. The municipality is situated on the bank of the Upper Nabaganga River in the South West Region (SWR) of Bangladesh. The intake point of municipal water supply is located conveniently on the Upper Nabaganga River, which is a schematized river of the South West Region Model (SWRM) developed earlier by IWM. SWRM covers the major river networks of SWR that includes the administrative divisions of Barishal and Khulna. Data collection and updating of model results are carried out by IWM every year. In this study, the whole model is calibrated and validated by using 20 years of historical data and is used to predict water resource availability for supplying drinking water in Lohagara municipality. A sample of water level calibration for the Upper Nabaganga River is presented in Fig 1.

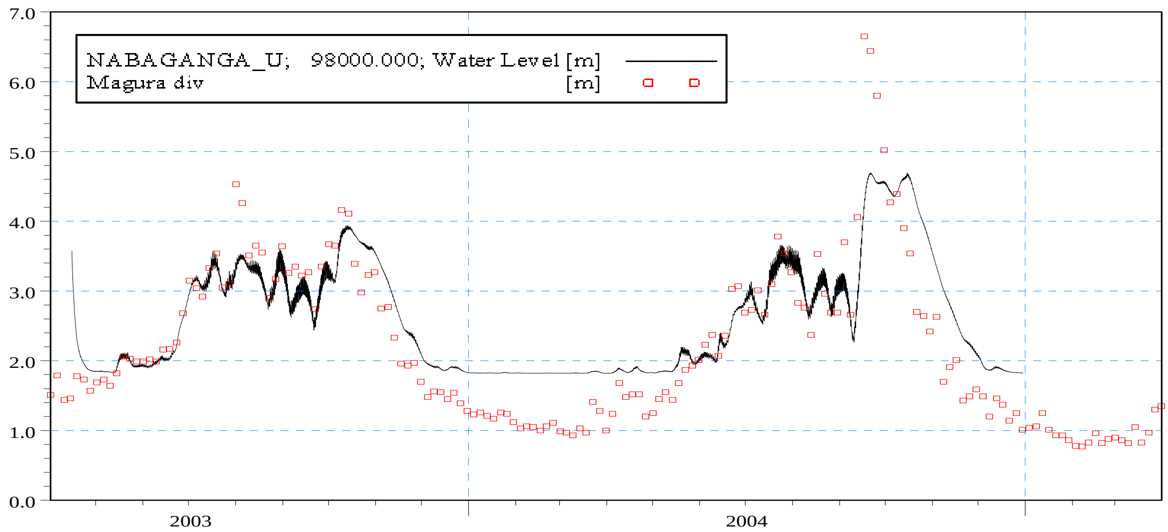

Figure 1: Calibrated water level of the Upper Nabaganga River at Magura

The Upper Nabaganga River is mainly a spill-channel from the Padma River, which drains the Gorai River basin. It has significant discharge in wet period, which decreases during dry season. However, simulation indicates that there is a continuous flow in the Upper Nabaganga River all through the year. The river has linkages with the Ganges through other spill channels but carry lesser discharges. Water availability assessment for the municipality is carried out from long-term (comprising of 20-years model run) simulated discharge of validated model available at IWM. The assessment is based on flow duration curve (FDC) developed (Figure 2) from simulation data for year round discharge at the selected station. The discharge at any percentage of probability in FDC represents the flow magnitude in an average year that can be expected to be equalled or exceeded. Dependable flows have been computed for year round analyses for the period of 1986 to 2009 for Lohagara municipality and are presented in Table 2. In this study, the Weibull flow duration method (Chow et al., 1988) is adopted for dependable flow analysis. This method determines the desired value (i.e. dependable flow) by ranking the daily flows in descending order and assigning each with an exceedence probability. The simple plotting system is expressed by the Weibull formula as given below:

$$P(X \geq x_m) = \frac{m}{n+1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

Where, m stands for the rank number from 1 to n and n is the number of ranked flows. This method has been chosen over several available plotting position formulas in statistics (Chow et al., 1988) because of its simplicity and easy adaptability to computers. The requirement of a water treatment plant (WTP) for its backwashing (Table 1) is estimated as 0.1001 m^3s^{-1} (8650 m^3d^{-1}). Therefore, SW is sufficient to fulfil the water demand of the dwellers in Lohagara municipality under Narail district of Bangladesh.

Table 2: Dependable flows of the Upper Nabaganga River at Lohagara

Location	River	Dependable flow for the dry season (m^3s^{-1})		
		50%	80%	90%
Lohagara	Upper Nabaganga	7.1	4.5	3.1

Required SW withdrawal for WTP use in Lohagara municipality	= 0.1001 m^3s^{-1}
80% dependable flow (from available water) of the Nabaganga River	= 4.5000 m^3s^{-1}
Available water in the Nabaganga River after withdrawal for WTP use	= 4.4000 m^3s^{-1}
Exploitable flow (60% of 80% dependable flow)	= 2.7000 m^3s^{-1}
Environmental flow requirement (40% of 80% dependable flow)	= 1.8000 m^3s^{-1}
SW availability for the dwellers in Lohagara municipality	= 233280 m^3d^{-1}

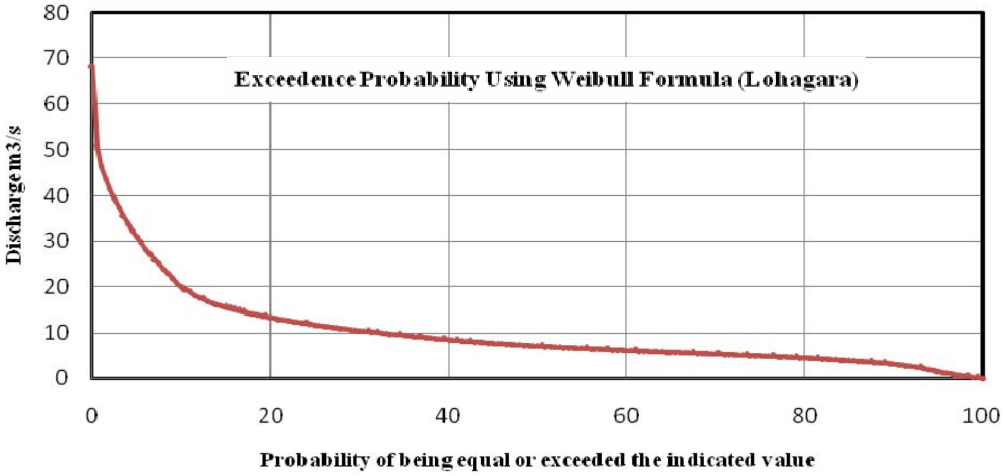

Figure 2: Flow duration curve for the Upper Nabaganga River at Lohagara

Table 3: Summary of findings for surface water (SW) source assessment

(a) Projected population and water demand			(b) Availability of drinking water supply from SW source		
Projected population in 2010	Nos	23028	Existing nearby River/Khal	--	Upper Nabaganga
Projected population in 2025	Nos	38277	Dependable flow	m^3s^1	4.5
Projected population in 2040	Nos	59342	Exploitable flow	m^3s^1	2.7
Water demand in 2025	m^3d^{-1}	2524	Surface water availability	m^3d^{-1}	233280
Water demand in 2040	m^3d^{-1}	8053	Duration of SW availability	month	12

The estimated results of water demand in Lohagara municipality and available flow in the nearby SW source (Upper Nabaganga River) have been assessed for establishing a sustainable water supply scheme and presented in Table 3. The analysis indicates that the Upper Nabaganga River located near Lohagara municipality can be a safe and dependable source for sustainable drinking water supply in the municipality dwellers. However, it is emphasized that water quality must be taken into consideration to establish the conclusion regarding safe source of water supply in the municipality.

Table 4: Measured river water quality of the Nabaganga River at Lohagara

Sl.	Parameters	Unit	Allowable Limit for Drinking Water (ECR, 1997)	Measured Value Pre-Monsoon
1	pH	-	6.5-8.5	7.2
2	BOD_5 at 20^0C	mgL^{-1}	0.2	3
3	COD	mgL^{-1}	4	< 4
4	Turbidity	NTU	10	1.19
5	NO_3-N	mgL^{-1}	10	0.45
6	NO_2-N	mgL^{-1}	< 1.0	< 0.016
7	TDS	mgL^{-1}	1000	238
8	TSS	mgL^{-1}	10	6
9	PO_4-P	mgL^{-1}	6	0.35
10	Cr_{Total}	mgL^{-1}	0.05	0.011
11	Pb	mgL^{-1}	0.05	0.057
12	Cl^-	mgL^{-1}	150-600 (maximum 1000)	153
13	SO_4-S	mgL^{-1}	400	27

3.2 Assessment of Surface Water Quality

Water quality assessment (Table 4) has been carried out based on the primary water sampling data collection session at the selected locations on the Upper Nabaganga River near Lohagara municipality. The sampling sessions are performed twice in the study at 12:00 noon. The first sampling mission was carried out in May 2011 (pre-monsoon period), when the water quality is at the most deteriorated level. However, there was no flow in the river at the time due to which samples were not taken. The second mission was carried out in September 2011 (monsoon period), when the water quality is at an improved state due to significant flows in the river. The results indicate that river water can be a safe drinking water source for municipal water supply in Lohagara municipality.

4. ASSESSMENT OF GROUNDWATER SOURCE

4.1 Hydrogeological Setting and Delineation of Aquifer System

In Bangladesh, GW is usually considered as an acceptable source than SW because its quality is usually in the range of drinking water standard and treatment cost is also less. In this study, GW source assessment has been performed by two major sub-components such as hydrogeological studies and GW modelling. Hydrogeological investigation has been carried out to define the hydrostratigraphic layers in Lohagara municipality. Sub-surface lithological characterization and formation of hydrostratigraphic units have been produced by analyzing the individual lithological units and depth of different aquifers from the available eight lithological borelogs in the study area. However, maximum depth of the available borelog is found as 305m. IWM customized software “depth-storage” model is used for producing hydrostratigraphic columnar sections and determining specific yield of the sediment formations. The columnar sections indicate that a productive aquifer exits below the municipality with the alteration of fine sand aquifer, aquitard and aquiclude layers. The analysis of individual lithology and columnar sections demonstrates 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. However, developed hydrostratigraphic layers reveal 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 individual hydrostratigraphic unit are summarized in the Table 5.

Table 5: Summary of hydrostratigraphic units and their extents in the study area

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

4.2 Groundwater Modelling

For the purpose of GW source identification and resource assessment, large numbers of hydrogeological and meteorological data have been collected. For hydrogeological study and GW resource 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 (Anderson and Woessner, 1992). However, GW model setup involves a geometrical description and specification of physical characteristics of the hydrological system of the area under consideration. In this study, the model has been developed using

MIKE-SHE mathematical modelling software tool, developed by DHI Water and Environment Pty Ltd. 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. However, in the present study, the model domain covers an area of about 9,582 sq. km., which includes Meherpur, Kushtia, Chuadanaga, Jhenaidah, Magura, Rajbari, Faridpur and part of Gopalganj, Narail & Jessore districts of Bangladesh. The study area has been discretized into 1 km square grids in the horizontal plan and 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 SW and GW system 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. All forms 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. For river-aquifer dynamic flow exchange, leakage coefficients along with the hydraulic conductivity of the SZ are taken into account for most of the river reaches. The developed model is then calibrated for the period of 1997 to 2003. During calibration phase, overland leakage coefficient, vertical hydraulic conductivity, storage coefficient and river leakage coefficient have been adjusted. In the present model, calibration is performed against observed GW level and a total of 62 observation wells have been used for the calibration and validation purposes. In order to increase the reliability of the model, it is verified based on another set of data, which is taken as 2004 to 2007. After successful calibration and validation of the developed model, it is used for GW simulation and resource assessment purposes.

4.3 Assessment of Groundwater Resources

In this study, GW resource has been estimated based on the well-known GW fluctuation technique as well as GW balance study on the basis of long-term simulation. 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 (i): if thickness of first layer exceeds 6m or 7m depth, entire saturated thickness lies only in first layer.
- Case (ii): if thickness of first layer remains above GW level, entire saturated thickness lies only in second layer.
- Case (iii): if case (i) & case (ii) 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.

According to GW level fluctuation approach, saturated thickness of 1st and 2nd layers are multiplied by the corresponding specific yield values and summed up to find out the depth of available water in a model grid. GW availability in volumes is estimated by multiplying the depth of water availability with the area of the grid. Now, total available GW resource is estimated based on the number of grids lying within the study area. Finally, GW availability has been assessed for two different depths in the study area. The result shows that GW resources for Lohagara municipality under two different depths are found as 11.29 Mm^3 for 6m depth and 13.81 Mm^3 for 7m depth, respectively.

In general, the depth of 6m and 7m is used for the unconfined aquifer to calculate the available GW resources within the limit of suction mode pump. However, in case of long-term sustainability of the

GW resources for safe water supply as per required quantity, the deeper aquifer is found more reasonable. Keeping this fact in mind, long-term simulation is executed for the developed GW model and a water balance for the whole model domain has been performed. The net GW recharge for the deeper aquifer is estimated as net recharge = 0mm (from upper layer) – 23mm (for outflow) + 413mm (for inflow) = 390mm, which mostly comes from the horizontal flow. For this amount of GW recharge, the corresponding annual aquifer storage volume is estimated by multiplying the aquifer catchment area and found as 7.25 Mm^3.

4.4 Groundwater Quality Assessment

For GW quality study, a test well was constructed screening the target aquifer so that samples could be collected from the target aquifer. In this regard, it may be mentioned here that the exploratory drilling at Lohagara is completed and test tube well has been installed. However, GW sample collection is not done and result is not received. For this reason, data available from DPHE for a private well of Barba mouza of Lohagara is available and presented in Table 6. After analyzing the groundwater quality data it is evident that Barium, Boron and Manganese are of higher concentration in the GW sample than allowable limit of Bangladesh standard. However, some parameters of GW quality in deeper aquifer of Lohagara test tube well under this study are currently available. This set of GW quality data of Lohagara municipality test tube well of depth 600 feet is given below. By analyzing the data, it is found that:

- Iron (1.11 mgL^{-1}) is slightly higher than allowable limit
- Potassium (15 mgL^{-1}) is slightly higher than allowable limit
- Total dissolved solids (TDS) is not matching with total concentration of major ions
- Ionic balance is -23.71% (beyond acceptable limit)

Table 6: Water quality results of collected groundwater samples

SL	Water Quality Parameter	Unit	Allowable Limit (ECR, 1997)	Water Quality Result	
				DPHE Well	GWMP Well
1	Arsenic (As)	mgL^{-1}	0.05	0.05	0.004
2	Barium (Ba)	mgL^{-1}	0.01	0.05	----
3	Chromium (Cr)	mgL^{-1}	0.05	<0.02	----
4	Aluminium (Al)	mgL^{-1}	0.20	0.04	----
5	Boron (B)	mgL^{-1}	1.0	30	0.4
6	Bicarbonate (HCO_3)	mgL^{-1}	----	----	255.0
7	Calcium (Ca)	mgL^{-1}	75.0	65.6	61.0
8	Carbonate (CO_3)	mgL^{-1}	----	----	250
9	Chloride (Cl)	mgL^{-1}	150-600 (max 1000)	----	387
10	Cobalt (Co)	mgL^{-1}	----	<0.008	----
11	Copper (Cu)	mgL^{-1}	1.0	0.008	----
12	Fluoride (F)	mgL^{-1}	1.0	----	0.46
13	Iron (Fe)	mgL^{-1}	0.3-1.0	0.077	1.11
14	Potassium (K)	mgL^{-1}	12.0	1.1	15.0
15	Lithium (Li)	mgL^{-1}	----	0.004	----
16	Lead (Pb)	mgL^{-1}	0.05	----	0.022
17	Magnesium (Mg)	mgL^{-1}	30-35	30.7	29.0
18	Manganese (Mn)	mgL^{-1}	0.10	1.11	<0.05
19	Nitrate (NO_3)	mgL^{-1}	10.0	----	0.6
20	Sodium (Na)	mgL^{-1}	200	139	198
21	Phosphorus (P)	mgL^{-1}	0.0	0.4	1.61
22	Silica (Si)	mgL^{-1}	----	18.9	----
23	Sulphate (SO_4)	mgL^{-1}	400	6.6	9.0
24	Strontium (Sr)	mgL^{-1}	----	0.336	----
25	Vanadium (V)	mgL^{-1}	----	<0.006	----
26	Zinc (Zn)	mgL^{-1}	5.0	0.008	3.1

From the water quality analysis, it appears that almost all the parameters have been found within the allowable limit of drinking water standard of Bangladesh. However, little treatment is necessary for very few parameters that cross the allowable values. Therefore, GW source can be used as a potential source for municipal water supply.

5. CONCLUSIONS

In this study, assessment of the existing SW and GW source in the Lohagara municipality has been performed to investigate the potential source for municipal water supply. For the proposed water supply scheme in Lohagara municipality, the projected water demand is calculated as 8650 m^3d^{-1} for the design year 2040. The analysis and result indicates that both SW and GW sources have required amount of water resources. At the same time, level of quality in these sources is satisfactory for drinking water supply. However, if GW source is to be used for water supply, little treatment scheme is needed for few parameters to ensure the high standard. Finally, it is emphasized that regular monitoring of the water quality parameters must be ensured for safe and sustainable water supply in the Lohagara municipality under Narail district of Bangladesh.

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WASTE WATER USE IN AGRICULTURE: A PILOT STUDY IN RAJSHAHI CITY CORPORATION AREA

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ABSTRACT

Day by day, scarcity of water increasing is a prime consideration. In many developing countries wastewater is the only source of water for irrigation during drought periods. Besides, wastewater is high in nutrient content and requires less chemical fertilizers for crop cultivation. After a certain period it will be unable to irrigate with ground water because of its limited quantities and contamination. To overcome this difficulties waste water may be a suitable source to meet the irrigation demand if properly managed and utilized in convenient way. Water samples were collected from four places of Rajshahi City Corporation for determining water quality parameters. Analysis of these impurities were performed with strict quality control and quality assurance using the internationally accepted analytical chemical methods for the determination of their harmful effect on crops and found that TDS, TSS, Na, N, P, EC & SAR exceeds the permissible limit in the first three sample location. The major problem in using wastewater in agriculture in the project area is the high level of faecal contamination. The advantages of waste water irrigation is that high cropping intensity, less fertilizer are required, fast growth, cheaply available but some problem are arises that damage to the pump, mosquito spread and farmers are affected with various water related diseases such as skin rashes, scabies, dysentery, typhoid, malaria etc.

Keywords: RCC, Waste Water, FAO, WHO, BOD.

INTRODUCTION

Rajshahi District (Zilla), which covers an area of 2407 km^2, of which 62 km^2 is river, is located in the north west of Bangladesh bordering India to the south (BBS 1993). Rajshahi City Corporation (RCC), which was formed in 1987, covers an area of approximately 48 km^2 being bounded on the east, north and west by Paba Thana (subdivision of a district) and on the south by the Padma River.

Rainfall is limited and poorly distributed over the year in Rajshahi District and in drought season ponds, lacks, river becomes dry and then waste water becomes main source of irrigation. Ground water table during dry season becomes lower and difficult for cost effective pumping. In some areas salinity is a major problem that greatly hampered soil structure and crop yield. There is a possibility of arsenic contamination for deep pumping of groundwater which finally have a tendency to enter into the food chain by cultivating crop with it. In contest with surface and groundwater waste water are available in dry season in Rajshahi City Corporation drain and competition among the farmers will be observed for using waste water. Wastewater contains beneficial crop nutrients as well as suspended organic materials, microorganisms, and in some cases, heavy metals. Wastewater contains many nutrients needed for plant growth including nitrogen, phosphorus, potassium, zinc, and copper. If 30

cm of municipal wastewater were applied in the crop field, this would translate to an N loading of 30 to 60 kg/ha/yr, P loading of 6 to 18 kg/ha/yr, K loading of 15 to 120 kg/ha/yr (Alberta, 2000). A study was carried out in Rajshahi City by the Wastewater Agriculture and Sanitation for Poverty Alleviation in Asia also known as WASPA Asia project on wastewater irrigation and agriculture practices, which was conducted for the Comprehensive Assessment program of the International Water Management Institute (IWMI). The objective of the project was to improve the livelihoods of urban and peri-urban farmers who are using wastewater in agriculture; and the communities who are responsible for producing the wastewater or consuming the agricultural produce (Jayakody et al., 2007).
The main objectives of present study is to measurement of waste water parameters which is harmful for agriculture at field level and compare their test values with the standard FAO and WHO guidelines. To evaluate the potential harmful effect of waste water contaminants on crops yield and the process of reducing this harmful effect for optimum crop yield. To inspire the farmers to treat waste water by cost effective natural biological treatment process for successful application of waste water to meet irrigation demand and other facilities.

MATERIALS AND METHODS

Selection of Sample Location

A number of storm water drains flow from the south of Rajshahi City Corporation to the north either terminating in beels or in the Baranai River some 15 km away. An initial assessment of the city was conducted at the beginning of the study and four project locations were identified where waste water is being used in agricultrue. These sites are situated along two of the city drains. Circuit House drain also known as Basuar Beel drain because it drains through Bashuar Beel and Dargapara drain, also known as the Cantonment drain because it flows through the Cantonment agricultural area. In Ward 16, second drain joins the Dargapara Drain bringing untreated industrial effluent from the Bangladesh small and Cottage lndustry Corporation (BSCIC) area and meets at Sopura also known as BSIC drain. In Ward 3, the BSCIC Drain and the CITY Drain meet finally flows on Basuar Beel and is continuously used for agricultural purposes. The sites selected are listed in Table 1 and depicted in the map [Fig. 1].

Table 1 : Water Quality Monitoring Site Codes and Descriptions

Site Code	Sample Location Points in Rajshahi City Corporation
A	BSCIC Drain Sopura
B	CITY Drain
C	Combination of BSCIC and CITY Drain
D	Basuar Beel

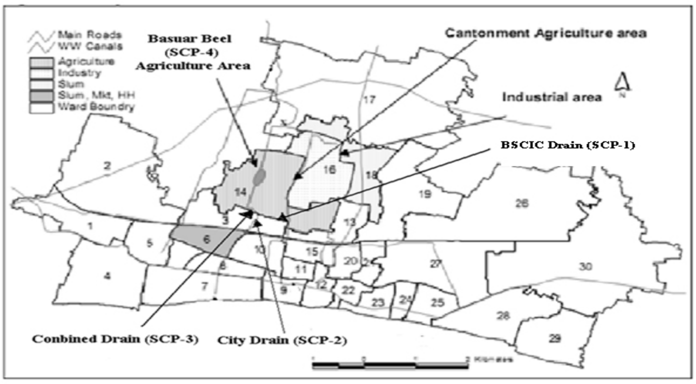


Fig.1: Sample collection point

Sampling Methodology

The analysis were performed with strict quality control and quality assurance using the internationally accepted analytical chemical methods of analysis. Measurements of all of the water quality parameters were conducted at Public Health Lab in RUET. The methods which was carried out during testing are listed below.

Table 2 : Water Quality Parameters Measurement Method

Water Quality Parameters	Methodology Reference
pH	Colorimetric or Electrometric method
Total Solids, Dissolved Solids and Suspend Solids	APHA 2540 C, APHA 2540 D
Biochemical Oxygen Demand (BOD)	APHA 5210 B
Chemical Oxygen Demand (COD)	APHA 5220 B
Dissolved Oxygen (DO)	APHA 4500 O G
Sodium (Na)	EDTA titration method
Magnesium (Mg)	EDTA titration method
Calcium (Ca)	EDTA titration method
Total Nitrogen (N)	Titanium Trichloride Reduction Method (Method 10021)
Total Phosphorus (P)	PhosVer 3 with Acid Persulfate Digestion (Method 8190)
Potassium (K)	APHA 3111 B
Total Coli form	Membrane Filtration Method

COST EFFECTIVE NATURAL BIOLOGICAL TREATMENT PROCESS

Waste water Stabilization Ponds

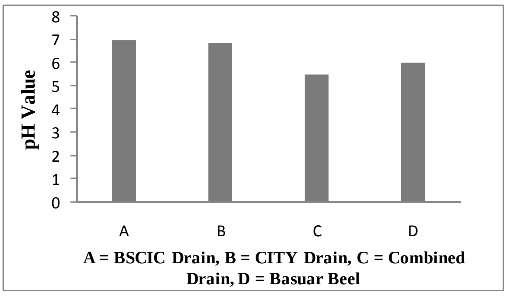
Waste Stabilization pond are shallow, usually rectangular earthen basin, open to the sun and air, in which raw waste water is treated by natural processes based on the activities of both algae and bacteria. The processes which may take place simultaneously include sedimentation, oxidation, digestion, synthesis, photosynthesis, endogenous respiration, gas exchange, aeration, evaporation, thermal currents and seepage (Ahmed and Rahman, 2000). Depending on the organic loading rate and the climatic condition and with respect to the presence of oxygen, waste water stabilization pond system can be classified as anaerobic ponds, facultative pond and maturation pond. An anaerobic pond is essentially a digester that requires no dissolved oxygen, since anaerobic bacteria break down the complex organic wastes. Anaerobic ponds are very cost effective for the removal of BOD, when it is present in high concentration as less than 1000 mg/l BOD_5. For high strength industrial wastes, up to three anaerobic ponds in series might be justifiable but the retention time t_{an}, in any of these ponds should not be less than 1 day (McGarry and Pescod, 1970). The BOD removals in Anaerobic Ponds Loaded at 250 g BOD_5/m^3d were achieved 50%, 60% and upto 70% after retention period 1, 2.5 and 5 days respectively (Mara, 1976). A facultative pond is one in which there is an upper aerobic zone (maintained by algae) and a lower anaerobic zone. The effluent from a facultative pond treating municipal sewage in the tropics will normally have a BOD_5 between 50 and 70 mg/l as a result of the suspended algae. Retention time in a properly designed facultative pond will normally be 20-40 days and, with a depth of about 1.5m, the area required will be significantly greater than for an anaerobic pond. (McGarry and Pescod, 1970). Each type of pond are effective in reducing the biodegradable organic matter and destruction of faecal pathogen. A maturation pond is one in which aerobic bacteria break down the wastes and algae, through photosynthesis, provide sufficient oxygen to maintain an aerobic environment. The effluent from facultative ponds treating municipal sewage or equivalent input wastewater will normally contain at least 50 mg/l BOD_5 . For sewage treatment, two maturation ponds in series, each with a retention time of 7 days, have been found necessary to produce a final effluent with BOD_5 < 25 mg/l when the facultative pond effluent had a BOD_5< 75 mg/l. Maturation ponds are effective in reducing the pathogen concentration.

Macrophyte Pond Treatment

Floating macrophyte species, with their large root systems, are very efficient at nutrient uptaking. In tropical regions, water hyacinth doubles in mass about every 6 days and a macrophyte pond can produce more than 250 kg/had (dry weight). Nitrogen and phosphorus reductions up to 80% and 50% have been achieved respectively. Water hyacinth acts as brushing of suspended solids in the water and cleans it. When wastewater passes through the channel covered with waterhyacinth, grass, crop fields is purified and intensity of pollution reduced. Water Lily has an extensive root system with rapid growth rates. It is an ideal plant for water treatment systems in warm climates. Duckweed (Lemma spp.) and Pennywort (Hydrocotyl spp.) has a good capacity for nutrient absorption. The leaves of the aquatic plants contain high nutrient (nitrogen and phosphorous) and they may be used as fertilizer at a later time.

RESULT AND DISCUSSION

The industrial and commercial units along main roads within the city and also by-roads in the city centre dispose of their waste water to the main drainage canals which finally discharge into the basuar beel drainage basin.



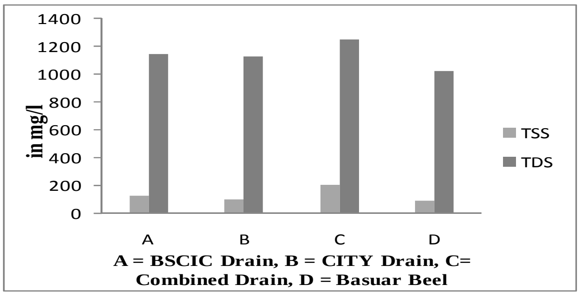


Fig.2: Variation of pH in the project location

Fig.3: Variation of TSS & TDS in the project location

The pH range suitable for the existence of most biological life is quite narrow and critical, and standard range for irrigation water is pH 6.5-8.5. The pH value in the project location was found mildly acidic to neutral [Fig.2]. The reason may be use of various chemical in industry where acidogenesis reaction take place for goods production that further slightly decrease the pH limit.

The Food and Agriculture Organization (FAO) has developed guidelines for the evaluation of water quality for irrigation and suggests that there need be: no restrictions on the use of irrigation water with an TSS and TDS concentration of less than 100 mg/l and 450 mg /l respectively; slight to moderate restrictions if concentrations are in the range 100-200 mg /l and 450 – 2000 mg /l; and severe restrictions for irrigation water with an TSS and TDS concentration of more than 200 mg/l and 2000 mg /l respectively (Ayres and Westcot, 1985). TDS & TSS was found higher in BSCIC Drain and City Drain [Fig.3]. The reason may be waste water from industry, hospitals, vehicle service stations and slaughter houses and meat stalls, hotels and restaurants, schools, technical colleges and tution classes were identified as significant sources of City waste water that hardly accumulate in the area.

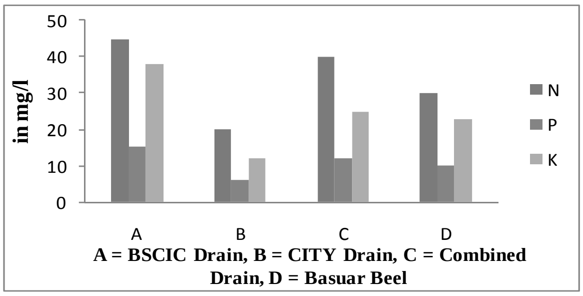


Fig.4: Variation of TSS & TDS in the project location

Nitrogen, phosphorous and potassium are necessary primary macronutrient for plants that stimulates plant growth. Wastewater with a total nitrogen and phosphorous concentration of 80 mg/ l and 20 mg/l is considered strong, 40 mg /l and 10 mg/l as medium and 20 mg /l and 6 mg/l as weak respectively (Pescod, 1992). If excess nitrogen is applied to a crop it can result in over-stimulation and excessive growth which increase susceptibility to pest and disease attacks, delayed maturity, failure to ripen, reduced crop quality and yield loss and excess phosphorus can lead to noxious algal blooms in water bodies (Pescod, 1992). In the BSCIC Drain and Combined Drain total N and total P concentration increases because of new sources of phosphorus and nitrogen entering the drain may be: more domestic waste; detergents from the industrial area and leaching from the agricultural area, especially if fertilizer is over applied. The concentrations declined in the beel and are even lower in the beel effluent; this may be due to natural treatment processes taking place or dilution. Generally irrigation water contains low potassium concentrations, insufficient to cover the plant's theoretical demand. The normal concentration of K in treated wastewaters is 30 mg /l and is 0-2 mg /l in irrigation water (Pescod, 1992; Ayres and Westcot, 1985). The reported K values for the project area are in the range of 12 to 38 mg/ l and the highest reported value is for the most downstream location on the BSCIC Drain [Fig.4].

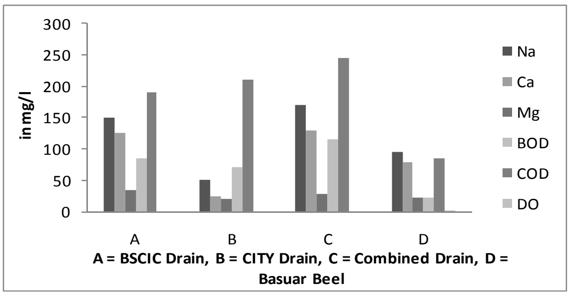


Fig.5: Variation of Na, Ca, Mg, BOD, COD & DO in the project location

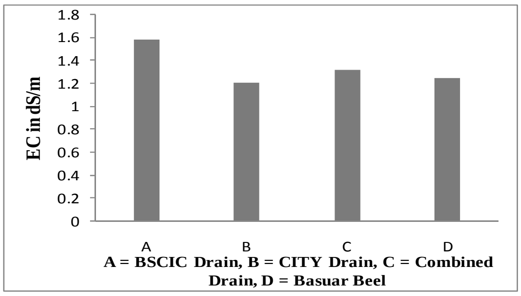


Fig.6: Variation of EC in the project location

The value of Na, Ca & Mg in the project location was found in the range of 95-150 mg/l, 80-125 mg/l & 23-35 mg/l respectively [Fig. 5]. The source for this may be during industrial process soda lime, detergent and various salts of sodium are use that further increases the Na in the waste water. The relative concentration of sodium to other cations is determined by the sodium adsorption ratio (SAR). When present in its exchangeable form sodium changes the physico-chemical properties of the soil and has the ability to disperse soil particles that results in reduced air and water infiltration to the soil and the formation of a hard crust when the soil is dry (Pescod, 1992). The EC of irrigation water is important because it is a measure of the salinity of the water. According to FAO no restrictions on the use of irrigation water with an EC of 0.7 dS /m, slight to moderate restrictions if concentrations are in the range 0.7 – 3.0 dS/ m and severe restrictions for irrigation water with an EC of greater than 3.0 dS /m (Ayres and Westcot, 1985). The reported SAR values for the project locations were between 13.5 and 14.8. The FAO guidelines suggest severe restrictions for irrigation water with an EC <1.3 within this range of SAR, which were recorded for locations B and D. The EC values for BSCIC Drain were >1.3 and the FAO guidelines therefore suggest moderate restrictions [Fig. 6]. The BOD, COD and DO values in the project location ranges from 22-115 mg/l, 85-245mg/l and 0.5-2.5 mg/l respectively. According to WHO (2006) municipal wastewater with BOD, COD and DO concentration in the range of 110-400 mg /l, 250-1000 mg/l and 4.5-8.5 mg/l can increase crop productivity. The value of DO is lower in the project location.The reason for that presence of organic matter is higher in three locations and for decomposition of this organic matter more oxygen is required.

The WHO guidelines for wastewater used in agriculture had a maximum Faecal Coliform of less than 1000 thermotolerant coli/100 ml for root crops likely to be eaten uncooked, and 10,000 thermotolerant coli/100 ml for leaf crops likely to be eaten uncooked but no standard for irrigation of cereal crops (WHO 2006). The results for Total and Faecal Coliform in all tested drain water samples did not comply with the WHO guidelines for use of wastewater in agriculture (WHO 2006).

CONCLUSION

On the basis of test results in this study, it is concluded that the biological quality parameters in wastewater which is used in agricultural purposes are not satisfied with the FAO and WHO guidelines. The most concerning factor is that, the wastewater in the study area having high level of faecal contamination. The TDS, TSS, Na, N, P, BOD, EC and SAR value in the study area is also high. So, waste stabilization pond with addition of gypsum and macrophytes pond can be effectively used to reduce the harmful effect of these parameters for proper irrigation.

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A STUDY ON PERFORMANCE ANALYSIS OF BASIN TYPE SOLAR STILL FOR WATER DESALINATION

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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). The BSS is a simple solar device used for converting salt /brackish water into potable water and the developing countries can employ this technology for provision of clean drinking water especially in areas where this is in short supply. A performance evaluation was carried out on an inclined still that was fabricated using locally available materials. The main parts of the still are the ferro-cement basin, plastic troughs, input bottles, glass cover, styrofoam, output bottle and support. Inner surface and bottom of the basin were covered with styrofoam which acts as heat insulator after that a glass cover of thickness 5 mm was provided at the top to make it airtight. The still was tested using black painted troughs which act as the heat absorbent to distillate yield was recorded. In this study, we have found that average production rate and unit price of distillated water of BSS is 3.45 lit/m^2/day (1.139 lit/day) and Tk. 0.34 per lit respectively. However, average production rate and unit price of distillated water of TSS is 3.95lit/m^2/day (0.29 lit/ day) and Tk.0.54 per lit respectively. It is concluded that the result presents clear information to understand the behavior of production rate of BSS. Nevertheless, a comparative analysis is also included to verify the reliability of BSS over TSS.

Keywords: Basin type solar still, Tubular type solar still, Desalination, Distillation, Production rate

INTRODUCTION

The shortage of drinking water is expected to be the major problem of the world in this century due to unsustainable consumption rates and population growth. Oceans constitute about 97.5% of the total global water, and the remaining 2.5% fresh water is present in the atmosphere, polar ice, and ground water. This means that only about 0.014% is directly available to human beings and other organisms (Bendfeld et al., 1998). So, development of new clean water sources is imperative.

Desalination of sea and/or brackish water is an important alternative option, since the only unlimited source of saline water. The term “Seawater Desalination” comprises all the processes that remove dissolved salts from seawater with the aim of obtaining water having a low content of dissolved salts and impurities for use as drinking water for human needs, industry and agricultural use. Desalination process is divided with and without phase change. In the developed countries, the most well-known and viable technique (without phase change) of desalination is Reverse Osmosis (RO) and Electrodialysis (ED) (Hiersig, 1995) which are not only energy intensive but also uneconomical.

Now-a-days, researchers are initiating to prove cost-effective desalination process (with phase change) such as Muti-Stage-Flash-Evaporation (MFS) (Naffey et al., 2006), Multi-Effect-Distillation (MED) (Fiorini et al., 2007), Thermal or Mechanical Vapor Compression (TVC, MVC) (Mabrouk et al., 2007), Solar Distillation (Alarcon-Padilla et al., 2007). However, the developments in the use of solar energy have demonstrated that it is ideally suited for desalination, when the demand of fresh water is not too large. So, BSS is feasible options for providing drinking water for a single house or a small community in coastal regions is the simple distillation technique comparing with others.

In this study, a Basin Type Solar Still (BSS) has been designed and constructed to evaluate desalination rate of BSS and compare cost with Tubular Solar Still (TSS) using principle of solar distillation and the basic heat and mass transfer relations.

MATERIALS AND METHODS

Solar still with operation and maintenance

The BSS is a rectangular shaped still of which components are basin, troughs, input bottles, glass cover, styrofoam, output bottle and support. Effective size of the still is 85 cm x 67.50 cm x 25 cm and made of ferro-cement using locally available materials. It is placed with an inclination of 15° to the horizontal. It is composed of six rectangular plastic troughs which are equally in size of 72.50 cm x 8.00 cm x 3.75 cm for storing saline water. Interior surface of these troughs is blackened to enable absorption of solar energy to the maximum possible extent. Glass which is transparent material with 5 mm thick is used as top cover to make it airtight. It permits the passage of solar energy into the trough which heats up the water in the troughs. The inclination of the still cover is selected so as to let the water droplets slide down on the inner surface of the glass to the drainage channel made of plastic. Silicon jell is used to seal any space between the glass and wall to ensure that vapors are not lost to the atmosphere. Inner surface of the basin were covered with 3.80cm thick Styrofoam which acts as heat insulator. One side of the still is drilled in no. of six feeding brackish water into the troughs partially. On the one side of the section, there is a U-shaped channel for the collection of the distillated water into collection bottle which is also insulated by styrofoam to protect of evaporation. One end of the still is elevated supporting a brick wall. Every morning a little amount of raw water (saline) was supplied into the troughs by the input bottles. Sometimes two or more times in day period was needed to supply the raw water in the troughs while it gets dry. Little amount of raw water is essential to occur evaporation quickly. After sun setting, volume of distillated water was measured (in ml) by empting the collection bottle and recorded. Sediment of salt gathers at the bottom of the troughs when water gets evaporated. The sediment of salt should be removed from the troughs to increase production rate of distillated water. Thus, field experiment was conducted from 7th January to 22nd March, 2009.

Production Principle

Production principle of BSS is illustrated in Fig. 1. Solar distillation produces clean water as simply as nature makes rain. 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 begins to evaporate i.e. energy from the sun heats water inside the still to the point of evaporation. Many types of heat transfer occur inside the glass and outside the glass e.g. evaporative heat transfer from the saline water to the cover, convective heat transfer from the saline water to the cover, convective heat transfer from the saline water to the trough, convective heat transfer from the cover to the atmosphere, radiative heat transfer from water surface to glass cover and from glass cover to the atmosphere. Water vapor rises, condenses on the inner glass surface of the still, releases its latent heat of vaporizing. The condensed water trickles down along the inner surface glass cover due to gravity and is stored in a collection bottle through a pipe provided at the lower end. This process removes impurities such as salts and heavy metals as well as eliminates microbiological organisms. The end result is ultra-pure water cleaner than the purest rainwater.

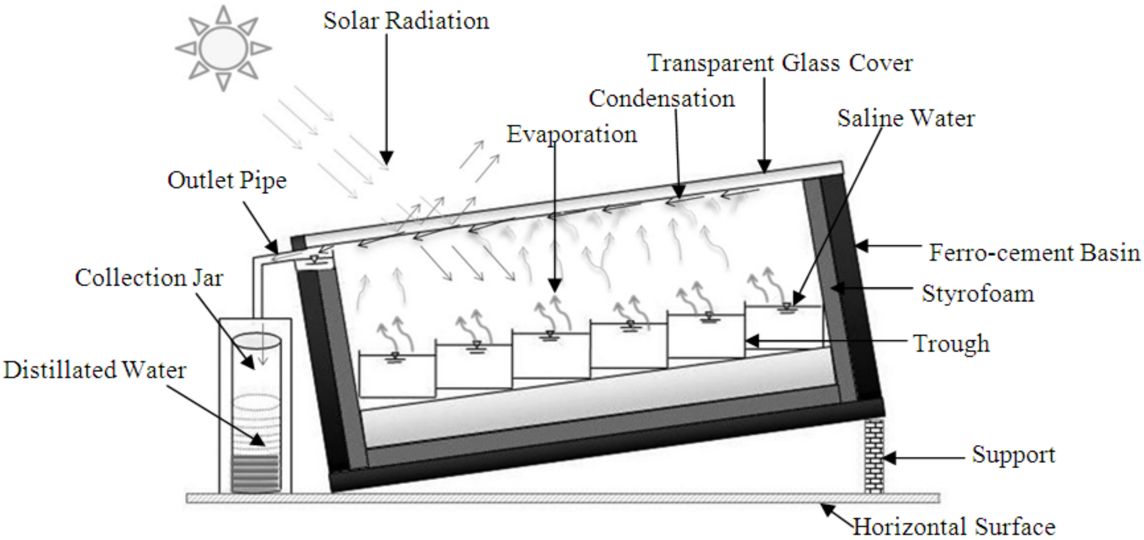

Fig. 1: Production principle of BSS

The most important parameter affecting the output of a solar still is, obviously, the intensity of the solar radiation incident on the still. If Q_t (Joules/m^2 day) is the amount of solar energy incident on the glass cover of a still and Q_e (Joules/m^2 day) is the energy utilized in vaporizing water in the still, then the daily output of distilled water M_e (kg/m^2 day) is given by

$$M_e = \frac{Q_e}{L} \Rightarrow Q_e = LM_e \tag{1}$$

Where, L (Joules/kg) is the latent heat of vaporization of water. The efficiency η of the still is given by

$$\eta = \frac{Q_e}{Q_t} = \frac{LM_e}{Q_t} \tag{2}$$

It is worthwhile to note that the efficiency of a typical basin-type solar still is quite low and is not greater than 35% (Malik et al., 1982).

Mathematical Formulation

Mass and energy balances of still

Assumptions:

The mass and energy balance equations are made up on the following assumptions:

i) Heat and mass transfer in still are formalized using the representative temperature of the saline water, trough and glass cover.

ii) Water vapors near the water and condensation surfaces are saturated.

iii) There is no water vapor leakage across the glass cover.

iv) Heat capacities of cover and trough are negligibly small in comparison that of water.

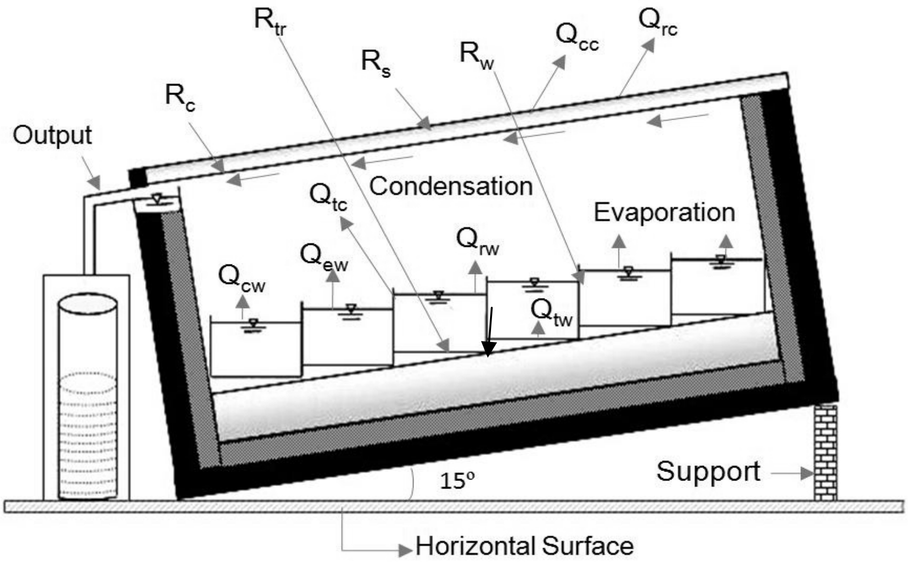

Fig. 2: Mass and heat transfer of BSS

Q_{cc} = Convection between cover and atmosphere
Q_{cw} = Convection between cover and water surface
Q_{ew} = Evaporation between cover and water surface
Q_{rc} = Radiation between cover and atmosphere
Q_{rw} = Radiation between cover and water surface
Q_{tw} = Convection between trough and water
Q_{tc} = Convection between trough and cover
R_c = Solar radiation absorbed by cover
R_s = Solar radiation
R_{tr} = Solar radiation absorbed by trough
R_w = Solar radiation absorbed by water

Mass Balance Equation

The mass balance of the saline water in a trough can be written as:

$$\frac{dh_w}{dt} = -\frac{m_{evap}}{\rho_w} \tag{3}$$

Energy Balance Equations

The energy balance equations of the BSS for the different components may be expressed as follows:

Saline water: $$(\rho CA)_w \frac{\partial (h_w T_w)}{\partial t} = R_w + Q_{tw} - Q_{ew} - Q_{cw} - Q_{rw} \tag{4}$$

Trough: $$(\rho CV)_{tr} \frac{\partial T_{tr}}{\partial t} = R_{tr} - Q_{tw} - Q_{tc} \tag{5}$$

Cover: $$(\rho CV)_c \frac{\partial T_c}{\partial t} = R_c + Q_{ew} + Q_{cw} + Q_{rw} + Q_{tc} - Q_{cc} - Q_{rc} \tag{6}$$

RESULT AND DISCUSSION

Production rate of distillate water

In this experiment, we have found the average distillation rate 3.45 Lit/m^2/day for this type of still, BSS. During the experimental period, it was observed that production rate of distillated water was maximum due to higher intensity for long day period in month of March'09 than previous month. On the other hand, the rate is minimum in January'09 due to water gets heated through lower intensity of solar radiation with foggy day period. However, in context of Bangladesh, month of January stays in winter season and in this period, atmosphere is covered with densely fog which needs more time to be removed. The character of these production rates differs due to fluctuation day by day from 7th January to 22nd March, 2009. The fig. 3 shows the production rate of distillated water in month of February, 2009. The rate of production of distillated water is influenced by some factors such as intensity of solar radiation, wind velocity adjacent the glass cover, humidity, temperature of water, cover and air, salt concentration remaining at the bottom of troughs, color of troughs, depth of water in the troughs and space between water surface and inner side of glass cover.

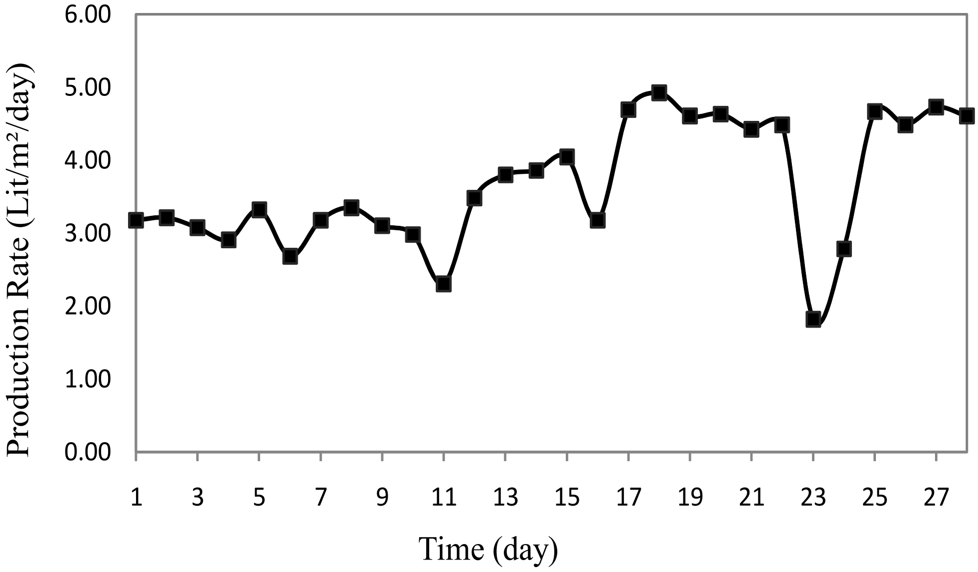

Fig. 3: Production rate of distillated water from BSS (February, 2009)

Cost of BSS

The material cost of the BSS for construction is calculated as Tk. 1000.00 which has been detailed in Table 1.

Table 1: Material cost of the BSS

Sl. No.	Prticulars	Unit	Rate (Tk.)	Quantity	Amount in (Tk.)
1.	Ferro-cement basin				
	Cement	kg	7.00	20	140.00
	Sand	cft	50.00	1	50.00
	GI net	sft	15.00	10	150.00
	GI wire	rft	1.25	40	50.00
2.	Trough	nos.	30.00	6	180.00
3.	Glass	sft	40.00	7	280.00
4.	Miscellaneous	--	--	--	150.00
				Total	1000.00

Cost Comparison with TSS

The material cost of BSS and TSS is Tk. 1000.00 and Tk. 400.00 respectively considering design life 10 years. The average production rate of BSS and TSS is 3.45lit/m^2/day (1.139 lit/day) and 3.95lit/m^2/day (0.29 lit/ day). If we consider that in a year 70% time one can get the calculated average daily output, then

a) In case of BSS:

Total Production of water in the design life= 1.139×365×10×0.7=2910 lit.

Production cost of water = (1000÷2910) = Tk. 0.34 per lit

b) In case of TSS:

Total Production of water in the design life= 0.29×365×10×0.7=740.95 lit

Production cost of water = (400÷740.95) = Tk. 0.54 per lit

In fact there is an increased demand for solar distillation development for environmental, ecological and economical reasons as it provides the viable option for providing hygienic potable water. The advantage of the solar distillation is attributed to simple operation and use of solar energy that are environmentally friendly and operating costs are too low.

CONCLUSION

It is concluded that the daily production rate of BSS is lesser than that of TSS. But the cost per liter of distillated water of TSS is greater than BSS. It is clearly understood that BSS is cost effective than TSS considering economic point of view. Since 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 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 BSS is a better solution to meet the small scale fresh water demand. It is recommended that a hybrid BSS could be designed using an additional solar panel to increase the saline water temperature that in a trough that would also increase the distillate production rate.

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BANGLADESH NATURAL DISASTER CHRONOLOGY AND TREND ANALYSIS

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ABSTRACT
Bangladesh is recognized worldwide as one of the country's most vulnerable to the impacts of climate change and climate variability. This is due to its unique geographic location, dominance of floodplains and low elevation from the sea, high population density, high levels of poverty, and overwhelming dependence on nature, its resources and services. The geographical setting of Bangladesh makes the country very much prone to natural disasters. The mountains and hills bordering almost three-fourths of the country, along with the funnel shaped Bay of Bengal in the south, have made the country a meeting place of life-giving monsoon rains, but also make it subjected to the catastrophic ravages of natural disasters. Its physiographic and river morphology also contribute to recurring disasters. Abnormal rainfall and earthquakes in the adjacent Himalayan range add to the disaster situation. Effects of El-Nino-Southern Oscillation (ENSO) and the apprehended climatic change have a great impact on the overall future disaster scenarios. Moreover some mountains in Asia have permanent glaciers that have vacated large areas during the past few decades, resulting in increases in glacial runoff. As a consequence, an increased frequency of events such as mudflows and avalanches affecting human settlements has occurred. The paper mainly focuses on current natural disaster chronology and also analyses the trend of Bangladesh disaster. The chronological assessment and trend analysis would ultimately help and support to disaster risk reduction.

Keywords: Morphology, catastrophe, ENSO, chronology etc

INTRODUCTION

Bangladesh is located between 20^{o} to 26^{o} North and 88^{o} to 92^{o} East. It is bordered on the west, north and east by India, on the south-east by Myanmar, and on the south by the Bay of Bengal. Most of the country is low-lying land comprising mainly the delta of the Ganges and Brahmaputra rivers. Floodplains occupy 80% of the country. Mean elevations range from less than 1 meter on tidal floodplains, 1 to 3 meters on the main river and estuarine floodplains, and up to 6 meters in the Sylhet basin in the north-east.

Bangladesh has a humid, warm, tropical climate. Its climate is influenced primarily by monsoon and partly by pre-monsoon and post-monsoon circulations. The south-west monsoon originates over the Indian Ocean and carries warm, moist, and unstable air. The monsoon has its onset during the first week of June and ends in the first week of October, with some inter-annual variability in dates. Besides monsoon, the easterly trade winds are also active, providing warm and relatively drier circulation. In Bangladesh there are four prominent seasons, namely, winter (December to February), Pre-monsoon (March to May), Monsoon (June to early-October), Post-monsoon (late-October to November). The general characteristics of the seasons are: (i) winter is relatively cooler and drier, with the average temperature ranging from a minimum of 7.2°C to 12.8°C to a maximum of 23.9°C to 31.1°C. The minimum occasionally falls below 5°C in the north though frost is extremely rare. There is a south to north thermal gradient in winter mean temperature: generally the southern districts are 5°C warmer than the northern districts; (ii) pre-monsoon is hot with an average maximum of 36.7°C, predominantly in the west for up to 10 days, very high rate of evaporation, and erratic but occasional heavy rainfall from March to June. In some places the temperature occasionally rises up to 40.6°C or more. The peak of the maximum temperatures are observed in April, the beginning of pre-monsoon season. In pre-monsoon season the mean temperature gradient is oriented in southwest to northeast direction with the warmer zone in the southwest and the cooler zone in the northeast. (iii)Monsoon is both hot and humid, brings heavy torrential rainfall throughout the season. About four-fifths of the mean annual rainfall occurring during monsoon. The mean monsoon temperatures are higher in the western districts compared to that for the eastern districts. Warm conditions generally prevail throughout the season, although cooler days are also observed during and following heavy downpours. (iv)Post-monsoon is a short-living season characterized by withdrawal of rainfall and gradual lowering of night-time minimum temperature. The mean annual rainfall is about 2300mm, but there exists a wide spatial and temporal distribution. Annual rainfall ranges from 1200mm in the extreme west to over 5000mm in the east and north-east.

NATURAL DISASTERS OF BANGLADESH

The mountains and hills bordering almost three-fourths of the country, along with the funnel shaped Bay of Bengal in the south, have made the country a meeting place of life-giving monsoon rains, but also make it subjected to the catastrophic ravages of natural disasters. Its physiographic and river morphology also contribute to recurring disasters. Abnormal rainfall and earthquakes in the adjacent Himalayan range add to the disaster situation. Effects of El-Nino-Southern Oscillation (ENSO) and the apprehended climatic change have a great impact on the overall future disaster scenarios. Since Bangladesh is a disaster prone country, it experiences frequent natural disasters, which cause loss of life damage to infrastructure and economic assets, and adversely impacts on lives and livelihoods, especially of poor people. Bangladesh is susceptible to floods, tropical cyclones, storm surges, and droughts. Among the disasters, flood and tropical cyclones are highly remarkable.

Floods in Bangladesh

Flood is mainly generated outside Bangladesh due to rainfall in the catchment areas of the Ganges, the Brahmaputra and the Meghna and also due to melting of snow in the Himalayas. Simultaneous rainfall over the three basins may cause abnormally high flood. Excessive rainfall within Bangladesh may aggravate and contribute to devastating floods in Bangladesh. Their total catchment area is approximately 1.6 million sq-km of which only about 7.5% lies in Bangladesh and the rest, 92.5% lies outside the territory. It is assumed that an average flow of 1,009,000 Million cubic meters passes through these river systems during the monsoon season. The Brahmaputra (Jamuna) river above Bahadurabad has a length of approximately 2,900 km and a catchment area about 5,83,000 sq-km. Started from the glaciers in the northernmost range of the Himalayas and flows east far above half its length across the Tibetan plateau. Total length of the Ganges River is about 2,600 km to its confluence with the Brahmaputra -Jamuna at Aricha-Goalondo and a catchment area of approximately 9, 07,000 sq-km. Started from the high western Himalayans glaciers, the Ganges has a short mountain course of about 160 km. The Meghna system originates in the hills of Shillong and Meghalaya of India. The main source is the Barak River, which has a considerable catchment in the ridge and valley terrain of eastern Assam bordering Myanmar. The rivers of Meghna basin are steep, highly flashy rivers, originating in one of the wettest area of the world, the average annual rainfall at Cherrapunji at Assam being about 10,000 mm.

Different types of flood are found in Bangladesh. Flash flood is found in the eastern and northern rivers with sharp rise followed by a relatively rapid recession with high velocities. Local flood happens due to localized rainfall of long duration in the monsoon season often generate water volumes in excess of the local drainage capacity. More than 50 mm or above rainfall in one day causes stress on local drainage system leading to localized flood and 300 mm or more rainfall in consecutive 10 days impedes the drainage and likely to cause rain-fed flood in the area. Major flood basically found in major Rivers (specially the Ganges-Brahmaputra-Meghna GBM river system). It generally rise and fall slowly; rise and fall may extend from 10-20 days or more and spilling through distributaries and over the bank of the river system.

Considering the area of inundation, a normal flood is considered when less than 28000km^2 is inundated throughout the country and low lying areas where cropping pattern adjusted to inundation and flooding is accepted every year. The frequency of high flood is around 3year and inundates 1/4th of the country. Area of inundation is considered in between 28000 km^2 to 36000km^2. Abnormal flood is found once in six years and area of inundation is considered in between 36000km^2 to 50000km^2. More than 50000km^2 area of inundation is considered as catastrophic flood and happens once in nine years. Each year in Bangladesh about 26,000 km^2, (around 18%) of the country is flooded. Abnormal/High Flood (flood that submerges more than 20% of the total area of BD) inundates vast area of Bangladesh Such abnormal flood in living history occurred in 1954, 1955,1962, 1974, 1982, 1987, 1988, 1998, 2004 and 2007.There are 10 big floods in just 50 years' time. It means that one abnormally high flood may occur once in every 5-6 years interval. Historical flood peaks and discharge in different major stations of the rivers of GBM system and areas of inundation are shown in the table below.

Table 1: Historical peak in flood major stations and area of inundation (% of total area)

River	Station	RHWL* (m PWD)	DL** (m PWD)	Peak of the year (m PWD)				
				1987	1988	1998	2004	2007
Brahmaputra	Noonkhawa	28.10	27.25	26.72	-	27.35	26.65	27.91
	Chilmari	25.06	24.00	24.56	25.04	24.77	24.51	24.81
Jamuna	Bahadurabad	20.62	19.50	19.68	20.62	20.37	20.04	20.40
	Serajganj	15.12	13.35	14.57	15.12	14.76	14.81	14.95
Ganges	Pankha	24.14	21.50	-	-	24.14	21.21	21.71
	Hardinge Bridge	15.19	14.25	14.80	14.87	15.19	13.69	14.00
Padma	Goalundo	10.21	8.50	9.52	9.83	10.21	9.89	10.01
	Bhagyakul	7.58	6.00	6.99	7.43	7.50	7.26	7.15
Meghna	Bhairab Bazar	7.66	6.25	6.91	7.66	7.33	7.78	6.94
Area of Inundation (% of total area of Bangladesh)				39%	61%	68%	38%	42%

* Recorded Highest Water Level (datum in meter PWD) ** Danger Level (datum in meter PWD)

Data source: Bangladesh Water Development Board

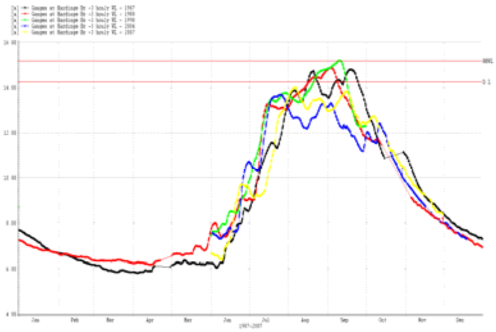
Table 2: Historical flood peak discharge value

River	Station	Peak discharge value (m^3/s)				
		1987	1988	1998	2004	2007
Jamuna	Bahadurabad	67200	68700	102535	96105.5	-
Ganges	Hardinge bridge	76000	70400	73091	37706	54217
Meghna	Bhairab Bazar	15200	19400	14670	10571	-

Data source: Bangladesh Water Development Board

All the rivers of Brahmaputra basin starts rising in March due to snow melt in the Himalayas which causes a first peak in May and early June. It is followed by subsequent peaks up to the end of August, caused by a heavy monsoon rain over the catchment. The response to rainfall to relatively quick, resulting in rapid increases in the river. Six to ten days elapsed from a period of rainfall in the upper catchment until the corresponding is felt within Bangladesh. It is a heavy alarm for Bangladesh if the water level of Brahmaputra rises may cause a severe flood in Bangladesh. The Ganges begins to rise in May and the period of maximum flow is well cantered in July and August. Occasionally September is the month of severe flooding. Floods in Ganges basin are mainly in the form of overbank spilling. The flood situation deteriorates when the Brahmaputra remains in space, transmitting backwater into the Ganges. The Padma, carrying the combined flow of the Brahmaputra and the Ganges, is more or less a straight channel with a relatively deep and narrow section. High water levels in the Meghna River are controlled downstream by the water levels of the Padma River during the flood season. When a peak stage of the Brahmaputra coincides with a peak stage of the Ganges and the Padma as well, heavy to severe flooding occurs. Figure 1 illustrates the hydrograph pattern for the flooding years in the three major points (Bahadurabad for the Brahmaputra; Hardinge Bridge for Ganges and Bhairab Bazar for Meghna) of Gangers-Brahmaputra-Meghna basin in Bangladesh territory.


a


b

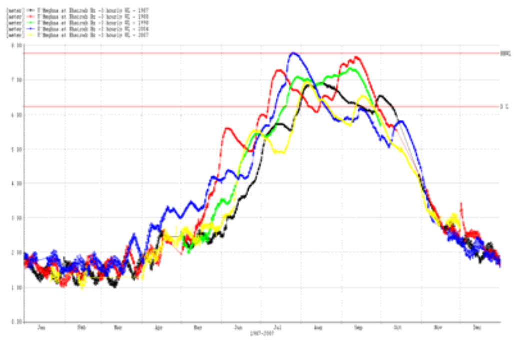

c

Figure 1: Hydrograph analysis for historical flood year (1987, 1988, 1998, 2004 and 2007)
a. Bahadurabad b.Hardinge bridge c.Bhairabbazar
Data source: Bangladesh Water Development Board

Tropical cyclones and storm surges

The Bay of Bengal is the breeding place of catastrophic cyclones. It is presumed that the Inter Tropical Convergence Zone (ITCZ), which is situated near the equator, and where winds from the two hemispheres meet, plays a vital part in the formation of the tropical cyclones in this area. A severe tropical cyclone hits Bangladesh, on average, every 3 years and generally form in the months just before and after the monsoon and intensify as they move north over the warm waters of the Bay of Bengal. They are accompanied by high winds of over 150 kmph and can result in storm surges up to seven meters high, resulting in extensive damage to houses and high loss of life to humans and livestock in coastal communities. A severe tropical cyclone hits Bangladesh, on average, every 3 years. These storms generally form in the months just before and after the monsoon and intensify as they move north over the warm waters of the Bay of Bengal. They are accompanied by high winds of over 150 kmph and can result in storm surges up to seven meters high, resulting in extensive damage to houses and high loss of life to humans and livestock in coastal communities. Table 3 shows the trend of cyclonic event near Chittagong and Cox's Bazar coast. Table 4 shows the total number of cyclone and depressions which crossed the different coasts in Bay of Bengal.

Table: 3 Cyclonic events in between Bangladesh coast (1970-2009)

Date	Landfall Area	Maximum Wind Speed (kmph)	Tidal Surge Height (ft)	Central pressure (mpa)
12.11.1970	Chittagong	224	10-33	-
28.11.1974	Cox's Bazar	163	9-17	-
15.10.1983	Chittagong	93	-	995
09.11.1983	Cox's Bazar	136	5	986
24.05.1985	Chittagong	154	15	982
18.12.1990	Cox's Bazar	115	5-7	995
29.04.1991	Chittagong	225	12-22	940
02.05.1994	Cox'sBazar- Teknaf	278	5-6	948
25.11.1995	Cox's Bazar	140	10	998
27.09.1997	Sitakundu	150	10-15	-
20.05.1998	Chittagong Coast near Sitakundu	173	3	
19.05.2004	Cox's Bazar – Akyab	65-90	2-4	990
15.11.2007	Sundarban	240	1.5-2	
18.04.2009	Chittagong- Cox's Bazar (near Chittagong)	120	-	988

Data source: Bangladesh Meteorological Department

Table: 4 Extreme of different cyclonic events formed in the Bay of Bengal in different coasts (1980-2009)

Year	WML	D	DD	CS	SCS	SCS(H)	Total
1980	-	1	2	3	-	-	6
1981	-	-	5	4	1	1	11
1982	-	6	-	-	3	1	10
1983	-	4	-	-	2	-	6
1984	-	1	1	-	1	2	5
1985	-	1	3	1	-	-	5
1986	-	3	-	-	-	-	3
1987	-	1	1	1	2	1	6
1988	-	-	1	1	-	1	3
1989	1	3	2	-	-	2	8
1990	-	5	1	-	1	1	8
1991	-	6	-	2	-	1	9
1992	-	1	2	2	-	1	6
1993	-	3	-	-	1	-	4
1994	-	1	-	-	1	1	3
1995	1	2	-	-	-	2	5
1996	-	3	-	4	-	1	8
1997	-	5	1	1	-	2	9
1998	-	3	-	1	-	2	6
1999	-	-	1	-	-	1	2
2000	2	-	1	3	-	1	7
2001	-	4	-	1	-	-	5
2002	-	2	-	2	-	-	4
2003	-	2	1	1	2	-	6
2004	-	1	-	1	-	-	2
2005	-	3	2	3	-	-	8
2006	-	6	1	-	-	1	8
2007	-	5	3	1	-	1	10
2008	-	-	1	1	-	-	2
2009	-	-	2	-	-	-	2

Data source: Bangladesh Meteorological Department

WML Well marked low; D Depression; DD Deep Depression; CS Cyclonic Storm; SCS Super Cyclonic Storm; SCS (H) Super cyclonic storm with a core of hurricane winds.

From 1970 to 2009, the total number of major cyclones striking Bangladesh was 26, where the number of occurrences increased significantly since 1990. It should also be noted that the highest number of affected people has been recorded after 1990. In 2007, the country was ravaged by Cyclone Sidr, which displaced 650,000 people and killed 3,447 (official record). In the year 2009, two cyclones hit (cyclone Bijli, April 2009, and cyclone Aila, May 2009). About 200,000 people were displaced by cyclone Bijli. The intensity of the damage caused by the cyclones in 2009 might not be as high as cyclone Sidr, but though the country was hit twice in the same year.

CLIMATE CHANGE AND NATURAL DISASTERS

Bangladesh is already experiencing the adverse impacts of global warming and climate change. The following impacts have been observed. Summers are becoming hotter, monsoon irregular, untimely rainfall, heavy rainfall over short period causing water logging and landslides, very little rainfall in dry period, increased river flow and inundation during monsoon, increased frequency, intensity and recurrence of floods. One-fifth of the country is flooded every year, and in extreme years, two-thirds of the country can be inundated. This vulnerability to flooding is exacerbated by the fact that Bangladesh is also a low-lying deltaic nation exposed to storm surges from the Bay of Bengal. It is found from the observed data that the temperature is generally increasing in the monsoon season (June, July and August). Average monsoon maximum and minimum temperature shows an increasing trend annually at the rate of $0.05^{O}C$ and $0.03^{O}C$, respectively. On the other hand average winter (December, January and February) maximum and minimum temperature shows decreasing and increasing trend annually at the rate of $0.001^{O}C$ and $0.016^{O}C$, respectively. It is found from IPCC model that there is a distinct extent of changes in precipitation pattern. About 12mm/month – 112mm/month change of rainfall is being predicted over next 100years. Sea level rise according to IPCC third summit, there might be a chance of 09 mm to 88mm whereas SMRC (SAARC Meteorological Research Centre) found the similar value of near about 100mm which assumes that that determines the vulnerability of Bangladesh to climate change impacts is the magnitude of sea level rise which may cause the increase flooding rate in Bangladesh.

CONCLUSION

It is found that development of Bangladesh is highly depended on biophysical, socioeconomic and institutional capacity those are at risk to natural disaster due to climate change. In order to maintain and enhance current thrust towards achieving sustainable development Bangladesh should find ways and means to reduce the threats of climate change, and increase resilience of the society and its physical systems to face the adverse impacts. Minimization of adverse impacts on crop agriculture; enhance resilience of vulnerable groups to extreme climatic events; minimize impacts on their property and livelihoods system; institutional capacity building; peoples participation; and integration of climate change issues in the sectoral development plan will assist sustainable development.

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PRESENT FLOOD FORECASTING LEAD TIME AND ITS IMPROVEMENT: THE CASE OF BANGLADESH

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ABSTRACT

Flood Forecasting is one of the most important non structural measures for flood disaster management. Bangladesh is lower riparian country of the Ganges Brahmaputra Meghna (GBM) basins. The total drainage area of Ganges Brahmaputra Meghna (GBM) basins is 1.75 million sq. km and the average annual water flow is 1350 billion cubic meters, which drains through Bangladesh but the GBM basin area within Bangladesh is only about 7% of the total area. So monsoon river floods in Bangladesh are caused by excessive discharge in the three major rivers coupled with retardation of outflow into the Bay of Bengal by high sea level and by excessive rainfall inside the country. So flood management is very complex issues. There are two aspects of flood management i) structural ii) non-structural measures. Flood forecasting is a major non-structural measures which can be used along with the structural measures for the better flood management. Flood forecasting of the major rivers is very important for the efficient flood management of Bangladesh. Flood Forecasting and Warning Centre (FFWC), Bangladesh Water Development Board (BWDB) under the Ministry of Water Resources (MoWR) provides flood warning upto up to 72 hours of 38 river stations (gauge locations). The current flood forecasting model is built on deterministic computational software developed by the DHI Water & Environment, Denmark. FFWC also provides 10 days forecast 18 river stations under CFAN project which is based on the probabilistic model. As per of satisfactory result of Climate Forecast Application Network (CFAN) probabilistic model, it is going to be expanded to 38 river stations for the long lead forecast in Bangladesh. To manage flood and its impact for disaster risk reduction properly with longer lead time is always desirable. GMB Basin Model can play a significant role in increasing lead time. Presently FFWC has taken initiative to increase its lead time by introducing prediction model based on the GBM Basin Model. The paper focuses on the existing flood forecasting lead time and its performance in terms of accuracy. This paper also highlights the major steps taken to improve its present lead time to medium range forecast.

Key word: Flood Forecasting, lead time, CFAN, Basin Model, Prediction model.

INTRODUCTION

Bangladesh is located in the North-eastern part of South Asia Stretching between latitude 20°34´ N and 26°38´ and longitudes 88°01´ and 92°41´ E . The country is the lowest riparian of the three great rivers of the world, the Ganges, Brahmaputra and Meghna. With a limited land area of about 147,540 sq-km, the country provides drainage route of a huge catchment area, which over 92.5% of its land

mass (NWMP -2000). Its Flood Management is complex in nature due to geographic location. High variability of spatial and temporal distribution of water resources causes many problems for the country. Too much water available during the monsoon where as water scarcity prevails during the dry period. Monsoon flood management is a challenge for Bangladesh for various reasons like food security; reduction of flood damages hence the economic development. There is some indication that flood hazard risks are changing due to natural and human–induced factors. Process related to human interventions include rapid urbanizations and unplanned development of flood plains, soil erosions due to over tilling and certain flood management activities. There are also upper riparian interventions such as flood embankments, damming of rivers and deforestation that are affecting river flows. In additions, Global climate change could affect could be affecting hydrology and water resources of Bangladesh‘s river network, a vast part of the country's surface is lower than 6 meters above sea level , which may ultimately result in more seriously flooding in future.
Among the various aspects of flood management, flood forecasting is very efficient tools for flood management. Only construction approaches like embankment, dam can not be used for total flood management purposes. Moreover, it requires huge investment. On the other hand flood forecasting is simple tool which can be easily applied with little efforts. Considering the whole aspect of complexity of floods in Bangladesh, the Government of Bangladesh has taken both the initiatives for effective flood management. FFWC is the national institution responsible for flood forecasting activities. It provides forecasts and warnings to many national level organizations.
The objective of the study is to analysis the existing flood forecasting lead-time and initiatives for its improvements.

PRESENT HYDRO METEOROLOGICAL DATA COLLECTION AND FLOOD FORECASTING SYSTEM

The flood forecasting technology and tools comprise of their core components, which includes data acquisition network, flood forecasting/modelling systems and the dissemination of forecasts and warning.
The basis of Flood Forecasting and warning is the monitoring of rainfall and water level measurements and the data at filed stations are used to both interpret the present food situation and also to predict future flooding. The data acquisition process can be divided into four separate parts and these are

- A near real time monitoring system under BWDB covering Bangladesh with which rainfall and water level measurements are collected.
- A data exchange agreement with India through which FFWC obtains rainfall and water level in India.
- Meteorological data from Bangladesh Meteorological department (BMD)

The data acquired by FFWC are processed and stored in a database, from where they are accessed by numerical models which form the basis of flood forecasting and warning systems. The flood watch system integrates the data with the modelling system.
MIKE 11, a one dimensional river modelling system developed by DHI Water & Environment, Denmark has been used FFWC to compute water level and discharge in the river systems. FFWC produce s water level forecasts at 38 gauge stations on different rivers up to 72 hours. (FFWC, 2011).

MEDIUM AND LONG-RANGE FLOOD FORECASTING INITIATIVES

FFWC took initiative to introduce medium and long range probabilistic forecast with the technical support from Atmospheric and Oceanic Sciences (PAOS) at the University of Colorado/Georgia Institute of Technology (GATECH), Atlanta USA. The initiative is known as "Climate Forecast Application in Bangladesh" (CFAB) and funded by the USAID office of Foreign Disaster Assistance (USAID/OFDA). The task of the PAOS /GATECH group was to increase the lead-time of flood forecasting in Bangladesh, while the task of ADPC was to identify broader forecast application opportunities and identifying ways to institutionalize CFAB in Bangladesh (ADPC and PAOS, 2004). The project was to develop and evaluate three tier overlapping forecast system with improved lead time during monsoon season 2003 and 2004, which showed a success in forecasting the discharges at Hardinge Bridge station of Ganges and Bahadurabad station of Brahmaputra rivers of Bangladesh. The three tier forecast system is as follows:

- Short-range forecast of rainfall and river discharge in probabilistic form provided each day with 6-10 days lead-time.
- Medium range forecasts of average 5 day rainfall and river discharges , updated every days , with 20-30 day lead time and
- Seasonal outlook starting at the beginning of the monsoon season and updated each month, providing 1-6 months lead-time.

METHODOLOGY OF FORECASTING EVALUATION

Two statistical criteria considered for the performance evaluation of the forecast results are as follows:

- Mean Absolute Error, MAE
- Co-efficient of Determination, r^2

MEAN ABSOLUTE ERROR, MAE

MAE is the mean of the absolute difference between *Observed* and *Forecast* levels as shown in the following equation:

$$MAE = \frac{\sum_{i=1}^{n} |x_i - y_i|}{n}$$

Where,

$x_1, x_2 \ldots\ldots x_n$ are *Observed* water levels

$y_1, y_2 \ldots\ldots y_n$ are *Forecast* water levels

n is the number of *Observed/Forecast* levels

CO-EFFICIENT OF DETERMINATION, R^2

r^2 is the *Co-efficient of Determination* for the correlation of *Observed* and *Forecast* water levels and is given by the relation as show in the equation below:

$$r^2 = \frac{\left[\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})\right]^2}{\sum_{i=1}^{n}(x_i - \bar{x})^2 \sum_{i=1}^{n}(y_i - \bar{y})^2}$$

Where,

$x_1, x_2 \ldots\ldots x_n$ are *Observed* water levels

$\bar{x}$ is the average of *Observed* water levels

$y_1, y_2 \ldots\ldots y_n$ are *Forecast* water levels

$\bar{y}$ is the average of *Forecast* water levels

n is the number of *Observed/Forecast* levels

Table 1 : Scales used for performance evaluation

Sl. No.	Scale	Value
1	Good	MAE <= 0.15 meter & r^2 >= 0.9
2	Average	MAE <= 0.2 meter & >0.15 meter and r^2 >= 0.7 & <0.9
3	Not satisfactory	MAE <= 0.3 meter & >0.2 meter and r^2 >= 0.4 & <0.7
4	Poor	MAE <= 0.4 meter& >0.3 meter and r^2 >= 0.3 & <0.4
5	Very Poor	MAE > 0.4 meter or r^2 < 0.3

RESULT AND DISCUSSION

Deterministic flood forecasting model simulation is done for maximum 72 hours forecast period and forecasts are saved in the database at 24-hour and 48-hour and 72-hour intervals. Using the two statistical criteria (Mean Absolute Error and Co-efficient of determination) the results of forecasted water level were evaluated. For the present study the results of a few important water level gauge stations in 2010 flood period are taken. Table 3. 4 and 5 shows the forecast evaluation results of water level gauges for three days and seven days. Evaluation results of one and two days show around 90 percent accuracy level with the observed water level. As lead time is concerned so, only three days and higher lead-time is considered for the present study. The result shows upto three days the forecasted water level good match with the observed water level. Table 3 shows the statistics of 72 hours forecast performance based on MIKE 11 output. MIKE 11 output shows upto 3 days coefficient of determination is high and mean absolute error is relatively low with some exception for the flashy rivers water level gauges like Sylhet, Sunamganj of Surma River.

Flood Forecasting and Warning Centre follows ensemble approach using ECMWF weather prediction data in its model to generate 51 sets of discharge forecasts at Bahadurabad and Hardinge-Bridge on the Brahmaputra and the Ganges Rivers respectively. Based on this data FFWC’s model generates 10 days lead-time probabilistic forecasts for mean, upper bound and lower bound water level at 18 water level gauge stations of major rivers in Bangladesh. The statistics of forecast performance for 7 strategically important water level gauge stations have been presented through Table 4 and 5. It shows 3–day probabilistic forecast mostly satisfactory with high R^2 value and low mean absolute error (Table 4). In comparison with the deterministic forecast, the present probabilistic forecast has a good performance upto three days. If we consider higher lead-time for example more than 3 days performance of some water level gauge stations is satisfactory (Table 5).

Both the model output (Deterministic and Probabilistic) shows as lead time increases the accuracy (variation of forecast & observe value) decreases. This means that forecasts are the best at 24-hour interval followed by 48-hour interval and then 72-hour interval. There are many uncertainty is associated with the increasing lead-time for instance model boundary estimation.

Table 3: Statistics for 3 day (72- hour) forecast performance (Mike 11 output)

Sl. No.	Station	MAE (m)	R^2	Performance
1	Bahadurabad	0.23	0.76	Average
2	Bhagyakul	0.10	0.91	Good
3	Bhairabbazar	0.08	0.93	Good
4	Dhaka	0.12	0.85	Good
5	Goalondo	0.14	0.89	Good
6	Hardinge-BR	0.27	0.86	Average
7	Rajshahi	0.26	0.89	Average
8	Serajganj	0.19	0.83	Average
9	Sunamganj	0.26	0.67	Not satisfactory
10	Sylhet	0.41	0.72	Poor

Table 4: Statistics for 3-day (72- hour) Forecast Performance (Probabilistic forecast)

Station	Lower Bound		Mean		Upper Bound	
	MAE(m)	R^2	MAE(m)	R^2	MAE	R^2
Bhagyakul	0.11	0.89	0.08	0.88	0.10	0.85
Bhairabbazar	0.12	0.87	0.07	0.85	0.09	0.89
Dhaka	0.10	0.85	0.10	0.85	0.10	0.85
Goalondo	0.11	0.91	0.09	0.91	0.12	0.90
Serajganj	0.24	0.72	0.17	0.73	0.16	0.73
Sunamganj	0.23	0.50	0.19	0.45	0.20	0.57
Sylhet	0.18	0.38	0.19	0.30	0.15	0.42

Table 5: Statistics for 7-day Forecast performance (Probabilistic forecast)

Station	Lower Bound		Mean		Upper Bound	
	MAE (m)	R^2	MAE	R^2	MAE	R^2
Bhagyakul	0.22	0.74	0.14	0.62	0.25	0.17
Bhairabbazar	0.20	0.68	0.13	0.60	0.18	0.72
Dhaka	0.13	0.70	0.14	0.69	0.16	0.66
Goalondo	0.23	0.78	0.15	0.71	0.31	0.40
Serajganj	0.40	0.55	0.25	0.47	0.30	0.29
Sunamganj	0.25	0.28	0.31	0.26	0.21	0.30
Sylhet	0.26	0.29	0.32	0.27	0.22	0.29

RECENT INITIATIVE FOR THE IMPROVEMENT OF LEAD-TIME

Basin Model Development

FFWC has taken initiative for the development of GBM basin model under the Comprehensive Disaster Management Program-II. The model includes the sub-catchments out of Bangladesh in in the Ganges basin, the Brahmaputra basin and the Meghna basin. The model will be used for the flood forecasting purposes to increase the lead-time from 3 days to 5 days.

Extension of Probabilistic Model forecast output

Based on the success of previous medium and long range forecast, it is necessary to institutionalize the forecast schemes on operational basis. This will provide more benefits to the community people to fight against flood disaster. FFWC is going to implement the medium and long range forecast systems with the technical assistance form Regional Integrated Multi-Hazard Early Warning System (RIMES) for Asia and Africa. Under this initiative FFWC will extend the coverage of the forecast schemes to more area. This project will help to develop more robust modelling system, so that it provides better results.

CONCLUSION

At present 3 days and 10 days forecast are being by using deterministic and probabilistic forecast approaches respectively. The study shows that the forecast quality gradually deteriorated where forecast intervals moved further away from the time of forecast.

Short range forecast is used to evacuation and emergency planning where as medium and long range forecast is used for agricultural planning which is essential to build food security. As probabilistic model can play an important role to increase the present short-term lead-time to medium to long range lead-time, so more focus should be given on its improvement. A vast area of Bangladesh is vulnerable to flash flood and coastal flood, so separate models can be developed to address flash flood and coastal flood respectively. Improvement of data collection system as well as regional cooperation is also necessary for increasing lead-time.

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COST OF MAINTAINING ENVIRONMENTAL FLOW IN KONTO RIVER BASIN, INDONESIA

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ABSTRACT

A sequential multiple hydropower system in the Konto River, Indonesia where water is used in three hydropower plants one after another from a upstream reservoir named Selorejo is examined to understand the existing operation of the system. Environmental flow requirements for the river are estimated. Current operation policy is observed failing to ensure environmental flow requirements downstream to Selorejo reservoir which might cause environmental degradation and loss of ecosystem goods and services for the basin. Analyses are carried out to estimate the tradeoffs between achieving two basic criteria of economic efficiency and environmental protection. Since environmental flow is not a static choice, three different levels (low, medium and high) of environmental flow provisioning downstream to Selorejo are used in the simulation study to understand how the overall benefit changes due to change in environmental flow levels. Ensuring environmental flow causes reduction in power production and this reduction is directly proportional to the level of environmental flow provisioning; however, benefit from flowing water is not accounted in this study. Ensuring low, medium and high level of environmental flow results respectively 7, 16 and 25% power reduction compare to no environmental flow level (existing operation policy). The study provides an in-depth insight into the tradeoffs between benefit maximization and environmental protection through provisioning different levels of environmental flow in rivers, which will eventually help basin managers to manage the system in a more sustainable manner.

Keywords: water-use benefit; environmental flow; tradeoffs; Konto River, Indonesia

INTRODUCTION

Growing water demands and limits over supply augmentation often entail competition and conflicts among the water users. Situation aggravates when environmental flow (EF) is considered for the rivers. Environmental flow is the provision of certain amount of flow to maintain the river health. While encountering such challenges, society constantly seeks to maximize the value that the limited resources provide and efforts are therefore committed to utilize the available water resources efficiently and effectively. Along this line, multiple uses of water i.e. using the available water for more than one uses or in production systems is inevitable to produce more with less water (Khan, 2010). However, intensification of multiple use of water in the catchment may affect downstream flow both in terms of quality and quantity. Hence, there is a need to revisit the multiple use water management activities and environmental sustainability at a system or catchment level (Bakker and Matsuno, 2001).

This paper aims to demonstrate a multiple use system at the Konto River, Indonesia, where water from an upstream reservoir is sequentially used in three hydropower plants and then in an irrigation project. The used water is not redirected back to Konto main course. The paper further widens the analysis by checking whether the system provides enough water as EF downstream to reservoir and in case when EF demand is not satisfied scenarios are run with EF provision and loss in power production is examined.

STUDY SITE

The Konto River basin in east Java, Indonesia is actually a sub-basin of the Brantas river basin. Selorejo reservoir is at the upstream of the Konto river. The Konto at the downstream of Selorejo flows towards north-west and finally drains to the Brantas. The total basin area of the Konto is 687 km^2 which can be divided into two parts, upper (above Selorejo reservoir) and lower (downstream of Selorejo). Catchment area for the upper part is about 236 km^2 that comprises Konto river 148 km^2, Kwayangan river 12.5 km^2, Pinjal river 44.3 km^2 and Selorejo reservoir 31.3 km^2 (Solihah, 2011). Lower part of Konto basin has an area of 451 km^2. Three small tributaries are found meeting the Konto in this lower part, namely: Sambong (catchment area 3.13 km^2), Nogo (catchment area 1.35 km^2) and Nambaan (catchment area 3.69 km^2) (Solihah, 2011). Average annual rainfall in the basin is about 2,700 mm whereas annual average evaporation is about 1,470 mm. Two season are mainly observed, wet (November -April) and dry (May-October) in the region. Roughly 80% of the rainfall occurs in the wet season. The plains and delta consist of alluvial soils (silt, clay loams) well suited to paddy cultivation. The annual average temperature is 23.5^0C, with maximum monthly temperature of 24.5^0C in January, and a minimum temperature of 22.7^0C in July. Annual average relative humidity is 79.9%, with a minimum humidity of 75% in January, and maximum of 83% in September (Solihah, 2011).

Selorejo reservoir commissioned on 1970 is situated at the upper part of Konto and is equipped with a hydropower plant, namely Selorejo power plant. In addition to Konto river, the reservoir is feed by Pinjal and Kwayangan river. The capacity of the reservoir is about 40 MCM with a water surface area of 4 km^2. The Selorejo hydropower started running since 1972 has capacity of 4.5 MW and the design discharge of 14.8 m^3/s. The release from Selorejo hydropower plant was redirected to the Konto before the construction of Mendalan and Siman hydropower plants in 2003. However, currently the release from Selorejo hydropower is subsequently being used by another two power plants Mendalan and Siman respectively and then in Konto irrigation project. Figure 1 shows the schematic of the lower part of the Konto River (the study site) including the water uses.

The topography of the region resonates well in terms of getting considerable heads in building these two other hydropower plants, namely: Mendalan and Siman after the Selorejo plant. The plants are run-off-river type. The tail water elevation of the Selorejo power plant is 582.00 m a.m.s.l. Part of the release from Selorejo plant is taken using a 3.25 km long tunnel to Sekuli daily retention pond which stabilizes the discharge from Selorejo and supplies water to Mendalan hydropower plant having an installed capacity of 7.0 MW and design discharge of 8.5 m^3/s. The tunnel capacity that feeds Sekuli retention pond is 9.25 m^3/s. A 5.55 m^3/s capacity pipe sends back the extra discharge from Selorejo plant to Konto main stream. Water Elevation of the Sekuli pondage is 573.17 m a.m.s.l. and the tail-water elevation of Mendalan power plant is 422.90 m a.m.s.l. that creates an effective head of 150.27 m for Mendalan power plant (PJT-I, 2007).

The release from Mendalan power plant is carried by a 3.2 km long tunnel and sent to Siman retention pond where Siman Hydropower plant is installed with the capacity of 9.0 MW and with design discharge of 8.5 m^3/s. The effective head of Siman hydropower plant is 106.4 m (PJT-I, 2007). At the point of Mendalan sabo-dam (Sabo-dam is a kind of silt arresting dam); facility to divert 2 – 3 m^3/s flow to Siman pond is available to stabilize the power production from Siman plant. Effective head for Siman power plant is 106.4 m (Solihah, 2011). The water used in Siman power plant goes to Siman reservoir from where the Left- and Right-Konto irrigation projects get supply of irrigation water. The entire water resources is managed by the public company named PJT-I.

METHODS AND DATA

The system is simulated for the current operation policy using a spread sheet model and power production from each power plant is calculated. Based on communication with PJT-I, it is understood that the system is currently running with the aim of maximizing power generation. PJT-I provided all the required data for the analysis. The obtained data includes monthly inflow to and release from Selorejo reservoir, monthly power production from three power plants, monthly flow for the three tributaries of the Konto, namely: Sambong, Nogo and Nambaan for last ten years (1999 - 2008). Power production from Mendalan and Siman plants were available for last six years (2003 – 2008) since both of the plants came into operation on 2003. Storage-area-elevation for the Selorejo reservoir was also collected from PJT-I database.

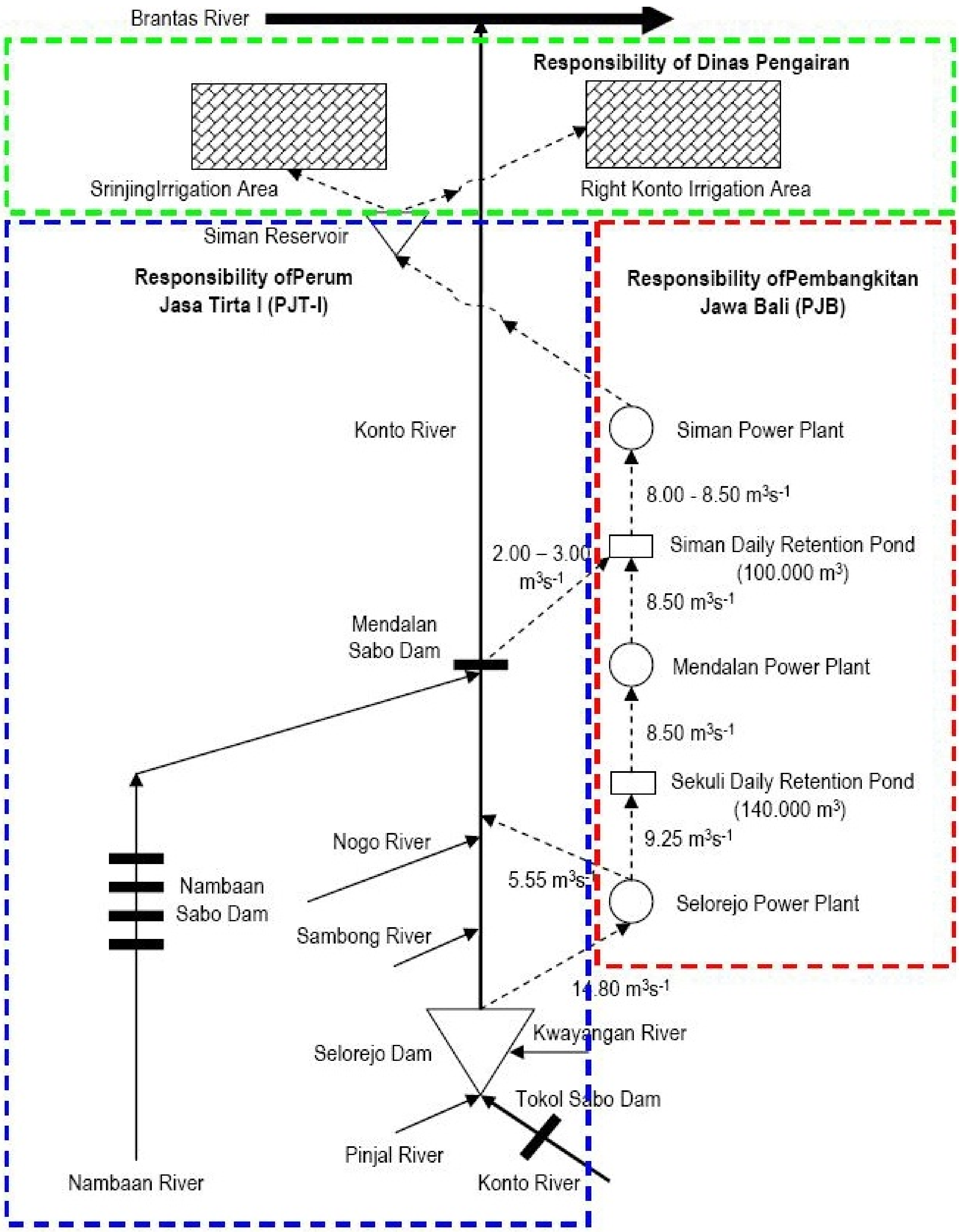

Figure 1 Schematic of the Konto river study site
Note: Discharge values mentioned in the figure are the capacities of tunnel/pipes

Environmental flow requirements are estimated for the Mendalan Sabo dam point and checked whether EF demands are meeting at this point in the current operation. In case EF is not ensured at Mendalan sabo dam point, the system is simulated with three different level (high, medium and low) of EF. For each simulation with different level of EF, power production is calculated.

Estimation of EF at Mendalan sabo-dam point of Konto

Information related to flow in the Konto at natural condition i.e. the flow before commissioning Selorejo reservoir on 1970 is not available. However, inflow to Selorejo is recorded and obtained from PJT-I. This flow can be used as proxy to the natural flow in the Konto if the Selorejo reservoir would not been there. At the downstream of Selorejo dam, three very small (in terms of discharge) tributaries met with the Konto (Figure 1). Up to the point of Mendalan sabo-dam, the length of the reach is about 8 km and after this point there is no major water abstraction from the Konto. Environmental flow is considered immediately downstream of this point after the diversion to Siman pond. Environmental flow is estimated based on the inflow to Selorejo with the addition of Sambong, Nogo and Nambaan flow. Only ten years monthly discharge data is in hand, hence Tennant method (Tennant, 1976) is adopted, which deals with only mean annual flow (MAF). Environmental flow requirements for wet and dry season are certain percentages of MAF for different environmental status according to Tennant.

RESULTS

Simulation of the system for existing operation

Inflow to Selorejo reservoir is known and acts as the upper boundary of the model. Reservoir operation is constrained by minimum and maximum storage i.e. 8.09 MCM and 39.6 MCM respectively. Hydropower generations are constrained as maximum discharge limit of the penstock. Based on this boundary and constraints, the simulation model is set up in a spread sheet and run with six years (2003 - 2008) mean monthly dataset, which represents the existing scenario. Downstream of Selorejo, only observed information is the Selorejo release, which is compared with the model output. The model output fits with the observed data with a correlation coefficient (r) of 0.88, root mean square error (RMSE) of 1.18 m^3/s and overall volume error (OVE) of 0.08 m^3/s.

In this simulation of existing operation, release from Selorejo first meets Selorejo power plant's demand. In case of higher release from the dam than the penstock capacity of Selorejo power plant, the extra flow spills to Konto main stream. Outflow from Selorejo plant is then going to Mendalan and Siman power plant subsequently. If it is necessary, small amount of flow is diverted to Siman plant from Mendalan sabo dam point. The observed and simulated power productions are compared for this existing operation and it shows a considerable well agreement. Performance of the simulation is measured using two parameters; namely: RMSE and OVE and reported in Table 1. Figure 2 shows the simulated and observed power for the Selorejo power plant. The simulated power productions from Selorejo, Mendalan and Siman power plants are 25,951, 78,844 and 57,611 MWh respectively. The total power production is obtained to be 162,407 MWh from the simulation model.

Table 1 Simulation model performance in terms of energy generation for existing condition

Power plant	RMSE (MWh)	OVE (MWh)
Selorejo	167.61	29.13
Mendalan	474.64	48.78
Siman	557.09	35.15

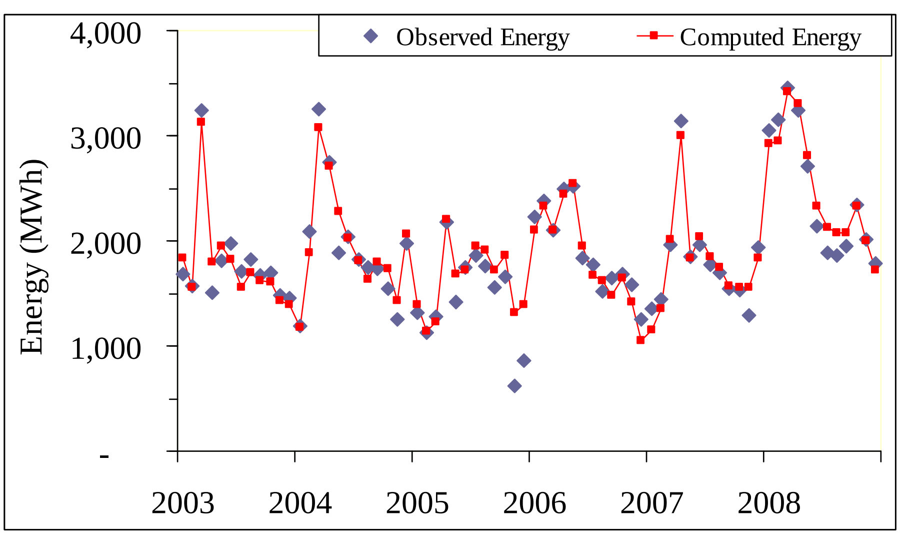

Figure 2 Observed and simulated monthly power production from Selorejo Power plant

EF requirements at Mendalan sabo-dam point of Konto

Based on ten years mean monthly inflow to Selorejo reservoir in addition to the flow of Nogo, Sambong and Nambaan river, MAF of Konto at Mendalan sabo dam point is estimated to be 10.74 m^3/s. Two seasons are considered, namely: high flow season (November – April) and Low flow season (May – October). Environmental flow requirements for both the seasons and for different environmental (habitat) status are calculated as prescribed by Tennant and reported in Table 2. Keeping in mind of maximization of overall benefit from offstream water uses 'fair or degrading' environmental condition is considered to be maintained which is 30% and 10% of MAF i.e. 3.22 and 1.07 m^3/s for the high and low flow season respectively.

Table 2 Environmental Flow requirements for the Konto based on Tennant method

Environmental status as defined by Tennant	Environmental Flow requirement (m^3/s)	
	High flow season (November - April)	**Low flow season (May - October)**
Flushing flow	20.48 (200%)	20.48 (20%)
Optimum range	6.44 – 10.74 (60 – 100%)	6.44 – 10.74 (60 – 100%)
Outstanding	6.44 (60%)	4.30 (40%)
Excellent	5.37 (50%)	3.22 (30%)
Good	4.30 (40%)	2.15 (20%)
Fair or degrading	3.22 (30%)	1.07 (10%)
Poor	1.07 (10%)	1.07 (10%)
Severe degradation	<1.07 (<10%)	<1.07 (<10%)

High flow season = November to April; Low flow season = May to October; EF= environmental flow; MAF= mean annual flow

Checking EF at Mendalan Sabo dam for existing operation

The monthly flow at Mendalan sabo dam is estimated from the simulation model for the existing operation system and compared with the EF requirements for 'Fair or degrading' level as estimated and presented in Table 2. It is observed that in dry season the actual flow does not meet the EF requirements. Comparison of the actual flow and EF requirements are plotted in Figure 3.

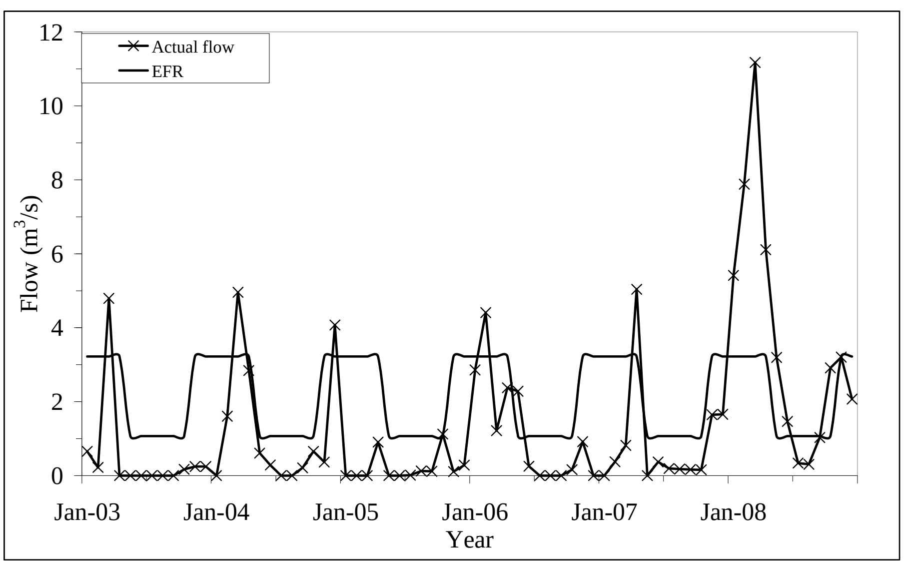

Figure 3 Existing flow and required environmental flow at Mendalan sabo dam, Konto

Power production after ensuring Environmental Flow

Environmental flows can be regarded as not exactly empirically determined figures, but they are more value judgments depending on the aim of river management. Specific physical situation and the expected state of the ecosystem should control the EF decision making. Without limiting the economic growth, achieving environmental sustainability is the main challenge along this line and that can be achieved within the context of wider EF assessment framework incorporating in the river basin planning. Considering such critical issues, three different values of EF are used and power production in each case is estimated.

Three different level of EF is tested, namely: (1) 'Poor' condition for the high flow season with a flow of 1.07 m^3/s and 'Severe degradation' for the low flow season with a flow of 0.75 m^3/s labeled as 'Low' level of EF, (2) 'Fair or degrading' for both the season with a flow of 3.22 m^3/s for the high flow season and 1.07 m^3/s for the low flow season labeled as 'Medium' level of EF, (3) 'Good' status for both high and low flow season is considered with the flow of 4.3 and 2.15 m^3/s respectively labeled as 'High' level of EF. All these 'poor', 'fair' and 'good' environmental status is as defined by Tennant.

Power production in the three levels of EF provisioning is estimated using the simulation model. The power production values are presented in Table 3. Figure 4 shows the power production relative to existing condition for various EF levels.

Table 3 Power production while ensuring EF at Mendalan Sabo dam

EF level	Power production (MWh)			
	Selorejo	Mendalan	Siman	Total
No EF	25,951	78,844	57,611	162,407
Low	23,556	73,354	53,723	150,633 (7%)
Medium	20,344	66,987	49,511	136,843 (16%)
High	17,492	59,615	44,854	121,962 (25%)

Note: value in parenthesis indicates % change with respect to no EF condition

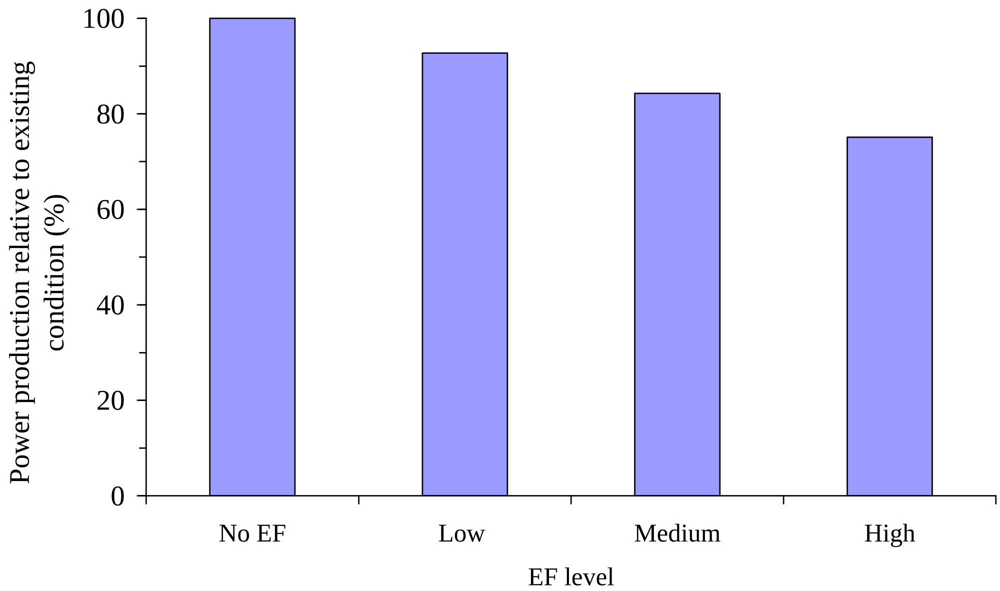

Figure 4 Power production relative to existing operation while EF is ensured

It is observed from Table 3 that higher the level of EF provisioning results higher reduction in power production. Higher reduction is observed in Selorejo plant. Total power production is reduces by 7, 16 and 25% from the existing condition for ensuring low, medium and high level of EF respectively.

CONCLUSION

In the present study, a sequential multiple water use system in the Konto river basin in Indonesia is thoroughly analysed. The system is simulated for the existing operation policy which does not ensure environmental flow downstream to Selorejo reservoir. Such operation of the reservoir might cause environmental degradation to the downstream part of the Konto. Environmental flow is therefore necessary to maintain, however, provisioning environmental flow results reduction in power production. The system is then simulated for three different levels of environmental flow; namely, low, medium and high. Ensuring environmental flow with high level results higher reduction in power production from the system and vice-versa. Nevertheless, at least low level of EF is strongly suggested to be ensured. The results from this analysis will help authorities to realize the cost of EF provisioning for the Konto river basin.

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USING OF GIS DATABASE FOR DRAINAGE IMPROVEMENT STUDY IN BANGLADESH

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ABSTRACT

Water logging problems create major effects for urban planning and it has been identified that improvement of the drainage system is one of the highest priority needs of the local authority for living environment of its urban population. The Pourashava suffers from drainage congestions and water logging especially during rainy season. It creates an unhealthy environmental situation and causes inconvenience to the residents of the Pourashava including damages to the infrastructure, loss of business and spreading of diseases. It is observed that there is a lack of planned and adequate drainage network system in the Pourashava. Existing drains are inadequate in capacities and lack in gradient and also do not reach the desired outfall. Moreover, those drains are insufficient to cover the full drainage resulting from rainfall runoff. The objectives of this drainage study is to assess the present drainage situation, identify the future requirements and suggest improvement of the drainage network system s by using GIS Database to provide the Pourashava a free area from water logging/congestion within an acceptable environmental condition.

Keywords: Drainage, Water Logging, GIS Database analysis.

1. INTRODUCTION AND OBJECTIVES

Improvement of the water supply and drainage facilities of the Pourashava has been identified as highest priority needs by the Pourashava authorities. Present drainage system is insufficient or not enough to handle the situation of draining the runoff resulting from heavy rainfall. There are a number of places in the Pourashava where water logging/drainage congestion occurs after heavy rainfall. The improvement of drainage system of the Pourashava has the following specific objectives:

- On the basis of outfall, dividing the total water shed into number of drainage zones to check the discharging capacity of the outfalls.
- Analyzing the existing conditions related to drainage facility in urban area.

- Improvement of drainage network by construction of new primary and secondary drains.
- Improvement of outfalls to accommodate runoff from present and future urbanized area.
- Cleaning and removal of blockage from existing drains and repair and rehabilitation of existing primary, secondary and tertiary drains.

2. APPROACH AND METHODOLOGY

Drainage system of an area is assessed through a sequence of analytical processes and it finally results in a proposed drainage system. The proposed drainage system is planned considering that the system is allowed for gravity drainage. The overall planning processes are: collection and quality study on maps of infrastructure/feature, contour, land use and image for a preliminary conceptual understanding of the Pourashava system with a review of reconnaissance and other available reports; identify rivers/khals surrounding the Pourashava and collect suitable hydrological gauging with a view to consider and process them for the understanding of hydrological response of the Pourashava; assess effective range of land levels which would be considered as a concern for planning process; make an intensive field visit for identification of possible outfalls and drainage routes in verification of the preceding planning processes; planning of drains & zones with identification of outfall locations/ reaches; and finally storm runoff assessment using empirical formula.Pourashava drainage systems are correlated with the average water level of Outfall River to review and iterate the proposed parameters of planned drainage systems.

3. DESCRIPTION OF THE LOCATION

3.1 *Location and Topography*

Chalna Pourashava is located in Dacope Upazila, Khulna District under Khulna Division. The change in elevation of most of the Pourashava area is gradual. The land elevation of the Pourashava effectively ranges between 0.64 mPWD and 3.64 mPWD. It is assessed that only 20% land of the Pourashava is below 0.94 mPWD.The use of present Pourashava’s area can be broadly divided into lands for agricultural (76%) and non-agricultural (24%).

3.2 *Rainfall*

Design rainfall storm intensity for the Pourashava is assessed from that of known design storm intensity of Dhaka applying a conversion factor which relates the rainfall events between Dhaka and reference station for the Pourashava. Chalna (R503) is a rainfall gauging station with reasonable length of records and is located nearest to the Pourashava.The average observed records of short duration (1986-09) of yearly 1-day maximum rainfall at Chalna is lower than the base station Dhaka.

3.3 *Flood*

The Pourashava lies in the Rupsha-Pashur River basin. The nearest water level gauging is available at Chalna (243) on Pashur River which is fairly calibrated by the regional model. The average year flood level for the Pourashava is estimated to 1.11 mPWD . The major parts of the Pourashava inside the Polder 31 and almost whole the part of it is flood free.

3.4 *Existing Drainage System*

There exist few lined and unlined drains within the Pourashava. These can drain some local areas of the Pourashava. The capacity and outfalls of existing drainage system is not planned with well defined consideration of drainage areas/zones for the whole Pourashava. Many of the drains randomly fall into relatively low lying areas. The lengths of existing lined and unlined drains are about 1.32 km.There are number cross drainage structures in the Pourashava as found during the survey. Following the field visits and survey, the main concerns for drainage issues of the Pourashava can be summarised as: i) undersized drains, ii) obstructions in the drainage system to outfall, iii) damages of drains, iv) inappropriate / temporary location of outfalls, and v) absence of planned and systematic drainage network system.

3.5 *River and Khal System*

The nearest river from the Pourashava is Pashur and Chunkuri which is just beside the east boundary of the Pourashava. On the other side the existing Titapara khal, Baraikali to satghoria Khal, Chalna Khal, Choto chalna Khal, kholisha gate khal, kadom tola khal, Achavua khal, Garkhati Khal, Katakhali Khal, Barow khal, Boro kalsha khal, Garkhati Khal, Zairbuner khal, Annandanagor Khal and Captain road side khal routes storm water from north-west part of the Pourashava and finally drains and routes to North-West and finally drains and routes to Jhapjhapia River.

4.0 DESIGN CRITERIA

The drains are designed to collect excess rainfall that is generated as surface runoff from urban area, convey the runoff and finally discharge them to outfalls. The design of drains involves hydrological computations of runoff from the drainage basin and also hydraulic computation of section of drain from result of hydrologic computations. Computation of runoff involves size and nature of the catchment area, computation of rainfall intensity, its frequency of occurrence, duration etc. Modified Rational Method is one of the simplest methods of calculation of runoff. It gives reasonably accurate result and widely used method for calculation of runoff for last few decades.In designing primary and secondary drains of Chalna Pourashava the Modified Rational Method is practiced.The runoff by Modified Rational Method is:

Peak runoff, $Q_P = C_s C_r IA/360$

Where; Q = Peak runoff flow rate (m^3/s)

I = rainfall intensity (mm/hr)

C_s = storage coefficient

C_r = runoff coefficient

A = catchment area (hectares)

4.1 Storage Coefficient

The rainfall after evaporation and infiltration accumulates first in the depressions, until these have been reached their capacity and then runoff. To take these effects a storage coefficient is used. The value of the storage coefficient is based on average ground slope and the nature of the ground surface. For estimating the Storage Coefficient of Chalna Pourashava **Table 1.1** is used.

Table 1.1: Storage Coefficients, C_s

Characteristics of surface	Storage Coefficient		
	Slope < 1:000	Slope < 1:500	Slope > > 1:500
Paved areas-road and market	0.8	0.9	1
Densely built up areas	0.8	0.9	1
Central area mixed commercial and housing	0.7	0.8	1
Residential areas with detached houses	0.7	0.8	0.9
Walled areas and garden	0.6	0.7	0.8
Large permeable areas (dry agriculture	0.5	0.6	0.8
Paddy field (flooded)	0.3	0.4	0.5

4.2 Runoff Coefficient

The runoff coefficient represents the ratio between the volume of runoff and the volume of rainfall. The runoff coefficient for the Pourashava is selected from list of C_r values given in **Table 1.2** below. Estimations of storm runoff for the proposed drains are done following the Modified Rational Method which is detailed in Annexure-5.

Table 1.2: Runoff coefficients, Cr

Land use designation	Runoff coefficient C_r
Paved areas-road and market	0.9
Densely built up areas	0.7
Central areas mixed commercial and housing	0.6
Residential areas with detached houses	0.4
Walled areas and garden	0.3
Large permeable areas (dry agriculture)	0.3
Paddy field (flooded)	0.8

5.0 DRAINAGE IMPROVEMENT PLAN

5.1 *Identification of Outfalls*

The eventual outfalls for the present and expanding core area of the Pourashava are mostly in the relatively low lying land and the topography of the Pourashava is such that the storm water runs through existing khals to Jhapjhapia River on North-West and Chunkuri on South-East. Titapara khal, Baraikali to satghoria Khal, Chalna Khal, Choto Chalna Khal, kholisha gate khal, Kadom tola khal, Achavua khal, Garkhati Khal, Katakhali Khal, Barow khal, Boro kalsha khal, Garkhati

Khal, Zairbuner khal, Annandanagor Khal and Captain Road side khal are natural drains which can serve as outfalls for drainage of most areas of the pourashava.

5.2 *Proposed Drainage System*

The area of the Pourashava has been planned for improvement under gravity drainage system. The whole Pourashava has been divided into 14 zones for drainage improvement plan shown in **Figure 1**. Zones 6, 7 and 8 are planned with proposed storm drains as they are in the core area of Pourashava or will be characterized as core area in near future. Zones 1, 2, 3, 4 & 5 will drain in West direction through existing khals and finally drain to Jhapjhapia River. Zones 9 & 10 will drain in South-West direction and finally drain to Chunkuri River.

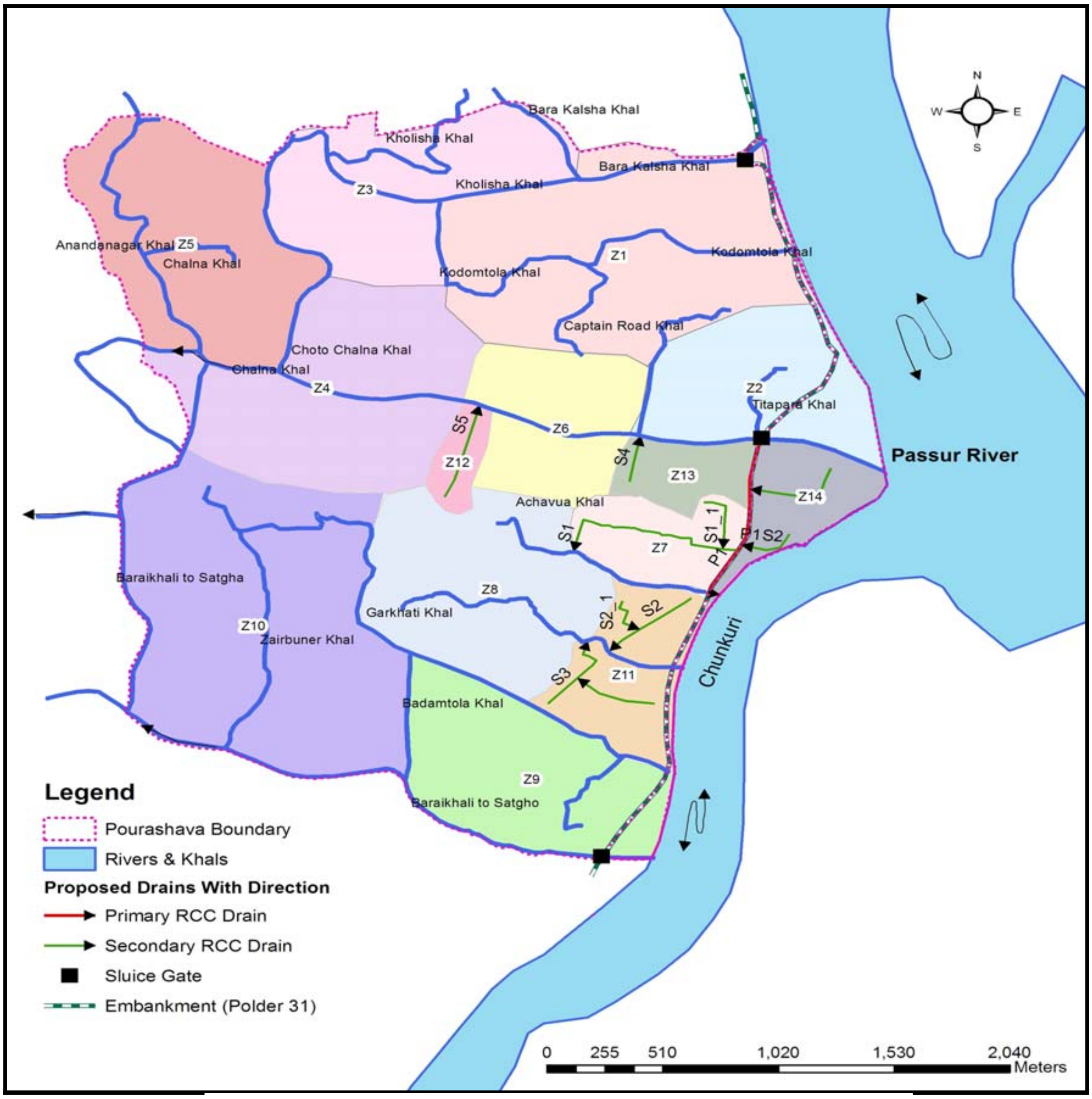

Fig 1: Drainage Zones and proposed drains with outfall

5.3 *Functional Assessment of Proposed Drainage System*

Functions of the proposed drainage system are assessed in respect of monsoon flood period. Average year water level in the vicinity of the Pourashava has been determined using the nearest water level gauging at Chalna (243) which is found as 1.10 mPWD in the Pashur River. Lands above and below the flood level as well as proposed drainage system is shown in Figure 1.2. More than 57% land of the Chalna Pourashava is above the average flood level.

6. RESULTS

Drainage Parameters **Table 1.3: Drainage Parameters**

Drain ID	Chainage	Model Flows Q_d (m3/s)	Bottom Width (m)	Actual drain depth* (m)	Design Capacity, Q_c (m^3/s)	Ground Level		Bottom level		Remarks
						U/S	D/S	U/S	D/S	
P1	0-242	0.21	0.65	0.85	0.30	2.32	2.15	1.30	1.10	Rcc
	242-530	0.46	0.8	1.0	0.49	2.15	2.25	1.10	1.05	Rcc
	530-876	0.76	1.0	1.2	0.78	2.25	2.20	1.05	1.00	Rcc
P1S1	0-452	0.65	0.8	1.15	0.98	2.23	1.76	0.85	0.60	Rcc
P1S2	0-256	0.18	0.5	0.75	0.30	1.89	1.85	1.2	1.05	Rcc
S1	0-60	0.11	0.7	1.15	0.25	2.49	2.15	1.62	1.0	Rcc
	60-893	1.42	1.0	0.85	0.64	2.15	2.88	1.0	0.90	Rcc
S1_1	0-321	0.43	0.65	0.95	0.44	2.02	2.10	1.25	1.10	Rcc
S2	0-277	0.29	0.6	0.95	0.39	1.94	1.72	1.15	0.8	Rcc
	277-451	0.48	0.7	1.0	0.53	1.72	1.77	0.8	0.7	Rcc
S2_1	0-227	0.18	0.55	0.8	0.27	1.55	1.72	0.75	0.65	Rcc
S3	0-183	0.19	0.6	0.9	0.36	1.71	1.84	0.9	0.8	Rcc
	183-449	0.37	0.7	0.95	0.49	1.84	1.65	0.8	0.7	Rcc
S3_1	0-373	0.42	0.7	0.95	0.49	2.21	1.82	1.25	0.8	Rcc
S4	0-240	0.34	0.6	0.95	0.39	1.55	1.60	0.8	0.85	Rcc
S5	0-495	0.78	0.8	1.2	0.83	1.64	1.95	0.85	0.65	Rcc

Considering Freeboard. For Primary Drain 0.2m & Secondary Drain 0.15m

Note: i) For RCC drain n=0.014 & Earthen drain n=0.025

ii) For this Pourashava avg. year Water Level is 1.10 mPWD

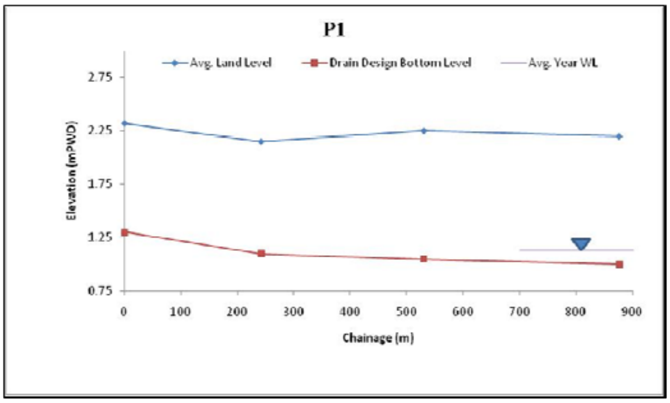

Fig 2: Longitudinal Profile of Drain P1

7. CONCLUSION

The highlight features of the Pourashava system in connection with the issues of its storm drainage are: Proposed drainage network is adequate in view of removing drainage congestion and water logging for the land above the flood level, proposed drainage network is sensitive to flow obstruction / constriction. The impact of obstruction propagates much up reaches and reduces drainage efficiency. The study is adequate enough depending upon the precision of present collected information. The analysis results indicate that if the design parameters are implemented the storm drainage congestion will be relieved shortly with the proposed drainage improvement plan provided that there will be no encroachment, maintenance of regular slope is ensured etc.

8. RECOMMENDATIONS

Following management / interventions are proposed for drainage improvement: P1, P1S1, P1S2, S2, S3, S4 and S5 drainage systems have priority needs while S1_1, S2_1 and S3_1 drainage systems are proposed in view of near future needs for the Pourashava. It is estimated that about 12 nos. of cross drainage works (e.g.; box culverts/ pipe culverts) will be required in connection with the whole proposed drainage network. Zones 5, 9, & 10 of the Pourashava drain and will drain overland across the Pourashava boundary to the low lying area and finally route and drain to Khals. The Pourashava authority will have institutional linkages with all relevant line agencies for the continuation of drainage provision of Zones 5, 9 & 10 in view of long term consideration.

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HYDRODYNAMIC PERFORMANCE STUDY OF GEOBAGS IN REVETMENT

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ABSTRACT

The applications of geotextile for manufacturing geobag, geotube and/or geocontainer in hydraulic engineering practices have been increasing. In last two decades geobags (sand filled geotextile bags) have been using both in coastal and riverbank protections. Geobag revetment/dike provided many successful cases in coastal protection around the world, but there are only two cases described experiences from riverbank protection works. Apart from the Changjiang River in China, geobag revetment was adopted during last decade for the Jamuna and Meghna River bank protection in Bangladesh. Unfortunately some parts of the revetment experienced bag displacement and thus fail to offer the design revetment life. The active hydrodynamic forces on coastal geobag revetments are different than those are in riverbank; due to lack of details information on hydrodynamic forces the revetment experienced early dislodgement performances become difficult. Based on critical review on all the relevant literatures as well as experience in laboratory and field, this reported study devoted to establish a relationship among hydrodynamic forces on geobag revetment and its stability. The proposed relationship is expected to provide useful information on other flexible construction materials i.e. geotube, geocontainer and also for discrete elements riprap, concrete cubes etc.

Keywords: geobag; riverbank; hydraulic forces; stability

INTRODUCTION

The applications of geotextiles have been increasing for the river bank protection. Geotextile products are relatively low in cost, materials are more readily available and obtainable, usually heavy machinery is not required, unskilled labour can be used and labour time is decreased (Kobayashi and Jacobs, 1985, Pilarczyk, 1998). In recent years geotextile uses for manufacturing Geobag, Geotube and Geocontainer for different hydraulic engineering practices. The Geobag contains dredged material and is used in bank protection for river or shoreline, an artificial island and other related projects. The general assumptions were made regarding the behaviours of the geobags in revetment, were:

- Firstly, friction was taken as a conservative mode (constant roughness coefficient = 0.6, (Gadd, 1988) or neglected (Zhu et al., 2004, Korkut et al., 2007). This includes friction between the bag and ground and friction between neighbouring bags;

- Secondly, to avoid 'interlocking' problem among bags, the fill ratio of approximately 75 to 80% was adopted as an optimum stability of the element (Breteler et al., 1998, Grüne et al., 2006). Then, the thickness of the geotextile selection varied based of fabric types. However the weight of the geotextile is neglected;
- Next, the bags were assumed to lay flat on rigid surface. The deformable foundation was more appropriate in riverbank case;
- According to Breteler et al. (1998) geobag becomes unstable above a flow velocity of 1.5 m/s. Finally, the stability of bag to withstand the wave was drawn by using Hudson's equations (Gadd, 1988) or in terms of the surf similarity parameter (ξ) (Kobayashi and Jacobs, 1985, Saathoff et al., 2007).

The hydrodynamic behaviour or stability of geobags depends on different properties of the bag. Gadd (1988) defined the mechanical and hydraulic properties of sand bag for arctic offshore applications. Oberhagemann and Kamal (2004) mentioned nine different mechanical and hydraulic properties of geotextile with their standard test values. These properties are the opening size, mass per unit area, California Bearing Ratio (CBR) puncture resistance, tensile strength, elongation at rupture, permeability, porosity, abrasion and UV resistance. Following sections are devoted to describe hydraulic properties of geobags and then the experiences from coastal protection works an also from the Jamuna Meghna River Erosion Mitigation (JMREM) project (2004 -2008).

HYDRAULIC PROPERTIES OF GEOBAGS

'Incipient motion' is an important criterion for the stability of geobag structure. The local depth averaged velocity (at 1/3 bank height) for filled geobags and the critical flow velocity are influential factors in this regards. Their combined effect is required for the geobag's settling distance computation.

Critical Velocity

The local depth averaged velocity (at 1/3 bank height) for filled geobags incipient motion is developed for JMREM project (Oberhagemann and Hossain, 2010), is:

$$V = \frac{K_{sl}^{0.5} g^{0.5} \Delta^{0.5} D_{50}^{0.4} Y^{0.1}}{\left(C_s C_v\right)^{0.4}} \quad \mathbf{1}$$

Where,

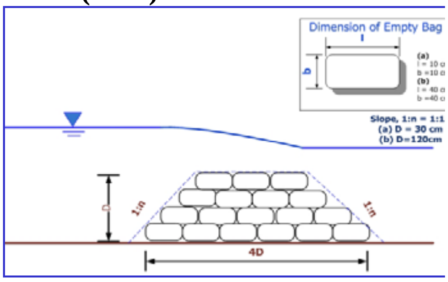
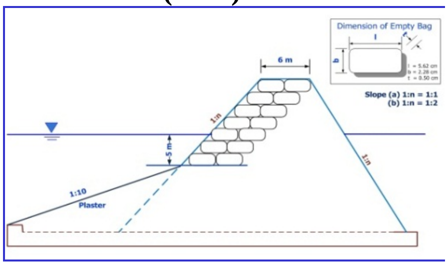
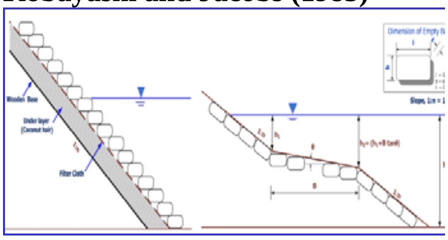
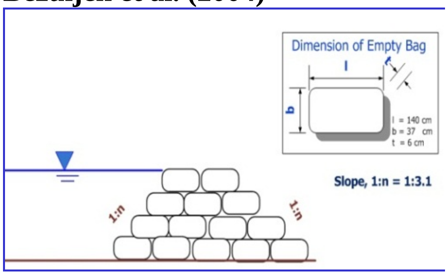
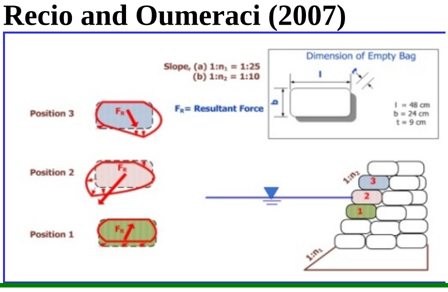
Y = Depth at 1/3 bank height = $\frac{H_{bank}}{3}$

C_s = Shape coefficient, value = 0.77

C_v = Coefficient of vertical velocity distribution, ranging from 1.0 to 1.28 for straight channel to abrupt bends

K_{sl} = 0.71 (for 1V:1.5 H) or 0.88 (for 1V:2H)

Table 1 a: Cross sectional view of physical models

	Details of the structure	
Venis (1968) 	• Model scale • Structure type • Bag Material • Filling Material (Filling Ratio)	1:5 and 1:20; Embankment ($1:n=1:15$); Jute and linen; and Sand (Wet and Dry); (80%).
Porraz *et al.* (1979) 	• Model scale • Structure type • Bag Material • Filling Material • Filled Weight (gm)	1:60; Breakwater ($1:n=1:1$ and $1:2$); Polyethylene; Mortar (Mastic); and 13.51.
Kobayashi and Jacobs (1985) 	• Structure type • Bag Material • Filling Material • Filled Weight (gm)	Revetment ($1:n=1:3$); Geotextile(Woven); Sand; and 367.
Bezuijen et al. (2004) 	• Model scale • Structure type • Bag Material • Filling Material (Filling Ratio) • Filled Weight (gm)	1:20; Dam ($1:n=1:31$); Geotextile; Sand; (50 - 70%); and 49.
Recio and Oumeraci (2007) 	• Model scale • Structure type • Bag Material • Filling Material	1:8; Revetment ($1:n=1:1$); Geotextile; and Sand.

Settling Distance

Bezuijen et al.(2004) expressed the fall velocity or settling velocity in terms of time when a sandbag released below the water surface, as:

$$V_f \qquad V'_f \frac{1-e^{-2t/t'}}{1+e^{-2t/t'}} \qquad (2)$$

Where,

V_f ,	=	Falling velocity
V_f'	=	Final velocity
C_d	=	Drag coefficient
t'	=	Characteristic time, the time necessary to reach a certain velocity

Table 1 a and b represent the comparative analysis of previous physical models of geobag as coastal structures based on their findings. Geobag revetments for riverbank protection are summarized in Table 2.

Table 1 b: Comparative analysis of physical model tests for geobags (as Coastal Structure)

Author	Bag Placement	Findings	Recommendation
Venis (1968)	Face to Face	-Influence of scale effect on sand circulation inside bags; and -Relationship between critical current velocity and the bag length.	-Sand fill ratio 80% is the optimum amount.
Porraz *et al.* (1979)	100% overlapping (with/ without)	-Steeper slope and higher overlapping confirm higher stability; and -Development of curves to estimate the required weight of bags in design waves.	-Bags placement with their longest axis perpendicular to the crest of waves; and -Placement of larger crest bag will enhance stability.
Kobayashi & Jacobs (1985)	50% overlapping	-Increased overlapping increases stability; and -Empirical relationship of uniform and composite slopes defined by surf similarity parameter (ξ).	-Quantify effect of scale; and -Wave period on the stability of armour units.
Bezuijen *et al.* (2004)	-	-Placing accuracy of bags is limited at water depths larger than 15 m.	-Recommendation for fill material, strength of seams, saturation of the fill and quality of barges.
Recio and Oumeraci (2007)	50% overlapping	-Most critical bags is located just below the surface water level; -Most critical phase for the revetment stability occurs during down rush; and -Internal movement of sand inside the bag induces deformation of bag and therefore substantially affects the revetment stability.	- Effect of the deformation of the sediment filled bags needed to consider in stability formulation.

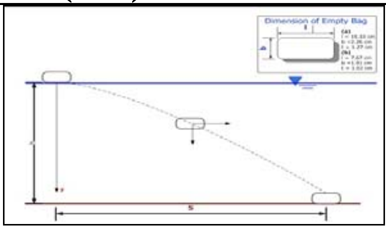
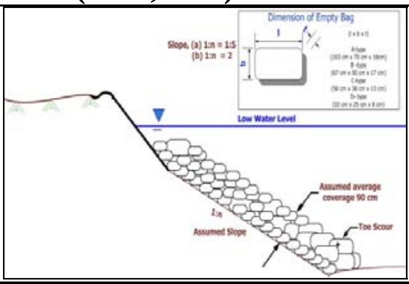
CONCLUDING REMARKS

There are only few of studies work on the geobag stability features. Studies on the physical properties of geobag show good agreement with field observation. The bag movement in riverbank protection cases still assume either as a coarse aggregate (stone or rock) or coastal structure. So, the instable hydraulic behaviour of geobag in reality for riverbank case cannot achieve its desired success. Experiences from the previous studies on geobag structure suggested as:

- 50% overlapping identifies as optimum measure to withstand the hydrostatic force (Table 1b);
- Due to internal sand movement with respect to wave action, uplift deformation observes; and there are interrelation among deformation, bag to bag contact area and the displacement;
- The bag to bag gaps influence permeability;

- Segregation observes in small bag structure, 126 kg weighted bags performed well in JMREM project;
- Due to friction among bags overhang appears in layer and clusters of bags slumped down;
- Total normal force acting on the contact areas among bag-bag is determined as resisting force of the geobag structure.

Table 2: Physical model setup

	Details of the structure		
Zhu (2004)			
	• Model scale	:	1:60
	• Structure type	:	Submerged Dike
	• Bag Material	:	Polypropylene
	• Filling Material (Filling)	:	Sand (-)
NHC (2004)			
	• Model scale	:	1:50
	• Structure type	:	Mega-container
	• Bag Material	:	Geotextile;
	• Filling Material (Filling)	:	Sand (80%)
	• Filled Weight (gm)	:	8000
NHC (NHC, 2006)			
	• Model scale	:	1:20
	• Structure type	:	Revetment
	• Bag Material	:	Polyester/cotton ;
	• Filling Material (Filling)	:	Sand (80%)
	• Filled Weight (gm)	:	16, 11, 5 and 1

The above mentioned studies also have some limitations, for instance:
- Absence in clarifications of overlapping between layer to layer or interlayer;
- No significant recommendation found on: required times for deformation and displacement occurrences; and their influence on geobag structure stability;
- Verification of this assumption is still left;
- In local scour test, failure modes for the 126 kg bag or small bag cases are not observed, and their influence on geobag structure formation and stability; and
- Clarification on the interaction among resisting forces are required to evaluate their activeness to secure structure stability.

Intensive studies are urgent on the geobag stability features a details understanding can only provide required information on the acting hydrodynamic forces (lift, drag, buoyancy) as well as the features of hydraulic parameters. An ongoing research work is currently carrying out in the hydraulic laboratory, department of Civil Engineering, Chittagong University of Engineering and Technology (CUET) Bangladesh in this regard.

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EFFECTIVENESS OF PLATFORM COLOR AS ARSENIC AND MANGANESE SCREENING TOOL IN SHALLOW DEPTH TUBE-WELLS: A STUDY FROM MATLAB, BANGLADESH.

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ABSTRACT

High level of geogenic arsenic (As) and Manganese (Mn) in groundwater is one of the major and adverse drinking water quality problems in all over the world, especially in developing country. The development of a simple low cost technique for the determination of As and Mn in drinking water wells is an important step to formulate this policy. The aim of this study was to evaluate the potentiality of tubewell platform color as low-cost, quick and convenient screening tool for As and Mn in drinking water wells (n=272). The result shows strong correlation between the development of red color stain on tubewells platform and As enrichment in the corresponding tubewell water compared to WHO drinking water guideline (10 µg/L) as well as Bangladesh drinking water standard (BDWS) (50 µg/L), with certainty values of 98.7% and 98.3% respectively. The sensitivity and efficiency of red colored platforms to screen high As water in tubewells are 98% and 97% respectively at 10 µg/L, whereas at cut-off level of 50µg/L both sensitivity and efficiency values are 98%. This study suggests that red colored platform could be potentially used for primary identification of tubewells with elevated level of As and thus could prioritise sustainable As mitigation management in developing countries. Due to lack of tubewells with black colored platform in the study area, the use of platform color concept for screening of Mn enriched water in the wells have not been tested significantly, which deserves further study.

Key words: Groundwater, Arsenic, Manganese, Tubewell, platform color.

INTRODUCTION

The geogenic occurrence of arsenic (As) and manganese (Mn) in shallow depth ground-water is a severe drinking water quality problem all over the world (Bhattacharya et al., 2010), especially in Bengal Delta Plan (BDP) consisting West Bengal, India and Bangladesh, where millions of tubewells are installed to shift drinking practice from surface water to groundwater to escape waterborne diseases. Millions of people are presently depends on these tubewells water, which extracted from

Figure 1: Platform color distribution a) Red wells; b) Not Identified (NI) wells; c) Black wells

shallow depth aquifer and contaminated with high level of As and Mn (Chakraborti et al., 2008). The concentration of As exceeded the World Health Organization (WHO) drinking water guideline value of 10 μg/L and Bangladesh Drinking Water Standard (BDWS) of 50 μg/L (von Brömssen et al., 2007). Arsenic concentration in tubewells water varies widely, both vertically as well as spatially within a very short distance, for which it is not possible to predict by testing a small number of tubewells. Recently, excessive concentration of Mn is also recognised as a significant drinking water quality problem in many areas (Tasneem & Ali, 2010). Consumption of drinking water with high level of Mn causes neurotoxic effects, such as reducing intellectual capacity of children. Although exposure to Mn is less harmful for human health than that of As exposure (Hug et al., 2011). As a part of sustainable drinking water management, regular monitoring of As and Mn in all the wells in a region is necessary.

Regular monitoring of As and Mn in all the wells in a region is a very difficult work due to lack of technology, transportation of prepared sample to laboratory, economy, social acceptance, manpower and time for a country like Bangladesh and India. To find out a proper system for determine As level in tubewells water by short time and at low cost a number of field test kits have been proposed by researchers like van Geen et al (2005) and Jakariya et al (2007). Some commercial field test kits also available, which are proven worth for As screening, e.g., Merck, Hach EZ, Quick arsenic, Wagtech Digital Arsenator (WFTK), and Chem-In Corp field test kit (CFTK) (van Geen et al., 2005; Steinmaus et al., 2006; Sankararamakrishnan et al., 2008; Biswas et al., 2012). Conversely, these kits are developed based on generating arsine gas, which are very harmful for analyst health. Moreover, these analysis techniques is time consuming (>20 minutes) to know As level for a single sample and even rural people have less access to these test kits. On the contrary, there are a limited number of Mn concentration measurement techniques in tubewells. Hatch field test kit (Model MN-5) is one of the Mn concentration measurement kits, which consistency has not been proven yet. Research is still going on to formulate and facilitated the As and Mn concentration measurement in short time with convenient way.

In very recent, McArthur et al. (2011) and Biswas et al. (2012) have proposed that color stain developed on tubewells platform, where available, can be used to screen As and Mn, which installed in shallow aquifer (<70m). In west Bengal, India, they observed that Black stained on platform arises due to precipitation of Mn-oxides, which are As safe (<10 μg/L) and red stained due to precipitation of Fe-oxides which are As enriched (>10 μg/L). This particular color formation on tubewells platform and relation with Fe, Mn and As concentration in groundwater is governed by different biogeochemical interactions (Biswas et al., 2012). During lowering of redox condition by different microbial oxidation of organic matter Mn-oxy-hydroxides are reduced before Fe-oxy-hydroxides since Mn-oxy-hydroxides is strong electron acceptor than Fe-ox-yhydroxides. After reducing Mnoxyhydroxides, releasing both Mn and As in groundwater and this released As is readily reabsorbed to Fe-oxy-hydroxides (due to charge difference) (Stuben et al., 2003) in the aquifer sediment and water become Mn enriched. During extraction of this Mn enriched water by pumping, this Mn, oxidised by atmospheric oxygen to Mn-oxides, which produced black coloration on tubewells platform and indicates as As safe wells. If the reducing condition reduced to Fe reduction stages, Fe as well as absorbed As released to groundwater and become Fe and As enriched aquifer. During extraction of this water Fe oxidised to Fe-Oxides and produces red coloration on tubewells platform. However, Mn is more soluble than Fe in water at normal pH, hence Fe precipitates before Mn and vice-versa for higher Mn/Fe ratio (Gingborn & Wåhlén 2012). If the reducing condition is limited to Mn reduction only, groundwater enriched with Mn and produced black coloration of Mnoxides on tubewells platform. According to this, color stain developed on tubewells platform may be used as screening tool for Mn and As in groundwater.

In this study we have assessed the effectiveness of tunewells platform color as a effective As and Mn screening tool in Bangladesh well waters, which can be used in wide application in As and Mn affected area. For As, the tool is evaluated corresponding to both WHO drinking water guideline value of 10 µg/L and BDWS of 50 µg/L. Fascinatingly, at present WHO do not have any recommended value for Mn in drinking water and hence, the evaluation is done corresponding to BDWS of 100 µg/L and India of 300 µg/L.

MATERIAL AND METHODS

The study area is located in Uttar Matlab Upazila in Chandpur District, Bangladesh with an area of 30 km^2. The groundwater sampling was carried out from 272 shallow (<70) aquifer tubewells during September to October 2011. During sampling, each wells was purged for few minute (depth in ft=no. of purged) to get the filter level sample water for laboratory analysis of As, Fe and Mn. The sample water was filtered through 0.45 µm Sartorius membrane filter and collected in 20 ml prewashed high density polyethylene vials. To preserve metal trace element in dissolved state, the sample water was acidified by ultrapure nitric acid (14M HNO_3). The geographical coordinates of each tubewells was collected by Global Positioning System (GPS, Garmin-GPS60). Beside this, the major coloration on platform was examined carefully and owner opinion on coloration also recorded. A picture of each tubewells platform was taken by a digital camera (Sony Cyber Shot-W220, 12MP, 4x optical zoom). The tubewells depths and installation year for both tubewells and platform also recorded from owner. The platform color was re-examined in laboratory by an unbiased operator for cross-veryfication of the color identifications. The reciprocal agreement on platform color was more than 74% (n=202). All disagreement (n=70) was due to separation of mixed color from red and black colored platform.

The entire water sample was carried out to Sweden for chemical analysis. The trace elements such as As, Fe and Mn of samples water were analysed by Inductively Coupled Plasma-Optical Emission Spectrometry (ICP-OES) equipped with auto sampler at the department of Geology and Geochemistry, Stockholm University, Sweden.

To evaluate the effectiveness of platform color stain as As and Mn screening tool, a statistical analysis based on Baysian statistics was carried out. At the same time, sensitivity, specificity, efficiency and predictive values was determined, with respect to WHO drinking water guideline value and BDWS for both As and Mn to validate the platform color as screening tool.

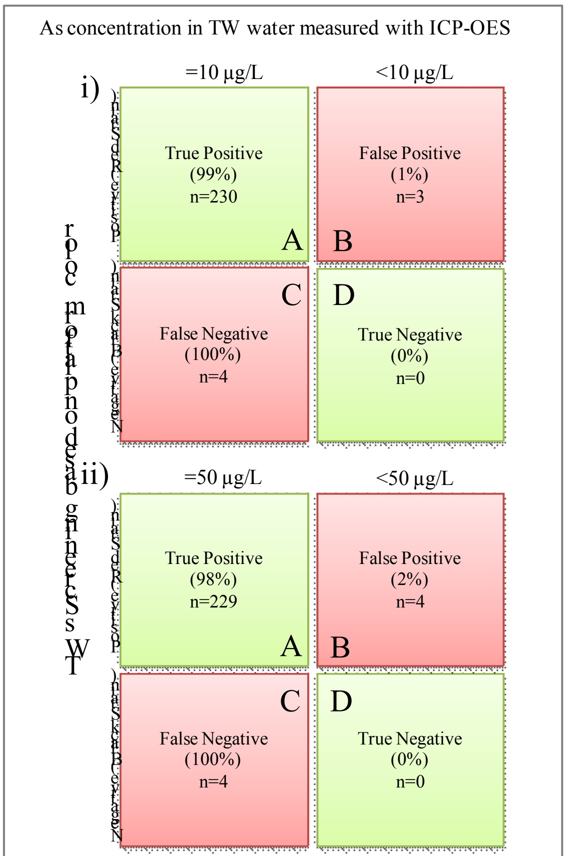

Figure 2: Generic approach for validation of platform color as As screening tools in shallow depth wells

RESULTS AND DISCUSSIONS

Tubewells classification according to Platform color

According to tubewells platform color, the tubewells were classified into three different color group namely red, black and not identified (NI) (Figure 1). Within 272 wells, 237 wells platform (87%) were classified into a particular color, where 233 (86%) wells platform as red, 4 (1%) as black, and rest 35 (13%) wells platform as NI due to undeveloped platform color. Undeveloped color may be the reasons of regular cleaning of platform, biofilm formation or algal growth on platform. The age of the platform has also an effect on color formation on platform, because newly built platforms are less exposed to built coloration on platform regardless excessive Mn or Fe in well water. Some platform has complex color (not black or not red), which may be due to overlaping redox transition of Mn and Fe in aquifer.

Evaluation as a Screening Tool for As

Arsenic concentration within the surveyed (n=272) wells without considering platform color, 99% (n=269) exceeded the WHO drinking water standard value of 10 μg/L, and 98% (n=267) exceeded BDWS (50μg/L). In the contrary according to platform color, within 233 red colored platform wells, 99% (n=230) exceeded WHO guideline value and 98% (n=229) wells exceeded BDWS. Tube-wells with black color (n=4) platform is also As rich and exceeded BDWS value. However, the wells, which are allocated as NI color are 97% (n=33) exceeded BDWS and all (n=35) the wells exceeded WHO guideline value.

On the other hand, if it is consider on the basis of As safe TWs, without considering platform color, it was shown that only 1% (n=5) within 272 TWs are safe with respect to BDWS (50 μg/L) and three TWs by WHO guideline. If platform color is considered then for red color platform wells, three and four wells is safe (within 233) by considering WHO and regional standard respectively. Only one well is safe from NI wells (n=35) even for BDWS and no one from black colored platform wells. Thus NI wells can also be considered as As enriched wells.

Effectiveness as As Screening Tool

Effectiveness of platform color as As screening tool in TWs depends on high probability of truepositive and negative values and correspondingly on low probability of false-positive and negative values relating to specific drinking water standard (Biswas et al., 2012). Figure 2 shows the relationship of the relevant true-positive and negative and false-positive and negative values for WHO and BDWS values. Table 1 summarizes the sensitivity, specificity, efficiency and positive and negative predictive values which also the indicators of effectiveness for screening capacity.

At WHO guideline value for As (10μg/L), the positive predictive value (PPV) of red colored

Table 1: Assessment of the Effectiveness of TW Platform Color as a Screening Tool for As in TWs

Indices for Validation of Platform Color Tool (%)	Basis for Calculation	WHO guideline (10 μg/L) 95% Confidence interval			Bangladesh Standard (50 μg/L) 95% Confidence interval		
		Estimated value	Lower limit	Upper limit	Estimated value	Lower limit	Upper limit
Sensitivity	A/(A+C)	98.3	95.4	99.4	98.3	95.4	99.5
Specificity	D/(B+D)	0	0	69.0	0	0	60.4
Efficiency	(A+D)/(A+B+C+D)	97	-	-	96.6	-	-
Positive Predictive Value (PPV)	A/(A+B)	98.7	96.0	99.7	98.3	95.4	99.4
Negetive Predictive Value (NPV)	D/(C+D)	0	0	60.4	0	0	60.4

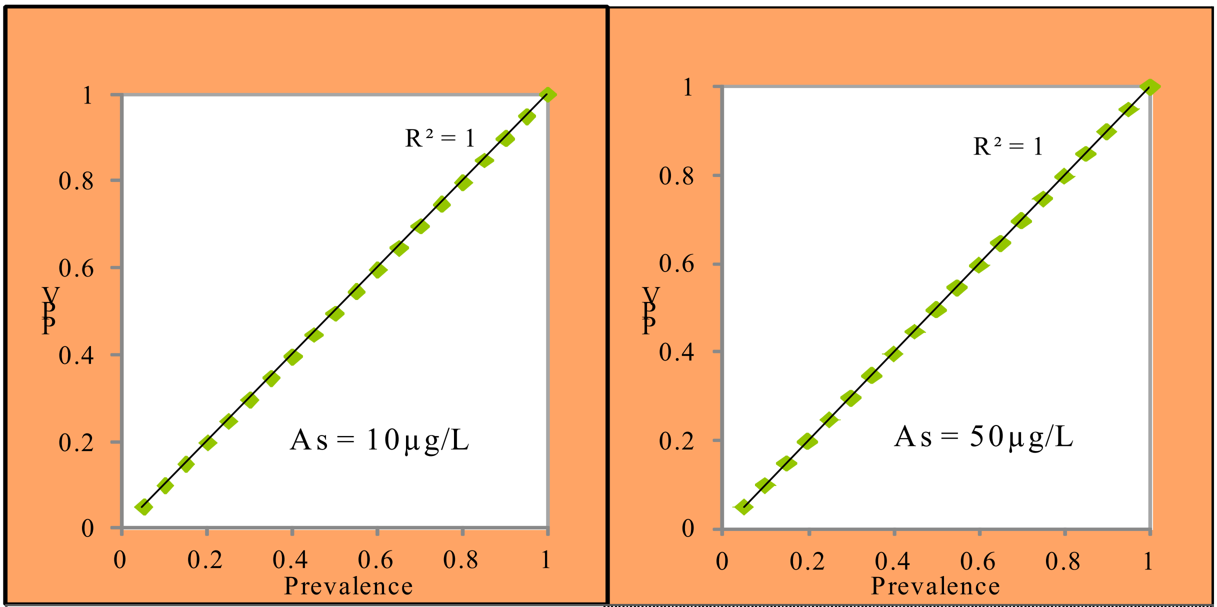

Figure 3. Relation among Positive predictive values (PPV) and prevalence of tubewells with As concentration = 10 µg/L and = 50µg/L.

platform wells is 98.7%, while negative predictive value (NPV) is zero (less no of black colored wells). The corresponding efficiency, specificity, and sensitivity of the tool are 97%, 0% and 98.3%. This result shows that platform color can be used as an initial As screening tool in groundwater. However, for BDWS (50µg/L), the PPV is 95.4% and NPV is zero due to same reason. The corresponding efficiency, specificity, and sensitivity of the tool are 96.6%, 0% (no black colored wells with <50µg/L) and 98.3%. This also indicates that at BDWS, red color platform can be used as As screening tool as unsafe tubewells, but black colored platform wells cannot be used as an indicator to screen As safe wells.

Evaluation and effectiveness as Mn Screening Tool

The Mn distribution over the surveyed tubewells (n=272) without considering platform color, shows that almost all (n=271) exceeded 100 µg/L of Mn. On the contrary according to platform color distribution within 233 red colored platform wells, 99.6%, n=232) wells exceeded Bangladesh standard (100 µg/L). This result shows that red colored platform wells cannot be used as Mn safe wells for groundwater considering Bangladesh drinking water standard, which is the opposite of the hypothesis. On the other hand, all the tubewells with black colored platform (n=4) and NI colored tubewells (n=35) exceeded 100 µg/L of Mn and state that black and NI colored platform wells are Mn unsafe wells.

Since the number of black colored platform is very less (n=4) and also contains high ranges of As so it does not indicate that black platform wells should have less As as mentioned by McArther *et al.*(2011) and Biswas *et al* (2012).

Predictive value and Prevalence of As and Mn screening tool

The relation among prevalence and predictive values also indicate the effectiveness of platform color as screening capacity of As and Mn in groundwater. Since the black colored wells is very less, prevalence and predictive values is not calculated in relation to Mn scrrening but done only for red colored platform to screen As in groundwater. Bayesian model is used at different cutoff level to evaluate the effect of prevalence and predictive values (Figure 3). The figure shows that the PPV varies linearly with prevalence for both standard (50µg/L and 10µg/L) due to very less black colored platform tubewells (n=4). That means, in a particular area, the performance of the tool to identified as As unsafe tubewells is increases if the concentration of As in that wells is increase and vice versa.

CONCLUSION

From this study, it can be said that tubewells platform color can be used as rapid screening tool for As but not possible to say for Mn screening in well water. For Mn screening, further study is demanded in area where black platform wells are avaible.

So,it can be said that, red color platform can be introduced at the policy level to takle the problem associated with As safe drinking water supply as well as primary guide to screen As inriched wells. This indicator can save time and cost of testing wells significantly.The great advantage of platform color tool is simplicity. The villagers can use to identify the wells, wheather it is safe or unsafe who have not access to conventional test kits and may lead to reduces As exposure.

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EVALUATION OF TUBE-WELL PERFORMANCE AND AQUIFER CHARACTERISTICS OF DHAKA CITY USING PUMPED WELL DATA

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ABSTRACT
Dhaka WASA has divided the whole city into ten zones to administer its operations of potable water supply and safe sewerage disposal. The construction and the development test data of 50 selected tube-wells (5 at each zone) from these ten zones had been considered. Parameters, such as, well efficiency and specific capacity were evaluated to characterize the properties of the well while transmissivity and storativity were weighed to determine the properties of the aquifers. The development tests on the deep tube-wells of DWASA were performed without installing piezometers. Thus, the pumping tests conducted by DWASA can be treated as a single well test. Hantush-Bierschenk's method was used to find the relative co-efficient of the well.As the well efficiency were found scant in Dhaka city compared to its overall expansion, it dictate the use of an appropriate method which took care of these standards in its transmissivity determination.To determine transmissivity and storativity 4 suitable methods were selected from literature to overcome the lack of piezometer readings. Among these 4 methods, Jacob’s straight line method and Papadopulos-Cooper’s method were applied with AQTESOLV whichfeatures the most comprehensive set of solution methods for confined aquifers.Jacob’s method gave consistently higher transmissivityvalues than the other 3 methods. Papadopulos-Cooper method was valid for fully penetrated confined aquifer. Hurr-Worthington had been judged the most suitable method for the aquifer system in Dhaka city.The Logan approximation method used the specific drawdown data for transmissivity determination.By using AQTESOLV, the storativity of the aquifer was determined.

Keywords: Groundwater, Tube-Well, Aquifer Characteristics,Dhaka.

INTRODUCTION

The present research deals with the evaluation of the tube-well performance and aquifer characteristics of Dhaka city using the pumping test data of DWASA wells. Due to progressive trend of abstraction the Dhaka city aquifer system is withstanding, the recharge-discharge regime may significantly be changed as a function of time. Groundwater can be regarded as a reliable source of water supply for any climatic condition of a region. Groundwater Source provides clear water at almost constant temperature and is preferred compared to surface water for municipal water supply.

The city of Dhaka has an extensive water supply system dependent mainly on groundwater source. Dhaka WASA is supplying potable water to the metropolitan area by operating numerous deep tube-wells at different places inside the city. To keep pace with the expansion of the Dhaka city, water supply system has expanded through the development of numerous tube-wells, and extension of the distribution system.DWASA is currently operating about 710 deep tube-wells with a total production of water equal to 1200 million liters/day. Since the inception of the water supply system, about 295 wells have been drilled in Dhaka city out of which about 35 percent wells are not functioning at present.

An advanced knowledge of both of the parameters ofthe well and the aquifer is needed to analyze the transient hydrologic budget in the saturated portion of a groundwater basin.

Objectives of the Study

The pumping tests conducted by Dhaka WASA have never used piezometers to determine its time drawdowns at distances. As such this study will apply those data as a single well pumping test data to fulfill the following objectives:

- To assess the Efficiency of the selected wells in Dhaka City.
- To determine the Specific capacity of the selected wells.
- To assess the aquifer characteristics such as Transmissivity and Storativity by using pumped well data.

MATERIALS AND METHODS

General flow equation of groundwater movement is derived from Darcy’s law and the equation of continuity. Well flow equations derived from this flow formula are the effective tools to understand the flow of water towards a well. Through such simplifications, a number of flow formulas have been developed depending on different flow conditions, geological variations and different boundary conditions. The Dhaka WASA deep tube-wells have the construction and development test data. The measured drawdowns are available in a well itself. Thus the aquifer properties were obtained with the help of single well principle.

Collection of Data

This study involves an analysis related to deep tubewell construction and development data. The data generated from Dhaka WASA wells for the design, construction and development are preserved for each well in DWASA office. The step drawdown test data were used in this study considering it as a single well test.

Selection of Tube-wells

Based on the following criteria 50 wells in the 10 zones (5@ each zone) of Dhaka city are selected.

- The number of selected wells in a zone should be representative of that zone
- The wells are selected according to their time of construction, so that, they can represent the changing phenomenon with respect to time.
- Data must be available for the selected wells.

Single Well Pumping Tests

No piezometers are used in a single well test. Water level changes during pumping are measured only in the well itself. The methods(Kruseman et al.,1990) applied in the present study are:

1) Hantush-Bierschenk's method
2) Jacob's Straight Line Method
3) Papadopulos Cooper's Method
4) Hurr-Worthington Method
5) Logan Approximation Method

To determine Transmissivity and Storativity both of these methods are applied by using AQTESOLV software

Properties of Well

Well Efficiency: $E_W = \frac{B_1 Q}{(B_1+B_2)Q+CQ^2} * 100\%$ (1)

Specific Capacity: $Q/s = \frac{1}{(B_1+B_2)+CQ}$ (2)

Well Loss: $L = BQ$ (3)

Applying the Hantush-Bierschenk's method,the drawdown $s_{w(n)}$ in a well during the n-th step of a step-drawdown test as,

$$\frac{s_{w(n)}}{Q_n} = B(r_{ew}, \Delta t) \quad + CQn \qquad (4)$$

The following procedures are usedto determine the properties of the well;

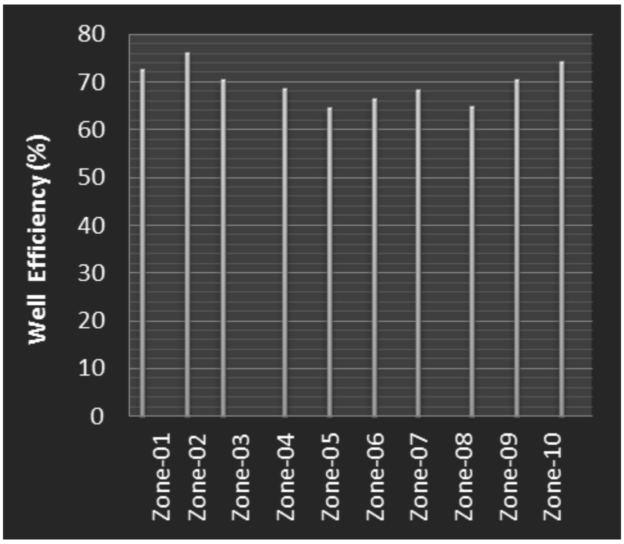
- Time-Drawdown Curve (From Collected Data)
- Determination of Specific Drawdown
- Determination of Well Properties [Refer to "Eq. (1), (2) & (3)"]
- Determination of the Parameters B and C [Refer to "Eq. (4)"]

Properties of Aquifer

To determine Properties of Aquifer (Transmisivity(T) &Storativity(S)), four methods are used. Among these methods Jacob's Straight line Method and Papadopulos-Cooper's Method are applied by using AQTESOLV software. AQTESOLV simulates the time-drawdown data with respect to the specified method. The calculation of the properties of the aquifer are easily analysed with AQTESOLV. Two other methods such as Hurr-Worthington Method and Logan Approximation Method are also used to find the required properties of the Aquifer by using the following equation;

ForHurr-Worthington Method; Transmissivity, $KD = \frac{r_{ew}^2 S}{4tu_w}$ (5)

For Logan Approximation Method; Transmissivity, $KD = \frac{1.22Q}{s_{mw}}$ (6)



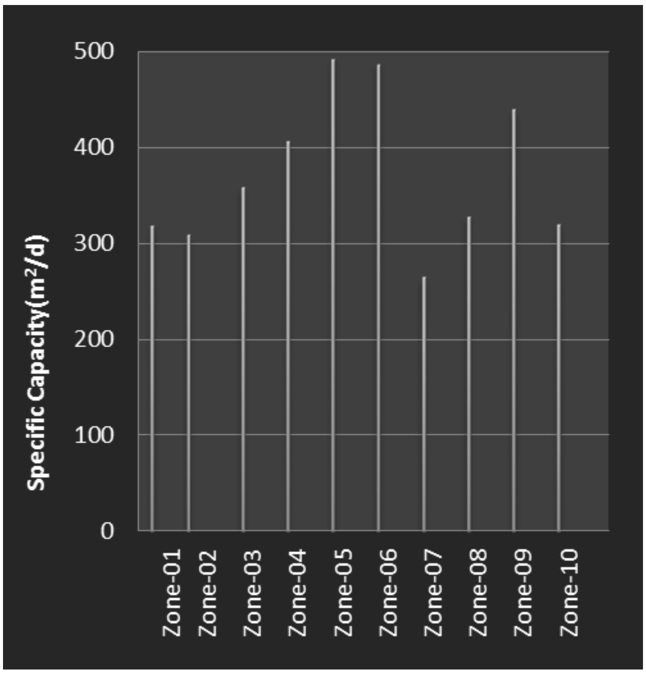
[Fig. 1]: Zone-wise variation of Well Efficiency

RESULTS AND DISCUSSIONS

Properties of Well

The well efficiency represents the productivity of the well only and its evaluation requires the difference of total drawdown into its aquifer and well loss components.Low efficiency as 70% is the indication of inadequate and poor well development.The variation of well efficiency is shown in Fig 1. Zone-2 represents the highest efficiency (76%) while Zone-05 signifies the lowest one(65%).

Specific capacity is important to measure the performance of a well. In Dhaka city, the specific capacity of the well varies from 224 m^2/day to 551 m^2/day.Fig 2 represents the zone-wise variation of Specific Capacity. Zone-5 gives the highest value while Zone-7 is in the lowest position with respect to specific capacity.Well loss arises from the combined effect of screen entrance loss and frictional loss inside the pumped wells. The total well losses varied from 3.52 m to 12.73 m.Well losses were found appreciable in Dhaka city compared to its overall drawdowns.

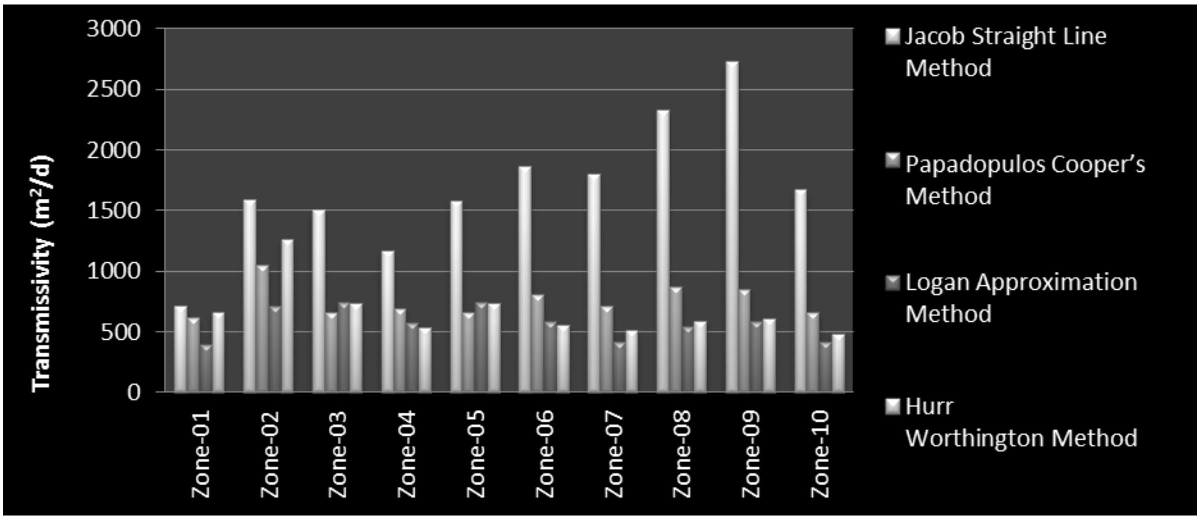


[Fig. 2]: Zone-wise variation of Specific Capacity

Properties of Aquifer

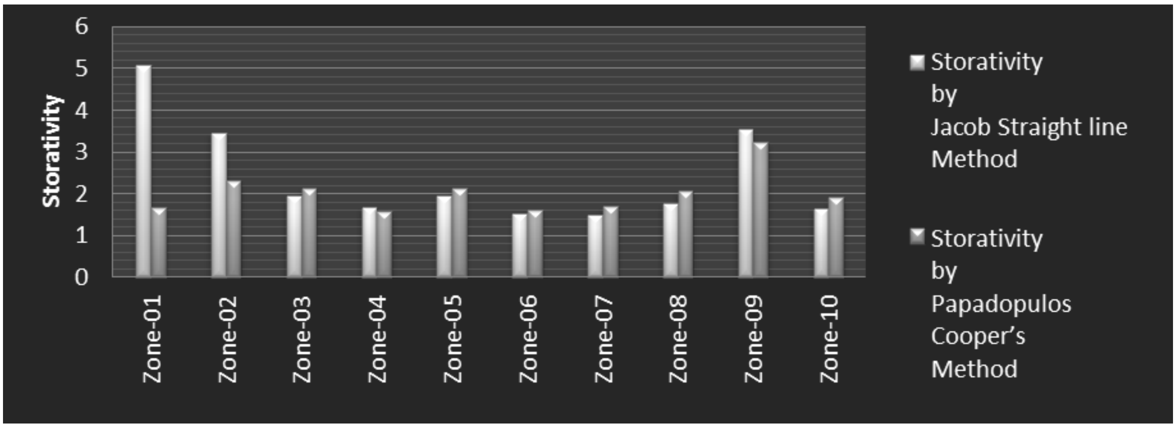
Transmissivity

Determination of transmissivity are therefore constrained with the utilization of the well tests under the confined aquifer system. According to Fig3, Jacob's straight line method gives consistently higher transmissivity values than other methods.Jacob's method is very sensitive to the non-linear well losses which have been found in appreciable magnitude for the most of the wells tested in Dhaka city.



[Fig. 3]: Zone-wise variation of Transmissivity

Storativity



[Fig. 4]: Zone-wise variation of Storativity

For the selected single well methods, the storativity values can be determined by the Jacob's Straight Line Method and Papadopulos-Cooper's method by using AQTESOLV software. Papadopulos Cooper's method gives better storativity result than Jacob's Straight line Method. That's why Papadopulos Cooper's method is chosen for comparison with previous studies.

CONCLUSION

Out of 50, 29 wells represent above 70% well efficiency which is not acceptable because low efficiency as low as 70 percent is the indication of inadequate and poor well development.Almost 60% wells in Dhaka city exhibit non-linear well losses above 0.5 m. Hurr-Worthington's method is selected as the appropriate method to calculate transmissivity for Dhaka WASA wells. But, under the single-well principle, Papadopulos-Cooper's method is used by DWASA to calculate storativity. In case of Zone-7 to Zone-10,most of the wells are newly constructed. For that reason the wells of these zones represent a higher percentage of well efficiency and specific capacity.

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APPLICATION OF DELFT3D MATHEMATICAL MODEL IN THE RIVER KARNAFULI FOR TWO-DIMENSIONAL SIMULATION

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ABSTRACT:

Karnafuli river, one of the most important rivers of Bangladesh which is playing a vital role in our national economy. The Chittagong port on Karnafuli river is considered as the lifeline of the economic activities of the country. Activities of Chittagong Port Authorities (CPA) and other organizations along the bank of the river Karnafuli is increasing day by day. Therefore, it is always a vital issue to keep the river active and live in terms of its navigability. Due to manmade interventions the river flow becomes interrupted and thereby may cause the change in river morphology. In order to study the effect of any intervention in the river, application of 2D (Two dimensional) model in the river Karnafuli is inevitable. This study focuses mainly on the application of 2D model to assess different hydrodynamic characteristics of the river. The model has been set with the recent bathymetry data collected from CPA hydrography division. The river reach between Kalurghat and Khal no-18 has been selected for model set up. The model uses a curvilinear orthogonal grid with variable dimensions of grid cells starting from 58 m up to 166 m. Calibration and validation is done against the water level data for the year 2009. Model simulation result includes flow (velocity) field, bed shear stress etc have been analyzed to know the hydrodynamic behavior of the river. The present model of Karnafuli river provides an opportunity to assess the impact of different development scenarios to be undertaken on river.

Keywords: Karnafuli river, River flow, Hydrographic, Flow velocity.

INTRODUCTION:

Karnafuli river**,** which is the major river of Chittagong, originates from Lushai hills in Mizoram state and flows about 270 km south and southwest through the southeastern part of Bangladesh to reach the Bay of Bengal. During this course this meandering river passes Kaptai hydroelectric power plant, Halda-Karnafuli confluence and several bridges [Fig:1]. The length of the river from Kaptai Dam to Halda-Karnafui confluence is about 45 km and from Halda-Karnafuli confluence to BN Aacademy is about 30 km. Karnafuli river is a tidal river having semi-diurnal characteristics. During flood period the flow travels long distance in the upstream direction of Halda river and very near to Kaptai Dam in upper Karnafuli river.

This study mainly focuses on the lower Karnafuli river spanning from Kalurghat to Khal no-18. Lower Karnafuli river is the most important portion of the whole river due to the vast economic activities. Regular maintenance of this portion of river is necessary to keep it navigable for safe transportation of vessels. Recently dredging works at different locations such as Sadarghat is running to maintain sufficient draft. Thus the application of a 2D mathematical model of lower Karnafuli River is highly envisaged.

Fig 1: Karnafuli and Halda river network

Maximum flow and shear stress during spring and neap tide have been computed at different locations. To know and predict the navigability and erosion pattern it is important to know the hydrodynamic and morphological characteristics of the river which is reflected by its flow velocity, shear stress and sediment transport. Previously various studies were conducted at different time to assess the behavior of Karnafuli river (Danish, 1990, CPA, 1987). This study gives an idea of the hydrodynamic characteristics of Karnafuli river and future application of 2D mathematical model to evaluate the behavior of the river under several scenarios.

METHODOLOGY:

A 2D hydrodynamic model has been applied to simulate Karnafuli river hydrodynamics. The model bases on the Delft3D modeling system (WL | Delft Hydraulics, 2001). Delft3D consists of different modules such as Flow, Wave, WAQ. For this study the flow module is used which is a multidimensional (2D or 3D) hydrodynamic and transport simulation program. This module is capable of calculating unsteady flow and transport phenomena resulting from tidal and meteorological forcing on boundary fitted curvilinear grid. It is also possible to take into consideration some other parameters such as temperature, salinity and different constituents and observe the 2D or 3D distribution of the result (Delft, 2001).

Mesh generation and Bathymetry:

In Karnafuli the 2D model has been applied to calculate the flow velocity distribution, shear stress and sediment transport. These results are the pre-requisite for in depth analysis of different phenomena. This model is based on the non-stationary equations of motion and continuity. The equations are solved numerically for boundary fitted orthogonal curvilinear grid of the modeled area. A horizontal curvilinear grid has been generated for the lower Karnafuli River. The horizontal grid consists of 2604 cells and uses Cartesian co-ordinate system [Fig:2].

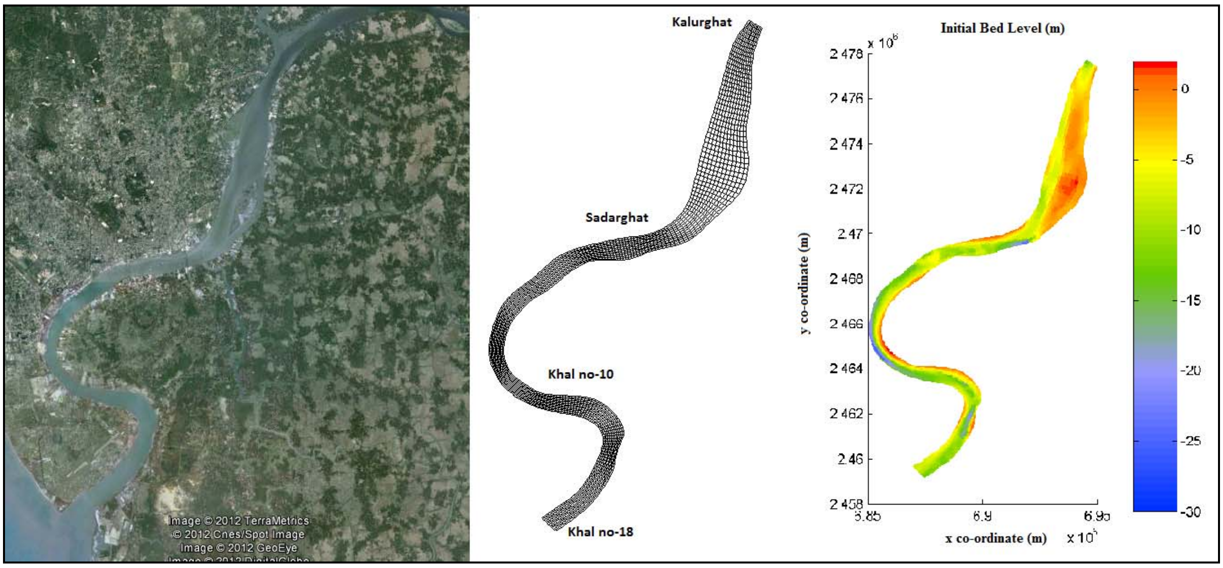

Fig 2: Lower Karnafuli river (left),2D cuvilinear grid(center), Bathymetry(right)

The cell dimension varies from 58m to 166m. The smaller cells are applied between Sadarghat to Khal no-18 where large velocity gradients are expected. Bathymetry data is collected from CPA, it was used to generate bathymetry file for Delft3D after a series of processes such as grid cell averaging, triangular interpolation and internal diffusion. The computational time step is equal to 1 minute.

Boundary condition:

The model consists of two open boundaries. Upstream boundary is taken at Kalurghat while 6 hourly discharges have been entered as upstream boundary condition. On the other hand downstream boundary is selected at Khal no-18 and the boundary condition is hourly water level data. Upstream discharge is quite high in monsoon season maximum value varies between 4500 m^3/s in spring period to 3500 m^3/s in neap period while for dry period these are 2500 m^3/s and 1500 m^3/s. Downstream water level varies approximately between 3.3 m to 5.2 m w.r.t ISLWL(Indian Spring Low Water Level) throughout the year.

RESULTS AND DISCUSSION:

The step first step involved after running the model successfully for the whole year was to calibrate and validate it with observed data. Water level data of Sadarghat station and Khal no-10 was collected from CPA which was used for calibration and validation. The model was calibrated against water levels for the month November (post-monsoon condition) and June (monsoon condition) for Khal no-10 [Fig. 2]. Calculated and measured water level was found quite close (Fig:3,4). After calibration the model was validated [Fig:4] at Sadarghat section for the month November (monsoon condition)[Fig:5].

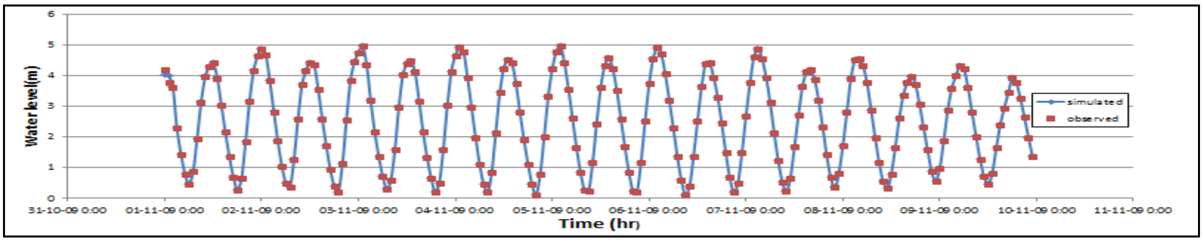

Fig 3: Calibration of the model at Khal no-10 for the month November

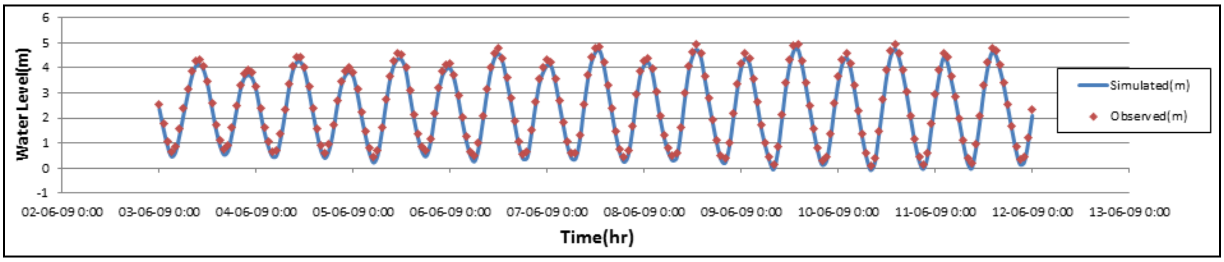

Fig 4: Calibration of the model at Khal no-10 for the month June

The calculated and observed water level showed quite good agreement with slight variation at some locations throughout the year. The model predicts the water levels quite well, sometimes it is little bit overestimated. Variable roughness values are used throughout the model which acted as the calibration parameter. Roughness values used ranges between 0.025 to 0.029 (Manning's roughness).

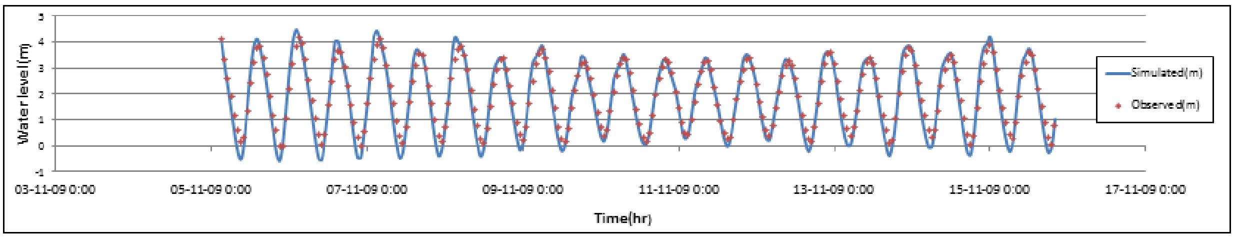

Fig 5: Validation of the model at Sadarghat for the month November

The model is capable of showing various parameters such as depth average velocity, bed shear stress, water level, vorticity, enstrophy at various points or sections. For this study we have shown velocity of four points (Fig:6) for the month October(1-17 October). Maximum simulated velocity amongst all the four points is 1.65 m/s and the minimum velocity is 0.5 m/s.

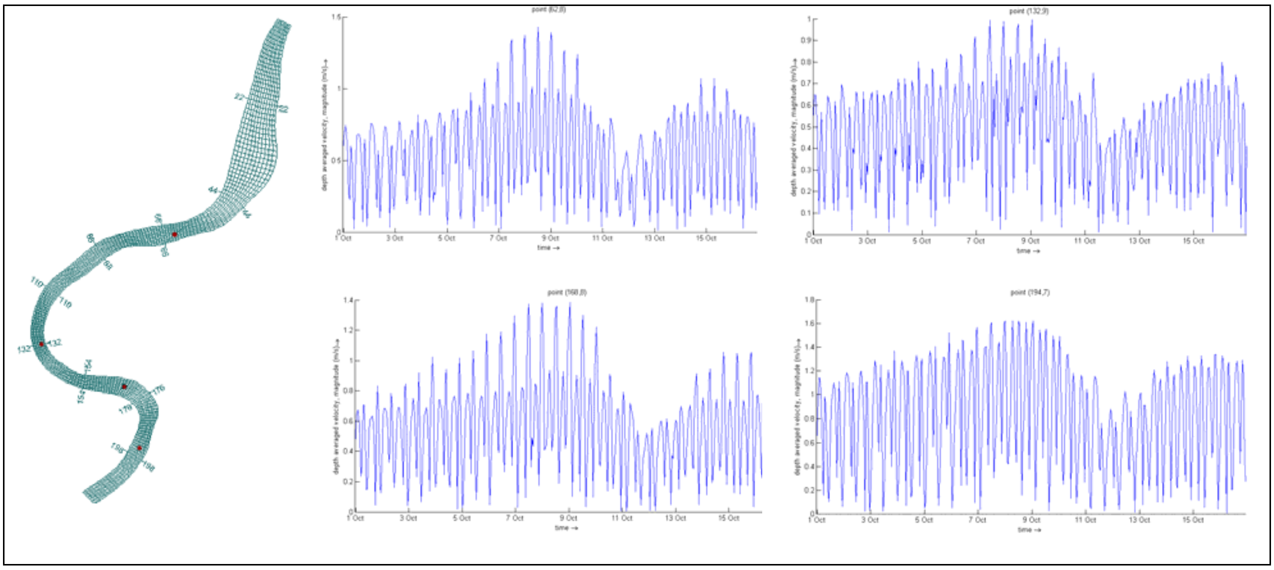

Fig 6: Magnitude of depth average velocity(m/s) for1to17 October(left),location of the monitoring points(right)

Bed shear stress is another important hydrodynamic property of river. For Karnafuli river bed shear stress at four points [Fig:6] has been shown for October 1 to 15. Within this period maximum simulated shear stress is 6.4 N/m^2 which occurs near Sadarghat on 8 Oct 2009- 6 am [Fig:7]. Bed shear values give an idea of the locations which are much susceptible to erosion. From this simulation the susceptible regions found are mostly concave sides of each loops and places having shallower depth as shown in [Fig:7].

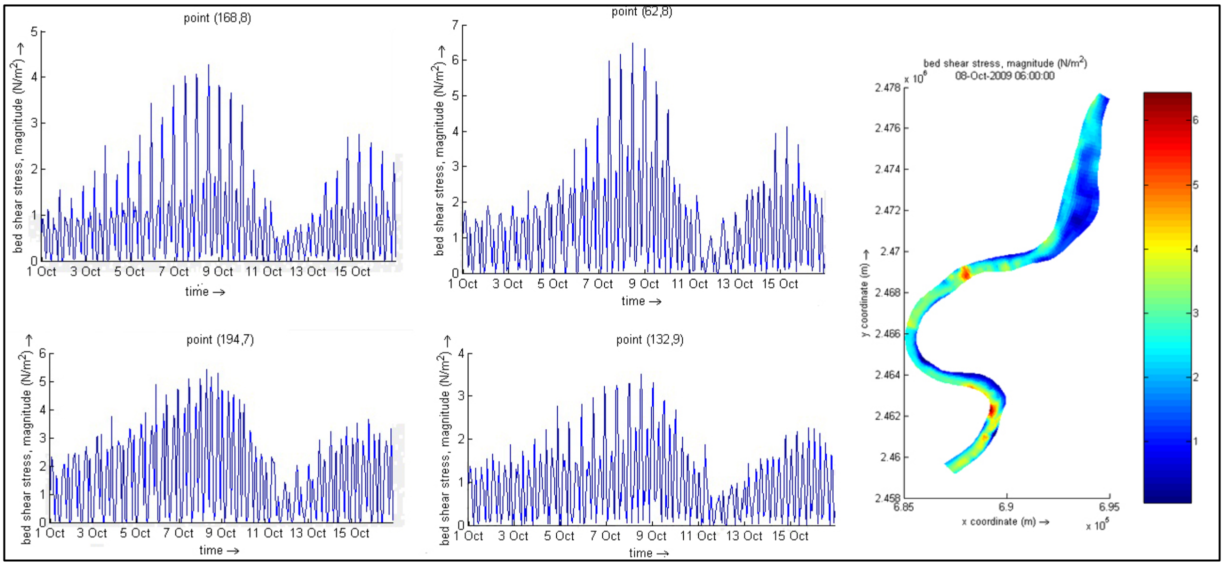

Fig 7: Bed shear stress (N/m^2) for 1to17 October (left), Maximum shear stress condition (right)

CONCLUSION:

Two-dimensional (2D) depth averaged model has been developed for the lower Karnafuli river. Calibration and validation of the model shows significant compliance with the observed data. Model results are generated for velocity filed, shear stress, and water level at various locations. Further use of this model is to asses the morphological changes such as erosion or sedimentation of the river for ebb and flood tides. Through the application of this model, the change in river behavior under different scenarios can also be assessed for proper planning of any development project on Karnafuli River.

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DEAD ZONE CREATING STRUCTURE IN RIVER FOR RESTORATION OF FISH HABITAT

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ABSTRACT

Fishes are good indicators of the environmental health of rivers and their catchments as well as important conservation targets. To meet with increasing population demand humans destruct the habitat places of small native fishes and their fish extinctions face in danger. The main stream of a river, where the velocity is high, is not suitable for weak and small fishes. To protect our small native fish, alternative habitat can be created in river by installing side cavities, enlarged zones or groyne type structures that create dead-zone like flow field at the side of the main stream. The low velocity zone created by such structures can be used as safe zone for young and small fishes. In this study, firstly the relation between the sustainable water velocity with the size of fishes is analyzed based on previous research. Then the flow fields of different dead-zone creating structures are simulated and analyzed. Based on the analysis, the applicability of such structures in river for restoration of fish habitat is explained.

INTRODUCTION

Freshwater species of many taxa are declining due to increased stresses placed on freshwater resources from human populations. Habitat destruction and alteration have been identified as the primary reason for the decline of fish species and other aquatic organisms (Matthew, 1985). Matthews also reported that the body form of the fishes may be as important as size in determining critical current speeds. For this reason small fish production faces a horrible danger and it also affect their extinction. To maintain ecosystem balance it has become important to protect these fishes by creating alternative shelter. An example is given below:

In case of natural channel if we consider Brahmaputra-Jamuna, the mean annual discharge of the river is 20,000 cumec, and the mean velocity is 1.70 m/sec with a average depth of 6.6 m. Flow velocity at main stream is intolerable for all small and young fishes, because they cannot hold their position in this high velocity region of flow. To solve this problem, low velocity region can be established by constructing dead zone creating structures along the river width. 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 is 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. The flow fields of these types of dead zone creating structures have been reported by many researchers

(Kimura and Hosoda, 1997; Muto, et al. 2002; Mizumura and Yamasaka, 2002; Uddin et al., 2011; etc.) considering their engineering applications. However, as far as authors' knowledge, no research has been reported on the applicability of such structures in increasing the biodiversity of aquatic species by creating habitat and providing shelter for them. In this study, firstly the relation between the sustainable water velocities with the size of fishes is analyzed based on previous research. Then the flow fields of different dead-zone creating structures are simulated and analyzed. Based on the analysis, the applicability of such structures in river for restoration of fish habitat is explained.

METHODOLOGY

The methodology of this study contains two parts: one is to determine the sustainable velocity of water with respect to body size of fishes, and another one is to analyze the suitability of dead zone creating structures to create shelter for small and young fishes. Following two subsections explains the methodology employed for this study.

Sustainable Velocity of Fish Habitat

Based on previous studies the relation between sustainable velocities with body size of fishes was reported in this study. The experimental results by Layher (1993) were used in this study. The experimental system he used consists of a three-inch clear PVC pipe, where the flow of water is made using a submergible pump. A petcock at one stage allows air to bleed from the tubing. An access plug located at the end of the clear pipe allows fish entry and exit to the system. A strainer at each end prevents fish from moving downstream or upstream out of view. Flows are diverted back into an aquarium equipped with temperature control. Individual fishes were placed into the testing apparatus; then allowed to acclimate to a low velocity. The fish was again acclimated for approximately two minutes. Eventually a point was reached at which the fish encountered some difficulty maintaining position. This point is referred to simply as "difficulty". Velocity was again increased until the fish was swept through the tube with no recovery upstream. This point was termed "cannot hold". The velocities produced by the apparatus ranged from 11.124 cm/sec to 72.890 cm/sec.

Flow Field in Dead Zone Creating Structures

Three–dimensional numerical simulation is carried out to study the fundamental properties of flows in an open channel with dead zones. In this paper, the flow field in three types of dead zone creating structures have been studied, which are Rectangular Side Cavity, open channel with Sudden Enlargement and dead-zone created at the downstream of a Barb. A non-linear k-ε model is employed using finite volume method with a curvilinear coordinate system. Based on the simulated result, the low velocity region in the flow field was identified as the potential shelter for young and small size fishes.

RESULT AND DISCUSSION

In this study, firstly the relation between the sustainable water velocities with the size of fishes is analyzed based on previous research. Then the flow fields of different dead-zone creating structures are simulated and analyzed. Based on the analysis, the applicability of such structures in river for restoration of fish habitat is explained.

Flow Field in Dead Zone Creating Structures

The mean water velocities where the fishes show difficulty to stay and cannot hold their position with respect to their body size is analysed here based on the previous experiments. The indivisual and average trend of sustainable velocity for four different fish species is presented here using the experimental results by Layher (1993). The fish type includes *Etheostoma blennoides, Etheostoma radiosum, Campostoma Anomalum and Cultured golden shiners*, whose length were varied from 3.0 to 11.5 cm. The first two species had the length less than 7.5 cm and latter two fishes had the length up to 11.5 cm.

Figure 1 shows the relation between the mean water velocities where the experimented fishes shows difficulty to sustain in their position with respect to their body size. Figure 2 shows the critical velocity where the fishes cannot hold their position. Both the figures show that the sustainable velocity for all the indivizual fishes depends on their body length. For example, the *golden shiners* fishes, which are about 4 cm long cannot sustain in a velocity of more than 30 cm/sec, although those fishes with a body length 10 cm can sustain in a velocity more than 60 cm/sec. Similar relation is observed for other types of fishes. The sustainable velocity for some *Etheostoma blennoides* fish of 3 to 4.5 cm length is about 25 to 35cm/sec.

If an average trend line is drawn considering all the fish species, it can be concluded that the sustainable velocity of fishes is linearly related to their body length. The average "difficulty" velocity of fish is found about 5.8 times of their body length and the "cannot hold" velocity of fish is about 6.4 times of their body length.

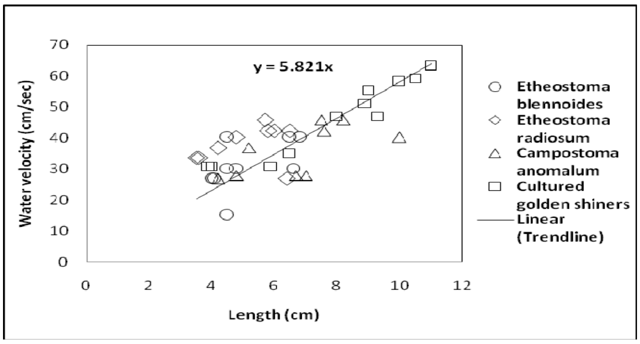

Figure 1: Relation between the mean water velocities where the experimented fishes shows difficulty to sustain in their position with respect to their body size.

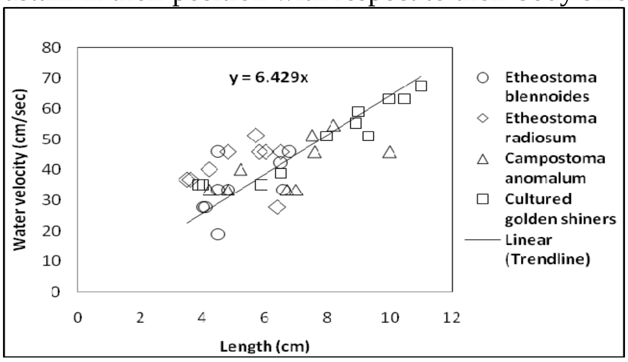

Figure 2: Relation between the mean water velocities where the experimented fishes can not hold their position with respect to their body size.

Flow Field in Dead Zones Creating Structures

In this study, the simulated flow field in three types of dead zone creating structures have been studied, which are Rectangular Side Cavity, open channel with Sudden Enlargement and dead-zone created at the downstream of a Barb. A FORTRAN code based on navier-stokes equation (Ali, 2008; Kimura et al., 2009) is used for simulating the flow field of this study. The results are presented below.

Table 1: Hydraulic parameters for the simulations of open channel flows with (i) a rectangular side cavity, Case C1 (ii) a sudden enlarged zone, Case E1 (iii) Barb, Case B1

Case no.	B (cm)	L (cm)	W (cm)	L/W Ratio	Q_0 (cm^3/s)	h_o (cm)	Fr no.	Bottom slope
C1	10	22.5	15	1.5	747.0	2.02	0.83	1/500
E1	10	135	15	9	255	1.00	0.81	1/1000
B1	80	20	20	-	0.028×10^6	4.4	0.977	0.005

Rectangular side cavity

It is an enclosed portion in a side of open channel where there is no direct intrusion of longitudinal flow from upstream and no exit of flow in downstream direction 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 flow visualization in a side cavity (Muto et al., 2002). Figure 2 shows a typical sketch of a rectangular side cavity, where the dead zone like flow is generated.

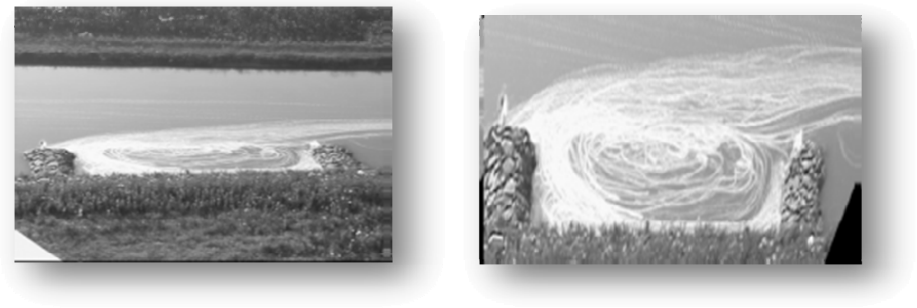

Figure 3 : Example of flow behavior in open channel flow with rectangular side cavity (Muto et al., 2002)

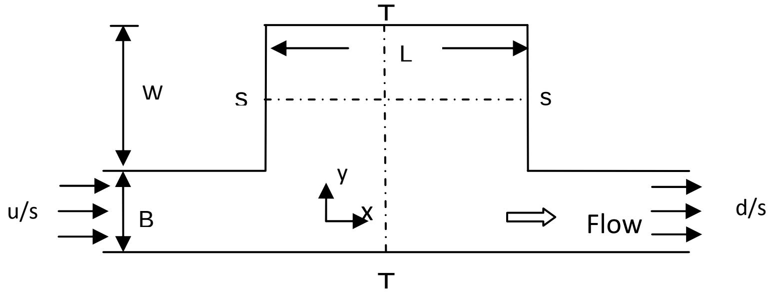

Figure 4 : Sketch of the flow domain with rectangular side cavity (L=length of the cavity**,** W=width of the cavity, B= width of the main channel)

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. Figure 5 shows the simulated time averaged flow field. The stream-wise velocity (u) along the transverse cross-section at centerline of dead zone (T-T section as shown in Fig. 4) is shown in Fig. 5. To examine the simulation accuracy, he comparison of simulated result with previous experiment also compared and shows good agreement. 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.

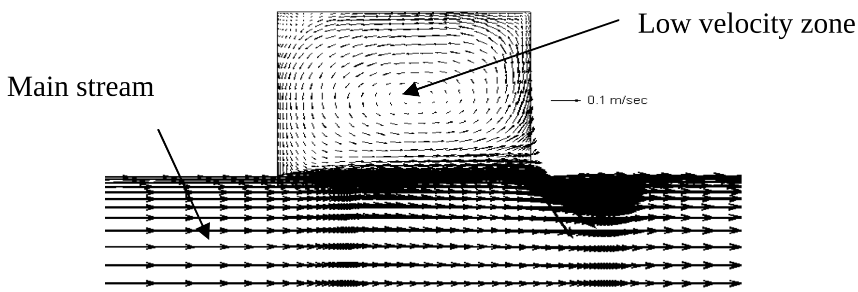

Figure 5: The simulated time averaged flow field in an open channel with a rectangular side cavity.

Enlargement in an open channel

Fig. 6 shows the definition sketch of open channel with sudden enlargement. The position of low velocity region (i.e. dead zone is also indicated in the sketch). In this case of width 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. 7. The main flow is observed to be deflected towards the enlargement (left side). A main circulation is observed in the enlarged portion of the channel, where the flow velocity is very small compared to main stream, which is called the dead zone area. This type of encroachment causes the increase of channel cross-section, and thus the conveyance capacity of the river. It also causes the morphological changes of the river course due to the deposition of sediment at the enlarged area. Since the dead zone is a low velocity region, the sediment carried by the stream can easily be deposited in that region.

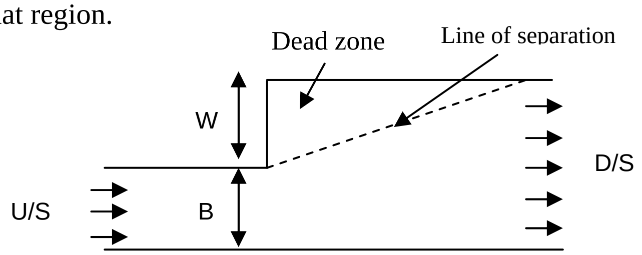

Figure 6 : Definition sketch of open channel with sudden enlargement (B=Encroached width, W=Enlargement in width)

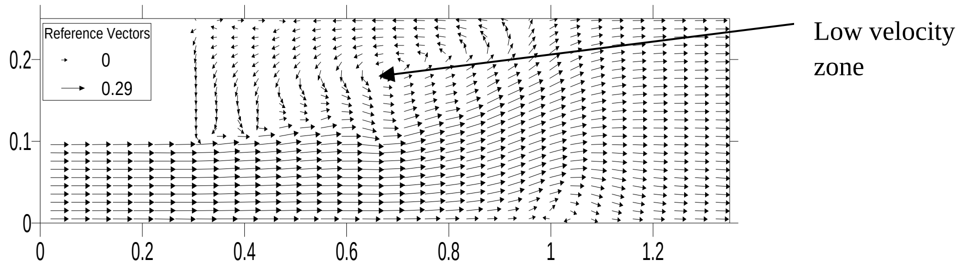

Figure 7: Simulated flow field of an open channel with width enlargement.

Flow field at the Downstream of a Groin/Barb:

Stream barbs are basically 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 are used for the purpose of deflecting current away from the bank and minimizing the erosion potential. 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 pointed upstream the submerged weir section forces the water flowing over the structure into a hydraulic jump. 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. Figure 8 shows a typical sketch of the barb structure.

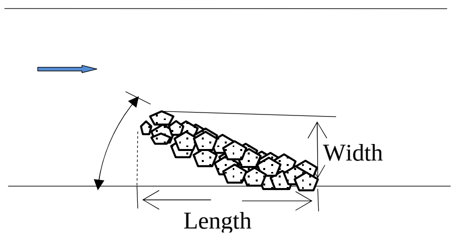

Fig. 8: Sketch of the barb prototype

Fig. 9 shows the simulation effect of velocity vector at the water surface by installing single barb after 5 hours. 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 low velocity 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.

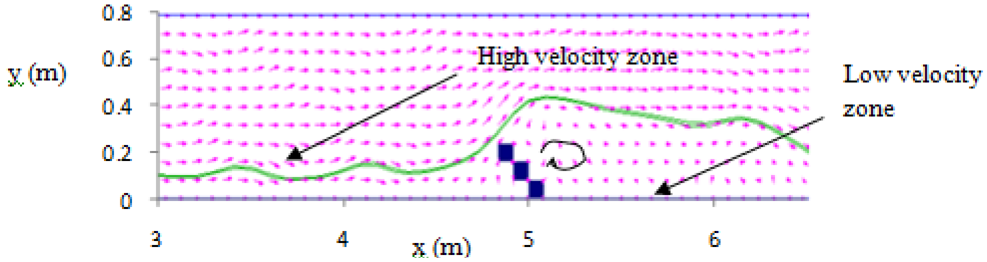

Fig.9: Velocity field with a barb installed at one sidewall of the channel after 5 hours simulation.

Applicability of Dead Zones Creating Structures for Restoration of Fish Habitat

The research shows that sustainable water velocity of fishes varies linearly with respect to their body length. Small and weak fishes cannot hold up in a high velocity of a stream, and generally the main stream of a river is under this category. Therefore, they need a low velocity region for their habitat. For example, from the experiment it is found that the sustainable velocity for some *Etheostoma blennoides* fish of 3 to 4.5 cm length is 25 to 35cm/sec. Therefore these fishes will search for a place where the velocity is below 25 cm/sec. In this study, the flow field of some dead zone creating structures have been studied to examine their applicability in reestoring fish habitat, espacially for small and young specieses those cannot sustain in the high velocity of main stream.

CONCLUSION

The indivisual and average trend of sustainable velocity for four different small fish species is presented and based on the result it can be concluded that the sustainable velocity of fishes is linearly related to their body length. The average “difficulty” velocity of fish is found about 5.8 times of their body length and the “cannot hold” velocity of fish is about 6.4 times of their body length. Since the sustainable velocity of such small fishes is low, they need suitable shelter for their habitat. The flow field of some dead zone creating structures have been studied to examine their applicability for providing such small and young species those cannot sustain in the high velocity of main stream. Open channel with Side Cavity, Sudden Enlarged Zone and downstream of Groin have been studied as the probable structures. It is found that they can create dead zone at the downstream of the structures, where the flow velocity is very small compared to the main stream. Such dead zone creating structure can be used for restoration of fish habitant by providing shelter for them.

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EXPERIMENTAL STUDY OF REDUCING BED SHEAR STRESS ON THE DOWNSTREAM SLOPE OF EMBANKMENT WITH VEGETATIVE BARRIER

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ABSTRACT
Overflow is the most damaging factor for earthen embankments on the downstream (d/s) slope due to the erosion of surface material which is caused by the high bed shear stress and supercritical flow over the entire d/s slope. Vegetation planted on d/s slope may have a considerable effect in reducing bed shear stress which makes erosion. However, the influence of vegetation for flow resistance as well as bed shear stress control has been studied a little. Therefore, the aim of this research is to introduce the use of natural resources like vetiver grass for controlling bed shear stress on the high flow region. A small-scale experiment is performed with four different densities of vegetal cover considering different discharges on d/s slope. Vegetative barrier affect the mean flow rate by induced drag force against hydraulic forces. Hydraulic characteristics and mechanism of vegetation in open channel during overflow are studied through flume experiment. Results demonstrated that vegetation markedly reduced the effective bed shear stresses which are the influencing factor for causing erosion on the d/s slope, toe and bed of embankment. Results also revealed a significant reduction of damage due to erosion and risk of failure from overtopping flow can be achieved by providing dense vegetation cover on the d/s slope of earthen embankment.

Keywords: Earthen embankment, Overtopping flow, Drag force, Bed shear stress, Vegetation.

INTRODUCTION

Bangladesh, with its repeated cycle of floods, cyclones, and storm surges, has proved to be one of the most disaster-prone areas of the world. River bank and coastal erosion, and embankment failures happen continuously throughout Bangladesh. These are endemic and recurrent natural hazards, causing loss of lands and livelihoods along major rivers and coastlines. From a strictly economic point of view, the cost for mitigating these problems is high. In addition, the State budget for such works is never sufficient which confines rigid structural protection measures to the most acute sections, never to the full length of the river bank or coastline and embankment. This bandage approach compounds the problem. Not only that, hard engineering structure makes the scenic environment unpleasant and helps only to transfer the problem to another place, to the opposite site or downstream, which aggravates the problem rather than reducing (Grimshaw, 2008). Establishment of vegetation as a soft bioengineering technique to rigid or hard structures accepted all over the world due to its low cost and longevity.

Bangladesh Water Development Board (BWDB) has been constructed nearly 13000 km of embankments (over 4000 km of coastal embankments along the coastline surrounding the Bay of Bengal and offshore islands, nearly 4600 km of embankments along the bank of big rivers and nearly

4500 km of low-lying embankments along the small rivers, haors and canals). A large number of sea dike or river embankment failures have been initiated from damages induced by wave or flow overtopping (Islam, 2000). For earth coastal dikes or river embankments, overtopping is one of the most damaging factors for the downstream slope. Eventually a failure of the downstream slope may lead a failure of the dike. Water infiltrates into dike crest, downstream slope and reduces the shear resistance of the soil (Hanson and Temple, 2001). An eco-friendly vetiver grass (Vetiveria zizanioides) has been used successfully over 120 countries for more than a century as traditional technology for riverbank stabilization and embankment erosion control (Truong and Loch, 2004). Recent research and case studies have shown that vetiver grass can be used as a cheap method to protect shorelines or embankments. Mature vetiver is extremely resistant to washouts from high velocity flow due to its extraordinary root depth and strength. The stiff shoots and strong roots can keep the plant stand steadily in water with 0.6-0.8 m deep and 3.5 m/s velocity of water flow (Ke et al., 2003). On this subject, Bangladesh is in advantageous position as it has been abundant supply of vetiver grass. But vetiver grass technology (VGT) is very new in Bangladesh and using vetiver as a living barrier against erosion remains untouched excepting as few small-scale trials in some foreign aided projects. Moreover, its application has been based on experience rather than hydraulic principles (Das and Tanaka, 2009). Basically, the upstream side of earthen sea dikes or embankments are armored with stones or concrete blocks where there any exist of important infrastructures while the downstream slope often covered with grass. Very little is known about the strength and stability of downstream slope of sea dikes or river embankments covered with grass during overflow. To minimize the impact of natural disasters as well as to achieve the aim of agricultural production, sustainable and cost-effective maintenance of those embankments is a sine qua non for Bangladesh.

MATERIALS AND METHODS

Experimental Set up and procedure

A small-scale laboratory flume experiment is performed to investigate the effects of vegetation (commercially available vegetation model with 5% and 20% blocking) roughness on the behaviour of d/s bed shear stress. Keeping those in mind, four different densities of vegetal cover considering different discharges on the d/s slope are used. Firstly, row vegetation (2D type) and then grid type 3D vegetation is used. Afterward the experiments are conducted with staggered type 3D vegetation. All these experiments is conducted with maintaining a ratio of 0.25 (5cm spacing case) and 0.75 (15cm spacing case) with 20% uniform blocking, where, ratio= width of spacing within the vegetation rows over width of vegetation rows, here 5cm and 15 cm spacing and 20 cm fixed vegetation width is used. Finally, the test is conducted with all over vegetation on d/s slope and bed with 5% and 20% blocking respectively, to evaluate the comparative results with no or WOV case in case of different vegetation effects as well as to investigate the effects of vegetation density for different blocking effect for controlling the bed shear stress on the d/s slope of embankment. A model of wooden embankment was constructed and placed at the middle of the flume which separated the flume along its length, forming main stream on one side (called upstream) and the floodplain on the other side (called downstream). The size of the embankment is 0.25 m in height, 0.25 m in crest width and 1.5 m in length, with slopes 3H:1V and 2H:1V in the upstream and downstream sides, respectively as shown in figure 1. To understand the processes and properties of hydraulics of overtopping flow on the downstream slope of the embankment with the vetiver hedges, a small-scale laboratory flume experiments with an embankment using emerged vegetal cover were carried out with three different unit discharges as 0.018, 0.013 and 0.010 m^3/s (later we called it discharge 1, discharge 2 and discharge 3) was taken in our experiments. The longitudinal flow velocities and water depths were measured along the centre line of the flume at 26 points with an interval of 0.10 m up to upstream (u/s) crest of the model and later 0.05 m interval from u/s crest to the d/s end of the flume until steady flow condition was established. A scale factor (ratio of a variable in the model to the corresponding variable of its prototype) 0.0625 was kept constant throughout the tests. In the tests the velocities and the water depths were measured for situations with and without vegetation in the flume. The height of the vegetation model was kept constant, 0.05 m and width and length was kept same as the d/s side of the embankment model considering emergent flow conditions.

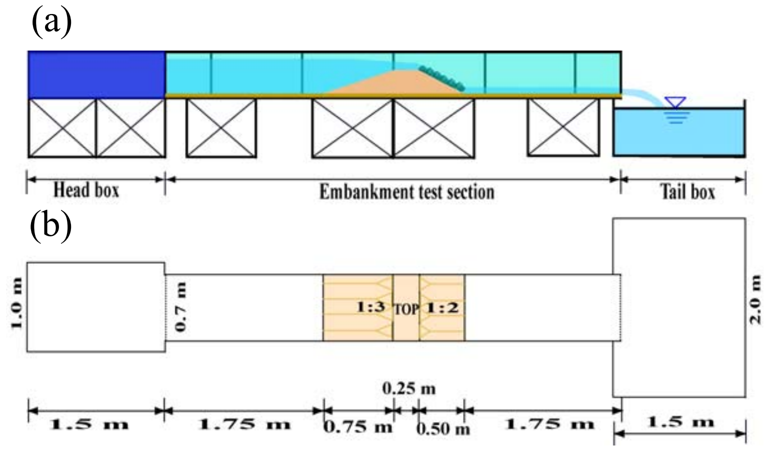

Figure 1. Profile of testing facility (a) front view and (b) top view

Data Collection

The discharges were measured with an electromagnetic flow meter (model: MK -515/ 8510 -XX, paddle flow sensor, Georg Fischer Signet LLC, USA). An electromagnetic velocity meter (type of main amplifier: VM-2000; type of sensor: VMT2–200 -04P, Kenek Company, Ltd.) was used to measure the flow velocities at the centerline of the channel and the model. The water surface elevation was measured at the same locations as the velocity profiles by the point gauges (with accuracy up to 0.1 mm), fixed and mounted on a movable sliding carriage.

Drag force measurement

A two-axis load cell (streamwise (X) and transverse (Y) directions, type LB-60, SSK Co., Ltd.) with a resolution of 1/1000 that can measure a maximum load of 10.0 N was used to measure the drag force on the vegetative roughness model. The measurement technique was almost the same as the method in the earlier study (Takemura and Tanaka, 2007).

Calculation of effective shear stress

During the experiments the steady uniform overflow condition was established at the upstream side of the model and during overtopping flow became unsteady and non-uniform. But for simplicity we consider steady non-uniform flow for calculation of effective shear stress. For a control volume of unit area along the slope the balance of horizontal momentum in this case can be expressed as total shear stress which is equal to the sum of the bed shear stress and the equivalent shear stress due to the vegetation drag:

$$\tau_w = \tau_b + \tau_v \tag{1}$$

where, τ_b= bed shear stress transfer to the soil; τ_w= stream wise component of the weight of water mass and τ_v=resistance due to the drag around the vegetation. The drag force was measured directly by a two-axis load cell apparatus as mention above. The streamwise weight component of the weight of water mass per unit bed area is expressed as,

$$\tau_w = \rho g h i_e (1 - \lambda) \tag{2}$$

where, λ = area concentration due to vegetation.

RESULTS AND DISCUSSIONS

Overtopping flow over earthen embankment in without vegetation (WOV) condition shows that the flow was static at the beginning upstream, accelerating sub-critical flow state over a portion of the embankment crest; through critical flow on the crest and supercritical flow over the remainder of the crest; and supercritical flow on the downstream slope and extend to the further downward same as observed in the previous study (Powledge et al., 1989b). It is clear that the initial erosion process begin anywhere within supercritical flow region due to high bed shear stress, from the point of slope discontinuity i.e. from d/s slope to d/s toe region especially for embankment. The effective bed shear stress (EBSS) which actually transfers to the soil surface is well below the shear stress developed in the case of WOV because of the induced drag force due to vegetation.

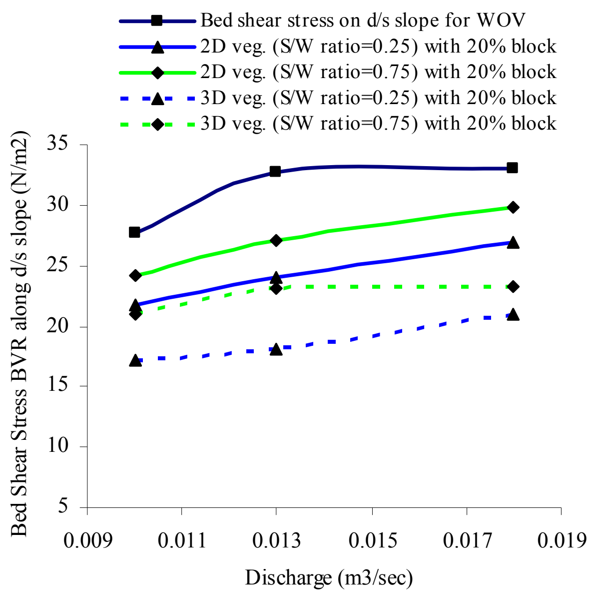

Figure 2. Comparison of effective bed shear stresses behind the vegetation rows (BVR) along the d/s slope in between 2D and 3D type vegetation effect

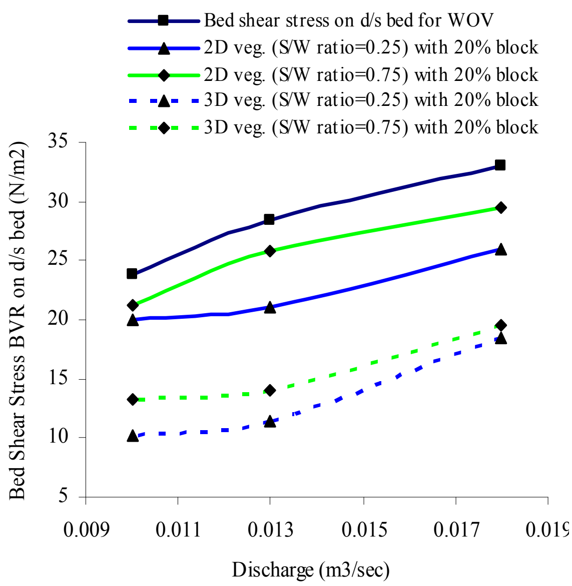

Figure 3. Comparison of effective bed shear stresses behind the vegetation rows (BVR) on the d/s bed in between 2D and 3D type vegetation effect

Figure 2 and 3 illustrate the comparison of bed shear stresses behind vegetation rows (BVR) along d/s slope and bed between the cases of row type (2D type) and grid type 3D vegetation on d/s slope and bed of embankment. The flow velocity behind the vegetation rows is much higher in 2D type row vegetation whereas, in grid type 3D roughness flow accelerates between the vegetation rows and therefore velocity is not much significant behind the rows, results the reduction of bed shear stress is attained on just behind the rows due to grid type 3D vegetation. Where, the reduction of bed shear stress from 28-40% is attained on the d/s slope. Vegetation also reduced the bed shear stress from 45-54% on the d/s bed. In comparison of the BSS behind the vegetation rows (BVR) in between 2D type row vegetation and grid type 3D vegetation, it is seen that the maximum reduction was 40% at d/s slope and 54% on bed, both for the grid type 3D roughness and for 0.25 ratios of rows with 20% blocking.

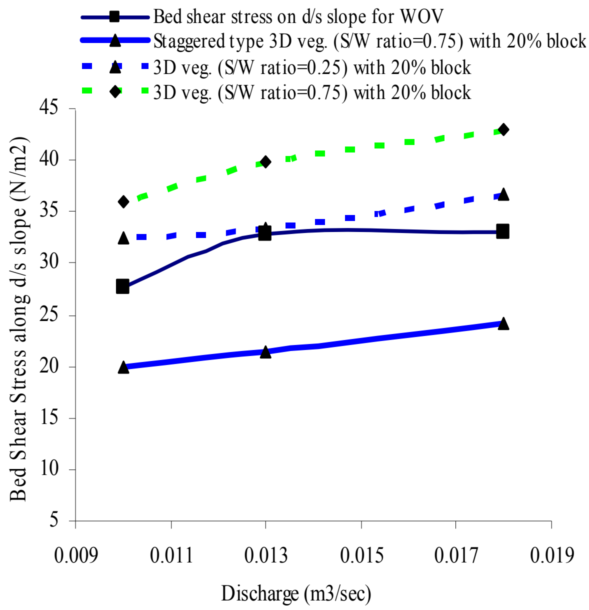

Figure 4. Comparison of effective bed shear stresses along d/s slope in between grid-type and staggered type 3D vegetation effect

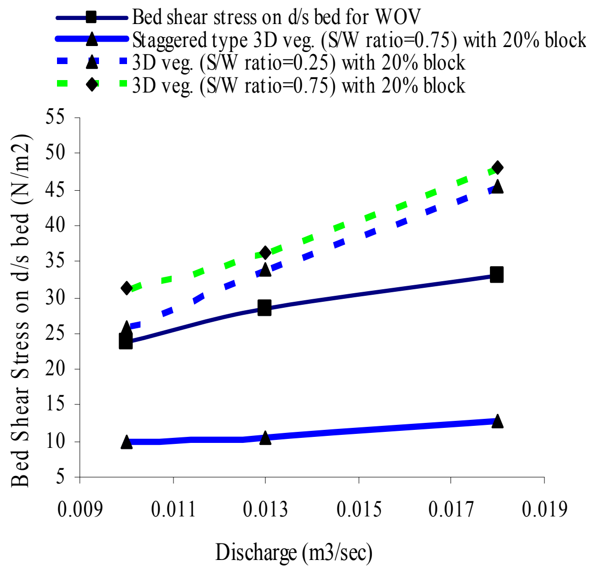

Figure 5. Comparison of effective bed shear stresses on the d/s bed in between grid-type and staggered type 3D vegetation effect

Herein figure 4 and 5 shows the comparison of bed shear stresses along the d/s slope and bed in between grid type 3D and staggered type 3D vegetation placed both on d/s slope and bed. For grid type 3D roughness, flow accelerates within vegetation rows and increased the flow velocity results in increasing of bed shear stress. The effects of grid type 3D vegetation shows that bed shear stress increased on the d/s slope as well as bed due to the acceleration of flow in between the vegetation rows. Bed shear stress increased from 10-27% on d/s slope and 22-35% on bed for 5cm and 15cm spacing rows (0.25 and 0.75 ratios) respectively. On the other hand, in staggered type 3D roughness the flow could not accelerate directly between the vegetation rows because flow retard by the alternate rows in this type and decreased the flow velocity that results decreased the effective bed shear stress on both d/s slope as well as bed of embankment. The reduction of effective bed shear stress is found out 30% and 61% on the d/s slope and bed respectively for staggered type 3D roughness.

Following figure 6 and 7 illustrate the comparison of effective bed shear stresses for row type 2D vegetation with different spacing and all over vegetation on the d/s slope and bed. It is found that, in row case, maximum reduction of bed shear stress is attained for 5cm spacing rows (for 0.25 ratio) with 20% blocking, on the other hand, maximum reduction of bed shear stress is achieved due to 20% blocking for all over vegetation case.

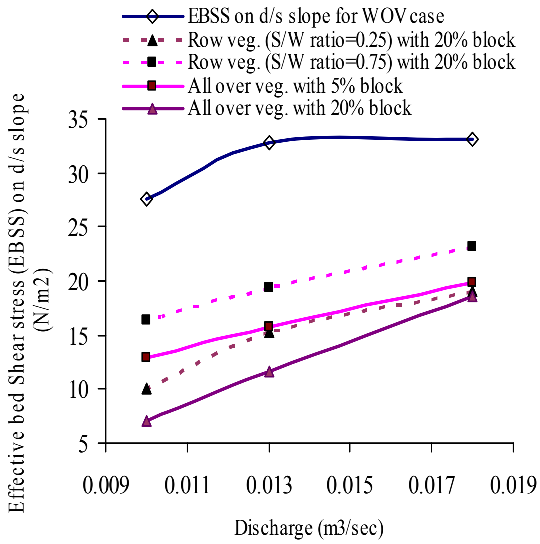

Figure 6. Comparison of effective bed shear stresses along the d/s slope in between different row type or 2D veg. and all over vegetation effect on both slope and d/s bed

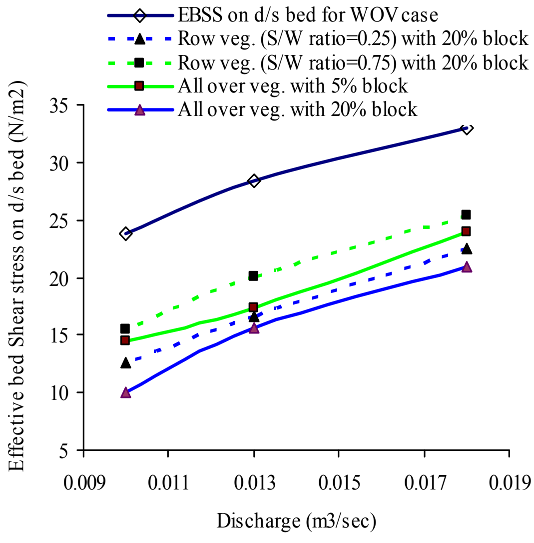

Figure 7. Comparison of effective bed shear stresses on the d/s bed in between different row type or 2D vegetation and all over vegetation effect on both slope and d/s bed

Comparative effects of row type 2D and all over vegetation on d/s slope show that, for all over vegetation the maximum reduction of EBSS is established as 61% on d/s slope and 48% on bed, both for 20% blocking case. Similar effects are found on the d/s bed. Vegetation reduced the bed shear stress about 47% for 20% blocking however, the bed shear stress reduction was 35% on the d/s bed due to low blocking case taken as 5% blocking in compared with 2D row type vegetation effect.

CONCLUSION

A cost effective and eco-friendly solution for stabilization of earthen embankment is presented in this paper. Vegetation markedly reduced the effective bed shear stress on the surface that demonstrated a significant reduction of damage and risk of failure from overtopping flows can be achieved by providing vegetation cover on the d/s slope of the embankment.

Based on the results of this investigation, the following conclusions can be drawn:

- Bed shear stresses behind the vegetation rows (BVR) are much lower in block type 3D roughness whereas, higher values occurred in 2D type row vegetation.

- BSS in between grid type 3D vegetation and staggered type 3D vegetation, it is seen that BSS is increased in 3D type vegetation due to the acceleration of flow within the rows whereas, BSS decreased on both d/s slope and bed of embankment as the flow retard directly by the alternative rows in staggered type.

- It is observed that the maximum reduction of effective bed shear stresses is obtained for 20% blocking all over vegetation on the d/s slope and bed of the embankment compared to all the cases.

- Spacing of roughness element is also the influencing factor in reducing bed shear stress.

- It is concluded that vegetation can be the effective and innovative solution for the stabilization of river bank or coastal embankments.

RECOMMENDATION

The present research is conducted considering the constant d/s slope of embankment with maximum of 20% blocking effect by vegetation density. Vegetative roughness affect the flow rate and results the reduction of bed shear stress, however high vegetation density increases the bed shear stress by trap water within them. Therefore, further study is needed considering more vegetation density effect to reach that level, for clearer conclusion.

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FEASIBILITY OF SUSTAINABLE URBAN DRAINAGE SYSTEMS (SUDS) IN REDUCING FLOODS IN DHAKA

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ABSTRACT:

The approach to flood management has changed over time worldwide. A more integrated, sustainable and natural flood management system is currently being adopted rather than focusing on engineering measures for flood defence. Dhaka, one of the largest megacities in the world, is extremely prone to detrimental flooding. The hazards associated with this are expected to increase in the coming years due to the impacts of climate change as well as rapid population growth and unplanned urbanization. SUDS measures for flood management in urban areas are becoming progressively more widespread in developed countries. However, the transfer of this technology to the developing countries like Bangladesh is not necessarily a straight forward process due to the constraints under different climatic, topographic and socioeconomic conditions. This study investigated whether Sustainable Urban Drainage Systems (SUDS) would be feasible for reducing flooding in Dhaka, either by decreasing the amount of runoff or by increasing the carrying capacity of the urban drainage system. Long term rainfall, water level and land use data for Dhaka were analysed to predict changes in runoff over time for the different types of land uses in Dhaka. The effects on storm water depth were calculated of introducing different SUDS measures judged to be relevant to the situation of Dhaka, namely – green roofs, rainwater harvesting, infiltration devices, pervious pavement, and swales. The calculations showed that the reduction in storm water depth was greatest (10 to 80% depending on the area coverage) when rain water harvesting was implemented and lowest (- 10 to 10% depending on weather condition) when pervious paving was implemented.

Keywords: Drainage System; Floods; Land use; Runoff; SUDS;

INTRODUCTION

Dhaka, the capital city of the South Asian developing country Bangladesh, is the world's eleventh largest megacity. The city is now facing an additional stress as one of the most vulnerable cities in the world to climate change which is predicted to bring erratic changes in temperature and rainfall patterns (IPCC, 2007). Of all the environmental problems and risks, including air pollution, surface water quality, depletion of ground water, improper solid waste and sewage management, traffic congestion and the expansion of slums and squatter settlements, flooding is the most widespread hazard the city is currently facing. Rapid unplanned urbanisation combined with excessive population growth and poor drainage systems, with little or no conservation of the natural drainage network, is aggravating the situation. Even though the Government of Bangladesh has taken a number of preventive measures to improve Dhaka's overall capacity to withstand floods, there are still additional opportunities to improve the situation. This study investigates whether an integrated and sustainable approach such as Sustainable Urban Drainage Systems (SUDS) would be feasible for reducing flooding in Dhaka.

AIM AND OBJECTIVES

The aim of this study is to assess whether SUDS can reduce flooding in Dhaka. To achieve this aim the research has the following objectives:

- To determine the frequency of occurrence of high rainfall/water level events in Dhaka.
- To estimate changes in runoff over time based on the appropriate values of runoff coefficients for the different types of land uses in Dhaka.
- To quantify how selected SUDS measures could reduce storm water depth in Dhaka, and hence reduce flood risk.

STUDY AREA

The study area is Greater Dhaka (Fig. 1), with an estimated population of 12.4 million (expected to be 16.8 million by 2015) (UN, 2006). Dhaka experiences a warm and humid summer, a short winter of moderate temperature and substantial rainfall during the monsoon from June to August. The city is located on low-lying land; of the city's total area of 276 km², 170 km² is <6 m above mean sea level (MSL) (JICA, 1987). The peripheral rivers of the city are shown in Fig. 1.

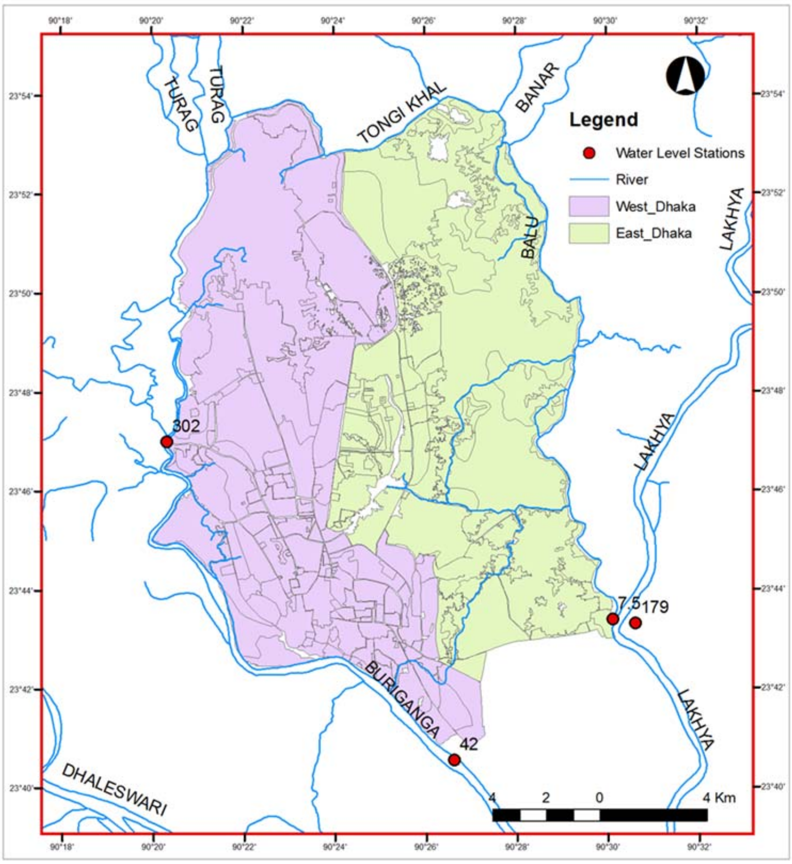

Figure 1. The study area (eastern and western Dhaka) and four water level stations used in the study (SW7#5, SW179, SW42, and SW302) located on the peripheral river system

A number of severe floods have struck Dhaka since its foundation, particularly in 1954, 1955, 1970, 1980, 1987, 1988, 1998 and 2004. The last three major floods in 1988, 1998 and 2004 inundated 47%, 53% and 43% of the area of Dhaka, respectively (Dewan *et al*., 2006) and caused enormous damage and economic losses.

SUDS FOR REDUCING FLOODS

The initial approach to flood management until the 1970s was focused on land drainage and flood defence, but during the 1980s and 1990s the approach shifted from flood control towards flood management. All these approaches had an emphasis on engineering measures rather than integrated sustainable measures (Ledoux *et al*., 2004). SUDS is one component of sustainable flood management, which is currently being adopted and promoted worldwide for its sustainability and cost effectiveness (Woods-Ballard *et al*., 2007). SUDS provide water quality, biodiversity and amenity benefits along with runoff detention and retention in urban areas (Ellis *et al*., 2002; Woods-Ballard *et al*., 2007). The systems focus mainly on urban areas and aim to replicate natural processes by allowing a developed catchment to function hydrologically as if it had remained undeveloped (Bastein *et al*., 2005; CIRIA, 2005). A feasibility study of implementing SUDS for stormwater control in another city in a developing country, Belo Horizonte, Brazil, was initiated in 2008 under SWITCH project (an EU-funded research programme for urban water management working in 15 European and developing cities worldwide). Like Dhaka, SUDS measures are not addressed in the city's current drainage system in spite of the fact that flooding and wet weather pollution are the major issues for the city. In addition the climatic condition of the city is overall similar to Dhaka with an annual rainfall of 1500 mm and average annual air temperature of 21°C (Silva *et al.,* 2010). SUDS types such as infiltration and detention devices, artificial wetlands and rain water harvesting systems were demonstrated in the research. Even though, the final result has not been reported, the preliminary results suggest that incorporating more integrated, diffuse and source control water management approach, such as SUDS, reduces stormwater runoff and enhances the opportunities for river restoration (Silva *et al.,* 2010).

In past guidance for SUDS the focus of water quantity management was to reduce the peak rate of runoff, but recently there has been an equal emphasis on reducing the total volume of runoff (Woods-Ballard *et al*., 2007). There are a number of SUDS devices, such as infiltration devices, green roofs, water harvesting, basin, trenches, soakaways, swales, pervious paving, which are considered to have a high to moderate potential for restricting runoff rate as well as minimizing runoff volume.

Due to the limited studies of SUDS in developing countries in this study the feasibility of SUDS in reducing floods in Dhaka was assessed using data for SUDS performance reported in the literature from other locations. Nevertheless, it is recognised that the efficiency of the performance of SUDS measures is generally site-specific and restricted to environmental conditions.

METHODOLOGY

The procedures applied to achieve the objectives of the study consisted of rainfall frequency analysis and water level frequency analysis and storm water depth analysis from the daily time series of rainfall and river water level data spanning years from 1957 to 2008 for four water level stations on the eastern and western peripheral river systems of Greater Dhaka. This was followed by analysing how selected SUDS measures can reduce storm water generation. Greater Dhaka was divided into western and eastern zones (shown in Fig. 1) based on the existing drainage system. Land use in Dhaka east was related to water level stations SW7#5 and SW179, whereas land use map in Dhaka west was used for stations SW42 and SW302. The overall methodology is shown in Fig. 2.

The missing maximum daily rainfall data for some years were estimated from the median value of the rest of the dataset (Dawson *et al*., 2006). The missing maximum daily water level for different stations were estimated from linear regression equations fitted between the highest water level value each year and the highest rainfall value of the respective year for each station. The frequency of occurrence of maximum annual daily rainfall and water level were analysed using the Gumbel distribution, the most widely utilized distribution by hydrologists (Yue *et al*., 2000). The recurrence interval for the danger water level for each station, at the water level above which flooding occurs, was estimated using the water level frequency graphs projected from the water level time series data. Next the rainfall depth corresponding to this recurrence interval was determined using the rainfall frequency graph. Then, storm water volume and depth generated by this amount of rainfall on the land uses surrounding each station were calculated, taking account of the runoff coefficients of different land use coverage obtained from the Urban Drainage Manual, Local Government Engineering Department, Bangladesh.

Finally, in combination with the land use map for Dhaka in 2000, the storm water depth reduction was calculated of five different SUDS measures (green roofs, rain water harvesting systems, infiltration devices, pervious pavements and swales). These SUDS measures were chosen for analysis as they have the potential to control both runoff rate and runoff volume (Woods-Ballard *et al*., 2007).

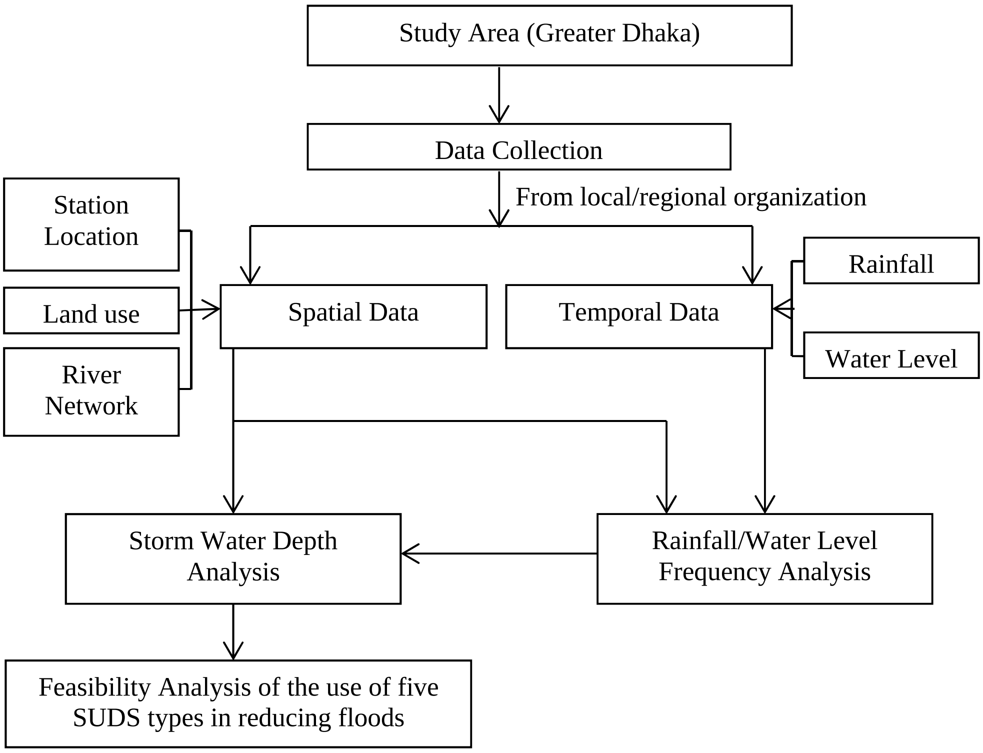

Figure 2. Overall methodology of the feasibility study of SUDS in reducing flooding in Greater Dhaka

RESULTS: ANALYSIS OF THE FEASIBILITY OF SUDS IN REDUCING STORM WATER GENERATION IN DHAKA

The results of the analysis described above for each water level station are shown in Table 2. The storm water depth calculated represents the storm water depth under current flood management conditions (i.e. no SUDS present).

Table 2. Estimation of the storm water depth at which flooding occurs at each of the water level stations under current conditions (no SUDS present)

Station Identifier	SW7#5	SW179	SW42	SW302
Location	Balu	Lakhya	Buriganga	Turag
Danger river water level (m)	5.75	5.5	6.0	5.94
Recurrence interval of danger level (years)	1.3	1.4	4.8	1.5
Daily rainfall depth corresponding to this recurrence interval(mm)	102.5	109	172.5	115
Total storm water volume generated by this daily rainfall(m^3/day)	$7.5 * 10^6$	$7.97*10^6$	$13.15*10^6$	$8.77*10^6$
Storm water depth over study East/West Dhaka corresponding to this daily rainfall (mm)	56	59	97	64

Finally, each of the five SUDS options was evaluated by recalculating the storm water depth if that SUDS option had been implemented and thereby assessing how much the storm water depth had been reduced using each option. The results of the evaluation are presented in Table 3.

Table 3. Reduction in storm water depth (expressed as a %) of implementing different SUDS options

SUDS Option	Area Coverage		Reduction of Storm Water (%)				Mean reduction(%)
			Dhaka East		Dhaka West		
			SW7#5	SW179	SW42	SW302	
Green Roofs	20%		5	5	6	6	6
Rain water Harvesting	10%		10	10	10	10	10
	20%		20	20	20	20	20
	30%		30	30	30	30	30
	40%		40	40	40	40	40
	50%		50	50	50	50	50
	60%		60	60	60	60	60
	70%		70	70	70	70	70
	80%		80	80	80	80	80
Infiltration Devices	10%		5	5	8	5	6
	20%		9	10	16	11	11
	30%		14	15	24	16	17
	40%		18	19	32	21	23
	50%		23	24	40	26	28
Pervious Pavement	Extended Dhaka(140.88 km²)	Runoff Coeffn = .7	-10	-10	-10	-10	-10
		Runoff Coeffn = .575	10	10	10	10	10
Swales	Extended Dhaka(140.88 km²)	Runoff volm reduction=21%	4	4	4	4	4
		Runoff volm reduction=52.5%	11	11	11	11	11

The results showed that implementation of rainwater harvesting systems gave the maximum reduction in storm water depth, with the reduction in storm water depth mirroring closely the % implementation of rainwater harvesting, i.e. reduction varied between 10 and 80% on average as shown in Table 3.In contrast, implementation of pervious pavement did not result in substantial reduction in storm water depth. This measure was selected for implementation in areas of new development in Dhaka rather than retrofitting the existing city area. The runoff coefficient for pervious pavements in wet conditions was used for the analysis as it represents the likely conditions when flooding occurs in Dhaka during the monsoon season. The result obtained from the analysis was negative for all four stations because this runoff coefficient for pervious pavements is higher than the runoff coefficient for the land use without SUDS present. However, if the average runoff coefficient of dry and wet conditions (0.575) (Anderson *et al*. 1999) was used to estimate the effectiveness of pervious paving, the reduction in storm water depth was estimated to be 10%.

CONCLUSION

Currently, the Government of Bangladesh has taken some non-structural flood management measures in Dhaka by emphasising adaption to floods, rather than mitigation, and has established a Flood Forecasting and Warning System (FFWS), flood shelters, emergency services and taken other post flood rehabilitation measures. However, questions can be raised about whether the city is prepared to face the impacts of ongoing climate change with more frequent and intense rainfall and flood events. SUDS approaches in combination with the existing drainage framework have been shown to reduce storm water depth (and hence flood risk) in Dhaka, and also bring other benefits such as cost effectiveness and other amenity value. Based on this feasibility study it is recommended that SUDS measures like green roofs, rain water harvesting, infiltration devices or swales can be provided to some extent in Greater Dhaka for controlling storm water, in particular the 200 km^2 area identified for the future expansion of Dhaka. However, the literature reviews of SUDS have shown that the performance of different SUDS types for flood control is affected by site specific conditions. Since no research has been conducted so far on SUDS implementation considering the climatic, topographic and socioeconomic condition of Dhaka, it is recommended that the field studies of SUDS performance are conducted in Dhaka and the results used to improve assessment of the feasibility of individual SUDS types for controlling floods in Dhaka.

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