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Organized by
Department of Civil Engineering
Chittagong University of Engineering & Technology
Chittagong-4349, Bangladesh

Table of Content

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COMPATIBILITY STUDY OF FAECAL SLUDGE DRYING BED WITH REED BED SYSTEM IN SECONDARY TOWNS OF BANGLADESH

S. K. Biswas[1]*, S. S. Hossain[1], M. S. Hassan[1] & S. Debnath[1]

[1] *Department of Public Health Engineering, Dhaka, 1000, Bangladesh.*
**Corresponding Author, <shishir_40@yahoo.com>*

ABSTRACT

Safe disposal of faecal sludge is one of the main components of improved sanitation in any country. Bangladesh though a densely populated country have very limited solutions of faecal sludge treatment. Considering the technical, environmental, social and financial aspects a treatment process of septic tank waste water and sludge has been implemented in a secondary town named Lakshmipur Pourashava. Faecal sludge drying bed with reed bed system has been introduced here. Tractor driven Vacutugs are given for sludge collection and sludge transport facilities. The treated waste water after sludge disposal, meets the waste water quality standard as mentioned in DoE, 1997. The drying bed is so designed that the waste water and septage sludge can be disposed in a bed continuously 5-7 years with septic tank empting interval 2-3 days per week. To accelerate sludge dewatering, locally available reeds of phragmites family have been planted. Reeds solved the probable malodour and aesthetic problems of the plant. The drying beds becomes profitable to the Pourashava and acceptable to the local people.

Keywords: Faecal sludge, treatment, reed bed, sanitation, quality.

INTRODUCTION

Faecal/septage sludge management concept is almost new in Bangladesh. In urban areas of Asia, Africa and Latin America, the excreta disposal situation has become dramatic: thousands of tons of sludges from on-site sanitation installations – so called faecal sludges (FS) – are disposed off daily untreated and indiscriminately into lanes, drainage ditches, onto open urban spaces, into inland waters, estuaries, and the sea (Steiner et al., 2002). Though Dhaka is the capital city it has only a sewage treatment plant at Pagla with a 120 ML/d capacity and currently only 30% of the city area is served by the sewer system of which only 20% of the city population is connected with this (DWASA, 2011). The facilities of sludge treatment plant in secondary towns (Pourashavas) are beyond imagination.

The Department of Public Health Engineering (DPHE) is the national lead agency provides drinking water supply, sanitation, drainage and waste management facilities in urban and rural areas excepting cities where WASAs operate. It has implemented the low cost technology of faecal sludge treatment including well equipped sludge transport system in some Pourashavas in Bangladesh. The aim of this initiative was the improvement of public health; Elimination of dumping of septic tank sludge into the environment; improving the functioning drainage system; Enhancing solids retention in septic tank to reduce solids accumulation in the drainage system and prevention of environmental pollution that is caused by effluents of not regularly de-sludged septic tanks. By product of the treatment, the dried sludge could be used as organic manure in agriculture and thus Pourashava’s extra income source.

EXISTING FAECAL SLUDGE DISPOSAL PRACTICE

On-site sanitation practice is prevailing in the Pourashavas in Bangladesh. Domestic wastewater generally pre-treated in septic tanks and tank effluent is collected in soakage pits. Emptying of Sludge from Septic Tanks is done manually by employing private labourers by individual household themselves and disposed of at places of convenience i.e. in drains, in dug holes, or open land without any treatment. Presently Pourashavas have very limited arrangements of centralized sludge collection and treatment for environmentally safe disposal of sludge collected from individual household's septic tanks. Sanitation, waste and sludge management are still neglected issues in Bangladesh. Peoples are not aware of improved sanitation system. They do not feel the essentiality of safe disposal of faecal sludge.

DESIGN CONSIDERATIONS FOR IMPLEMENTED FAECAL SLUDGE TREATMENT

Design of sludge treatment plant mainly depends on the characteristics of sludge ingredients, contents and degree of contaminants. The characteristics of sludge to be measured are strongly related to its ultimate fate (Malack Muhammad et al., 2007). Based on the results of the characterization, alternative treatment processes should be evaluated and finally, proper reuse or disposal schemes be recommended. Sludge drying beds are physical treatment processes that can be considered as an effective way of sludge dewatering, however, the performance of these processes depend entirely on the physical condition of the bed, type of sludge, temperature, detention time (drying period) and meteorological conditions (Malack Muhammad et al., 2007). Dharmappa et al.(1997) reported that sand drying beds are popular due to their reliability, ease of use and low cost. However, one of the basic concerns with sand drying beds is the requirement of large area of land. Hossam et al. (1990) assessed the drain ability of sludge generated by different treatment processes. They reported that the climatic conditions were generally favorable for dewatering sludge on sand drying beds except when there was heavy rainfall during wet seasons which resulted in prolonging the drying period.

After the issue of sludge drying process, killing of pathogens are the next important criteria of design. The numbers of *faecal coliforms* and *salmonella* decreased as temperature and rate of desiccation increased (Zaleski et al., 2005). Gautam et al. (2005) implemented irradiation of municipal sludge for safe disposal and agricultural use. They reported that the process parameters were adjusted to effectively eliminate coliform bacteria in the sludge and to prevent their re-growth. Irradiated sludge was found to be free of *faecal coliform* and could be directly disposed after drying in a landfill or used as manure.

During survey and design stage it had been found that Sludge of Septic Tanks had a total solid content of 0.5 to 3 % and contains high load of Biological oxygen demand (BOD), chemical oxygen demand (COD), ammonia nitrogen (NH_3-N), coliform and Helm. Eggs. Drying the sludge with maximum reduction of the mentioned parameters to the limits of Bangladesh standard (DoE, 1997) was the main consideration of the design so that the plant will not be the source of further pollution and hazards.

STUDY AREA

Lakshmipur Pourashava (figure 1) is one of the Pourashavas selected for implementation of the faecal sludge treatment plant concept. The core area of this Pourashava is 3.5 Sq. Km, fringe area 16 sq. km and population is around 122572. The causes of selection of this Pourashava were i. Public demand and awareness, ii. Support of the Pourashava providing land for plant construction iii. Availability of manpower for operation and maintenance. The core area of the Pourashava are normally developed, having concrete building and sanitary latrine with septic tank. This is why option of vacutug for conveyance of faecal sludge had been introduced here.

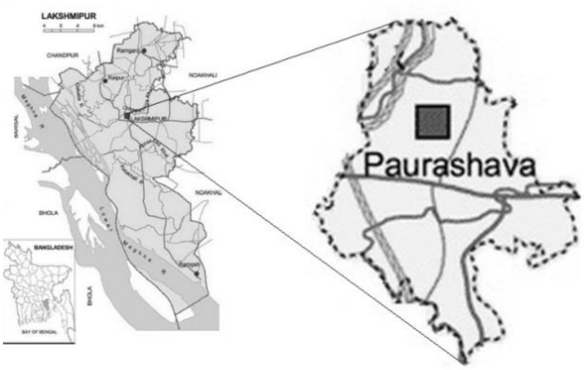

Figure 1: Study area Lakshmipur Pourashava

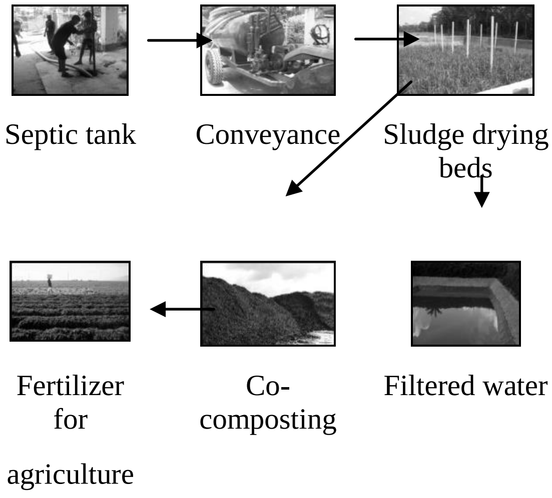

Figure 2: Process flow diagram

IMPLEMENTATION OF FAECAL SLUDGE TREATMENT PLANT

Implementation of faecal sludge treatment plants are in realization of the fact that, faecal sludge management must be an integral part of every sanitation plan, which builds on on-site sanitation facilities. Sludge management is an indispensable part of the maintenance of these facilities. In the absence of proper sludge management there will be serious negative impacts on the urban environment and on public health.

Safe disposal of faecal sludge had two major components i. Careful transportation and handling of sludge and ii. Protected disposal and effective treatment system. Tractor driven Vacutugs were given to the implementing Pourashava as the provision of equipment for Septic Tanks sludge collection and Sludge transport facility. In Lakshmipur Pourashava three vacutugs with different capacity (i.e. $2m^3$ and $0.70m^3$) had been provided with a view to access both the narrow and wide roads.

The basic concept of the implemented technology was to collect septic tank water and sludge through vacutug and be conveyed to the sludge drying bed. Treated water would be discharged in agricultural land or sewer or water bodies after ensuring the waste water quality standard as mentioned in ECR, 1997. Co-composting of the solid digested portion would produce fertilizer for agricultural use. The process flow diagram of the technology is given in figure 2.

There are different types of sludge drying bed practiced at different countries. Sludge drying beds are one of the simplest and oldest techniques for sludge dewatering can either be unplanted or planted. We have used the conventional sludge drying beds with simple impermeable beds filled with different layers of gravel and sand including planted vegetation for evapotranspiration which would faster the drying phenomenon. Planted drying beds do not need desludging before each new application / loading of sludge as root system of the plants maintains the permeability of the beds. The constructed treatment plant at Lakshmipur Pourashava required around 780 m^2 land area which consisted of two beds for alternative use. Each bed consisted of 144 m^2 area and had been designed to run around 5-7 years i.e. waste water and septage sludge can be disposed in a bed continuously 5-7 years with septic tank empting interval 2-3 days per week.

CHALLENGES OF IMPLEMENTATION

Faecal sludge treatment plant concept is almost new in Bangladesh. People are not aware of that the noxious impact of unplanned disposal and unhygienic handling of faecal sludge. Thus implementation of this treatment plant faced various difficulties. During implementation stage the following obstacles had been dealt with.

Pourashavas were not interested of this types of initiative;

Unavailability of required land. Lands of the Pourashavas are expensive.

General thoughts of the peoples are that sludge treatment plant spreads malodor which would hamper their life;

Monsoon rain and height of water during flood;

Aesthetic issues and

Selection of reeds.

The execution of works became sluggish at first. To overcome the problem details of the works and its benefits had been apprised to the Pourashava authorities and taken their help. Construction site had been selected outside of the core areas of the Pourashava and far from the dense habitation. Considering the flood issue the main structure and arrangement had been constructed so that the effluent would not mix with the flood water as well as structure would not be overflowed in future. To protect the area from malodor adequate ventilation pipe had been provided. To accelerate sludge dewatering, locally available reeds of *phragmites* family had been planted which has given double benefit. High growth of reeds covered the sludge drying bed and neutralizes the bad smell of faecal sludge. Evapotranspiration by planted reeds of *phragmites* family showing effective results and association of sun light in a bright day and percolation drying the disposed septic tank waste water and sludge within three to four hours. The reeds also solved the probable aesthetic problem and become acceptable to the local people.

OPERATION AND MAINTENANCE

After implementation of the faecal sludge treatment plant at lakshmipur Pourashava, desludging of septic tank waste water and sludge started from March 2013 and it is now in full operation. Though three vacutugs with capacity of $2m^3$ and $0.70m^3$ had been provided for collection of sludge, Pourashava authority is using the larger one. Desludging frequency is 2 septic tanks in 2 separate days of a week with 2-3 days rest. Pourashava’s three employees are working for operation & revenue collection. Treated waste water is collected in the effluent collection pond. After preserving effluent in the pond around 15 days to 1 month it is discharged to nearest channel or agricultural land ensuring the water quality.

Training had been given to the vacutug operators and sweepers before starting the plant in operation. The vacutug is automated and easy to control. At present there are no difficulties in sludge collection and disposal flow path. Pourashava has taken responsibilities to campaign the effectiveness of the plant among the citizens, collecting service charge from the house owner or building authority depending on their needs and providing service to them. It is required to mention that the Pourashava authority banned illegal disposal of septic tank sludge and waste water which also increasing the demand.

FINANCIAL AND ENVIRONMENTAL BENEFITS

Lakshmipur faecal sludge treatment plant came in operating from March 2013. For providing service to the households and managing the treatment plant operational costs, the Poura- authority imposed charge 1000 BDT ($12.82) Per Trip of vacutug of 2m3 capacity. Cost for operation is around 2000.00 BDT ($25.64) per week (Approximate). On average weekly service charge collection of Lakshmipur

Pourashava is around 5666.00 BDT ($72.64) per week. Thus the net income is 3666.00 BDT ($47.0) per week (Approximate). Construction cost of the plant was around 3000000.00 BDT ($38461.53) excluding the land value. Pourashava was helped providing around 780m2 of required land for construction of the structure. Based on the initial tariff collection the investment recovery will take around 16 years. Commercialize of bio-fertilizer after co-composting of dried sludge and reeds will reduce the investment recovery period.

The considerable positive impact found in environmental point of view due to implementation of faecal sludge treatment plant. Before operation of this plant it was quite common to see septage waste in open drain. House owner used to connect their septic tank discharge line to the open drainage system which convey the waste to the water bodies and pollute those tremendously. Since there was no safe dumping spot in the Pourashava, people used to remove septic tank waste to the abandoned places or pit in the night. This experience was not only harmful for the human health. It increased the possibility of ground and surface water contamination also polluted the air spreading stench. Now the peoples of the Pourashava are enjoying the fresh environment.

CONCLUSIONS

Faecal sludge treatment concept is still in researching stage and its exercise is not popular in Bangladesh. Public impression is also not clear thus have negative attitude towards it. Moreover, design and implementation of the low cost faecal sludge treatment plant was a big challenge. Since the construction budget was limited, planted sludge drying bed concept was chosen and designed considering the locally available effective reed species, sun light, rain and flood. Implemented structure has been well accepted by the Pourashava in all respect. After starting the plant operation there is still no considerable problems. Proper operation and maintenance of this plant can initiate a new era of improved sanitation in Bangladesh.

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DISCHARGING OF TOXIC CHROMIUM AFTER WET-BLUE PRODUCTION FROM TANNERY AT HAZARIBAGH, BANGLADESH CAUSING SERIOUS POLLUTION

M. A. Hashem[1*], A. Islam[1], S. Mohsin[1] & B. K. Mondal[2]

[1] Department of Leather Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.
[2] Apex Tannery Limited, 127 Hazaribagh, Dhaka-1209, Bangladesh.
**Corresponding Author, <mahashem96@yahoo.com>*

ABSTRACT

The article has focused on discharge of chromium from leather industries after wet-blue production. Results indicate that high concentration of chromium ranges 2656–5420 mg/L is discharged as a liquid waste after wet-blue production. Physical parameter of spent chrome liquor such as total solids (TS) is extremely high as well as chemical parameters dissolved oxygen (DO) and pH are low to survive for aquatic livings. A fraction of discharged spent chrome liquor is directly mixed with the river, Buringaga which is caused serious environmental pollution; other fraction of chromium is settled in the lagoon and suspected to be eluted chromium to groundwater in the tannery area that could be a great threat in the near future. Yearly Bangladesh could save a lot of money by practicing recoveries and reuse system of chromium that might be a good approach to make environment friendly leather production.

Keywords: Chromium, Dissolved Oxygen, Total Solids (TS), pH

INTRODUCTION

Leather process involves conversion of putrescible hide or skin into leather by the process of tanning. Since soaking to finishing operations a lot of tanning agents are used. The tanning agents take part to make the skin matrix permanent stabilization against biodegradation. After tanning operations, a good fraction of tanning agents is discarded as waste like solid, liquid or gaseous form which causes serious environmental pollution in soil, water and air.

To date, in unhairing and liming step a large amount of sodium sulphide and lime as well as other chemical products are used. In the conventional liming process leads to dissolving hair and epidermis, causing discharges with high biological oxygen demand (BOD), chemical oxygen demand (COD), high pH, and total solids (TS) loads in the effluents from leather industries (Jain et al., 2011). Although some of the method has been developed like enzymatic hide processing for replacement of sodium sulphide (Dettmer et al., 2013). Besides, after liming, lime requires removing from pelt mostly using by ammonium salts like ammonium chloride or ammonium sulphate which represent one of the liquid effluents containing high amount of nitrogen (Gutterres et al., 2011). In the conventional deliming yearly 3.5×10^6–12.8×10^6 m^3 ammonia is produced at Hazaribagh, Bangladesh; depends on solubility of ammonia at pH (8.5–9.0) a good fraction of ammonia is directly merged to atmosphere (Hashem et al., 2014).

There are 243 leather industries located at Hazaribagh in the western part of capital city Dhaka, Bangladesh (Salam et al., 1998; Ahmed, 2012). Per day hides and skins in Bangladesh are estimated to be processed for leather production 240MT in which generating huge amount of solid and liquid wastes (Ahmed, 1997). Solid and liquid effluents aew comprised of rotten flesh, soluble proteins, fat, toxic chemicals, dissolved lime, suspended and dissolved solids, organic matters, dyestuffs and coloring pigments, heavy metals like chromium etc. (Biswas et al., 2012).

Since the last decades of industrialization, Bangladesh is facing the environmental degradation of the river, Buriganga and other linked rivers due to receiving the discharged green solids and liquids from the leather industries (Zahid et al., 2006). Unfortunately, excluding for one modern leather industry (Apex Tannery, Unit-2) none of the leather industries has an effluent treatment plant (ETP).

Chrome tanning is the subsequent operation of pickling and most popular step in leather processing where 90% tanning industries are used basic chromium sulpahte (BCS) (Avindhan et al., 2004); it binds with collagen protein to make stabilization against biodegradation. Chromium has a tendency to precipitate at pH more than 4.0 (Thanikaivelan et al., 2000) as a result limiting the possibility of higher penetration of chromium at chrome tanning pH levels (2.0–4.0) (Sharphouse, 1972). In the conventional chrome tanning results in wastewater chromium containing 1500-3000 mg/L (Suresh et al., 2001).

Chromium recovery from tannery wastewater is an essential for economic benefit and environmental friendly; many countries are practicing recoveries and reuse system. Recoveries of chromium are practiced i) directly recycling approach which involves filtration of waste liquor followed by chemical replacement (Sharp, 1981) and ii) precipitation of chromium in waste liquor as chromic trihydroxide then filtration and dissolution in sulfuric acid to form BCS solution for reuse (Boast, 1988). In case of industrial wastewaters are chemical reduction by precipitation, ion exchange (Tiravanti et al., 1997), membrane technologies (Ashraf et al., 1997) and adsorption by adsorbents (Dahbi et al., 2002).

In Bangladesh, yearly 10,000 tons of basic chromium sulpahte (BCS) is used in leather process (Karim, 2000); after wet-blue production spent chrome liquor is directly discharged through drain to fall low lying area and finally fall to the river, Buriganga. It is the conventional practice to discharge spent chrome liquor and spent lime liquor simultaneously at the same stream causes to precipitate as chromic hydroxide or adsorb with sediment/soil. Chromium exists in several oxidation states, with trivalent Cr^{3+} and hexavalent Cr^{6+} species being the most common forms (Kotaś et al., 2000). The occupational exposure of chromium has been widespread and it is shown that chromium (III) under certain ligand in environments leads to cell death and structural modification of proteins (Balamurugan et al., 2002).

A good number of researchers have accounted on tannery wastes in Bangladesh by characterization of the parameters and their impact assessment on the environment (Das, 2000; Ahmed, 1997; Rouf et al., 2013). In most cases samples were collected after mixing of discharged from tanneries before fall to the river, Buriganga from where it is difficult to identify specific effluent situation in specific operations. In this study spent chrome liquor were exclusively examined after wet-blue production i) chromium concentration in discharged spent liquor ii) pH of the spent liquor iii) dissolved oxygen (DO) and iv) total solids (TS) to comply with environmental industrial regulations in Bangladesh.

MATERIALS AND METHODS

Sample Collection

The area is located at Hazaribagh, Dhaka where 90% tanneries of Bangladesh are producing leather all the year round. Large and medium categories nine (9) tanneries were selected for sampling of waste chrome liquor just after wet-blue production. Spent chrome liquors were collected into polyethylene bottles and were brought back to laboratory for experiment and samples were kept in the refrigerator at 4ºC until to complete the experiment. In Fig. 1 shows the operational sequences of chemical treatment for wet-blue production.

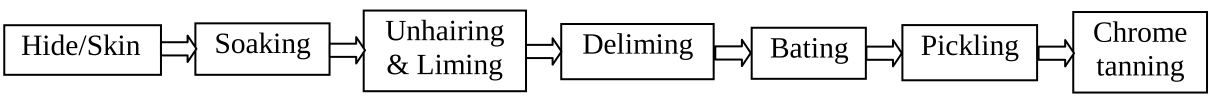

Fig. 1 Flow chart for conventional wet-blue production

METHODOLOGY

Determination of Chromium

After filtration the spent chrome liquor, total chromium was measured by the atomic absorption spectroscopy (Varian AA 240) at Dhaka University, Bangladesh.

Determination of DO and pH

Dissolved oxygen and pH of the samples were measured on that day by using the DO (HQ40d, HACH, USA) and pH (UPH-314, UNILAB, USA) meter, respectively. Before measuring DO and pH meters were calibrated by the standard solutions.

Determination of total solids (TS)

Total solids (TS) were determined gravimetrically as per standard methods of APHA (APHA, 1999). A 10 mL liquid waste was passed through the glass fibre filter and dried at drying oven at 103 to 105°C until a constant weight is obtained.

RESULTS AND DISCUSSIONS

Total chromium

The concentrations of total chromium (Cr) in the spent chrome liquors are illustrated in the Fig. 2. The level of chromium concentrations of the samples are 2656–5420 mg/L which are extremely high whereas the industrial standard limit for chromium in the waste water is 2 mg/L (ECR, 1997). It seems that concentration of Cr in the spent chrome liquor is 1328–2710 times higher than the standard level.

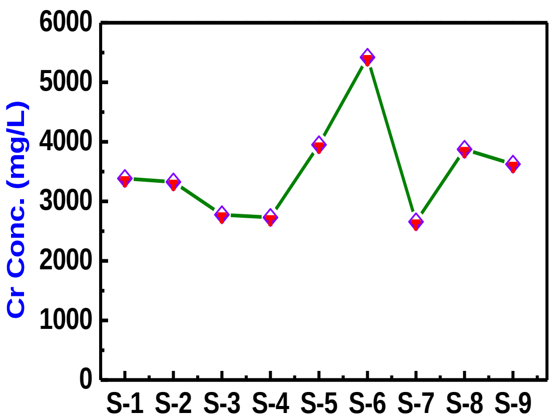

Fig. 2 Chromium concentration level at different samples

The high amount of chromium is discharged from the leather industries at Hazarigbagh, Dhaka which is discharged directly through drain without any applying alternative way like recover or reuse system. During carrying through drain; it mixes with simultaneously discharged lime liquor (pH 12.0–13.5) resulting produces hydrogen sulphide (H_2S) gas which causes environmental nuisance. Of course, a fraction of chromium is precipitated as chromic hydroxide or absorbed by soil/sediment or carrying as liquid phase and fall to the low lying adjacent areas. Onset of favorable condition chromium is eluted from sediment/soil that could be mobilize to aqueous phase or liquid phase of chromium is finally mixed with the river, Buriganga which means the linking rivers are becoming contaminated with chromium. Yearly 10,000 tons basic chromium sulphate is used in Bangladesh in the leather industry (Karim, 2000). In general pickle pelt is uptake 60% of the chromium in the form of basic chromium sulphate. Rest of the 40% chromium is discharged as wastewater to the environment.

pH

In Fig. 3 shows the pH values of spent chrome liquor from different the tanneries and pH varying 2.4–3.0. In the conventional leather process, after chrome tanning the pH is raised ~4 to fix chrome with the collagen (Covington, 2011). Usually before chrome tanning, in pickling stage float pH is maintained at 2.5–3.0 for penetration of chrome into pelts (Covington, 2011). In that context pH is very low and fixation of chromium is limiting thus discharge as waste.

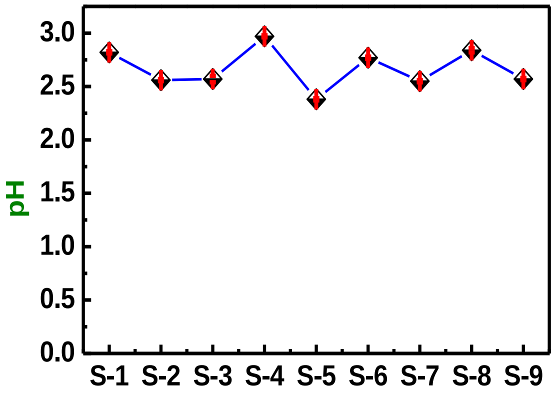

Fig. 3 pH levels at the different samples

After completing chrome tanning, spent chrome liquor is discharged without applying recoveries or reuse system. High acid concentration of the spent liquor is decreasing the pH level adjacent low lying area's aquatic life as well as the final reservoir of the river, Buriganga. It seems that all samples pH levels are beyond the standard level for wastewater discharge from the industrial unit (pH 6–9) (ECR, 1997). Lower the pH of the spent chrome liquor is produced H_2S when it is mixed with the spent lime liquor. It is the conventional practice to discharge spent lime and chrome tanning liquor simultaneously at the same stream continuously H_2S gas is produced.

Dissolved Oxygen (DO)

In Fig. 4 represents the dissolved oxygen (DO) levels of the measured samples. The DO values are laid between the 2.6 mg/L–8.3 mg/L. Even though, entire samples are from the same operations but different tanneries. The dissolved oxygen level in surface water is 8.5 mg/L at 24ºC (EPA, 2001) and standard for waste from industrial unit is 4.5–8 mg/L (ECR, 1997). The DO values for S-3 and S-7 are closed to lower boarder limit and only S-1 and S-2 DO levels are tend to upper boarder limits. The rest of the samples DO values are below the standard level.

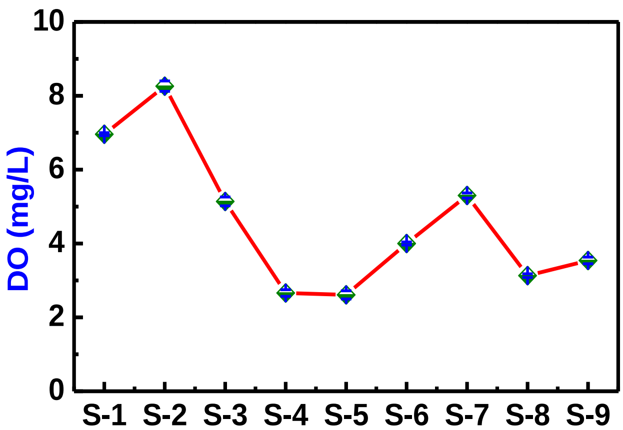

Fig. 4 Dissolved oxygen (DO) of the tested samples

The lower DO values of spent chrome liquors are directly discharged to drain to fall low-lying area and finally it is mixed with the river water of Buriganga. Liquid waste from various operations as well as from different tanneries falls to river, Buriganga which is one of the most reasons to decrease DO level. As a result, numbers of fishes as well as aquatic plants are disappearing due to lack of dissolved oxygen.

TOTAL SOLIDS (TS)

Total solids (TS) are inserted in the **Table 1**. It seems that total suspended solids are extremely high and beyond the permissible level. Total suspended solids (TSS) standard level is 150 mg/L but the experimental values are 36–95 times higher than the standard levels. The total dissolved solids (TDS) standard level is 2100 mg/L but the tested results are 22–53 times higher (ECR, 1997).

Table 1. Total suspended solids (TSS), total dissolved solids (TDS) and total solids (TS)

ID	TSS (mg/L)	TDS (mg/L)	TS (mg/L)	RSD
S-1	8630.0 ± 0.73	67377.0 ± 7.05	76007.0 ± 6.99	0.92
S-2	14297.0 ± 3.84	92076.7 ± 0.98	106373.7 ± 2.93	0.28
S-3	5401.1 ± 1.04	62993.3 ± 13.23	68394.5 ± 14.22	2.08
S-4	6800.0 ± 0.85	68103.3 ± 13.13	74903.3 ± 12.49	1.67
S-5	11986.7 ± 2.25	87253.3 ± 7.21	99240.0 ± 8.85	0.89
S-6	7480.9 ± 1.23	45293.3 ± 4.92	52774.2 ± 5.53	1.05
S-7	8259.2 ± 0.92	52273.3 ± 7.66	60532.5 ± 8.43	1.39
S-8	9261.6 ± 1.05	67176.7 ± 14.40	76438.2 ± 14.36	1.88
S-9	6283.3 ± 1.34	110553.0 ± 16.25	116836.7 ± 27.28	2.33

The total solids have negative effect on the aquatic life. The discharged suspended matters get deposited on the bed of the stream and kills aquatic organisms of the stream bottom. The floating solid interfere the streams ability for self-purification by re-generation of oxygen absorbing from the atmosphere and also interferes the photosynthesis activity of the stream plankton and aquatic plants. Suspended solids caused the turbidity which decreases infiltration of sunlight into water thus reduces photosynthesis activity of aquatic plants.

CONCLUSION

High concentration of chromium is discharged from the tanneries after wet-blue production as waste in the low lying area as well as to the river, Buriganga. The adsorbed chromium by the soil/sediment could be eluted from soil/sediment to groundwater in the residential area that could be a great threat for human health in the near future. The total solids (TS), DO and pH have negative effect on aquatic life. Due to lack of dissolved oxygen aquatic living organisms including fishes are becoming disappeared from the water bodies including the river, Buringanga. All concern authorities especially tannery owners should practice recoveries and reuse system of chromium so that country could save huge amount of money as environment will be friendly.

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ENVIRONMENTAL MANAGEMENT PLAN ON PROPOSED KALISHURI TO SURJOMONI UNION ROAD CONSTRUCTION PROJECT AT BAUPHAL, PATUAKHALI, BANGLADESH

Md. Hamidul Islam[1]*, Md. Shafiqul Islam[2], Sahadat Hossain[3] & Tania Yeasmin[4]

[1] *Department of Building Engineering & Construction Management, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.*
[2,4] *Department of Geography and Environment, Dhaka University, Dhaka-1000, Bangladesh.*
[3] *Institute of Water Modelling, Dhaka, Bangladesh.*
**Corresponding Author, <hamidcekuet@gmail.com>*

ABSTRACT

This article represents the assessment of environmental impacts, estimation of environmental impact value and Environmental Management Plan (EMP) of proposed six kilometre road construction project from Kalishuri to Surjomoni union at Baupahl in Patuakhali district. The entire study was conducted straightly in the field visit, Focus Group Discussion (FGD), Bangladesh Bureau of Statistics (BBS) and also getting information from union and upazila Parishad at Baupahl. Identifying and analyzing the potential environmental impacts, impact value and preparing environmental management plan are the main objectives of this study. The key findings of this study include environmental impacts on vegetation, hydrology, settlement, agricultural land and soil, water and air pollution of the proposed area. The study reveals that the environmental impact value is -1(negative one) regarding three environmental factors named ecological, physic-chemical and human interest. The EMP designed to various components such as the construction EMP, noise and blasting management plan, dust reduction and prevention plan, hydrology and drainage management plan and vegetation management plan for this project. A proper environmental management plan can ensure the environmental sustainability of a project. The mitigating options of various impacts on the project are environmental friendly technology, conservation of road side plantation, agricultural lands and regional hydrology. Also, the mitigation measures would be sustainable and long term durable of the proposed road construction project. The EMP helps to complete an environment friendly project by providing sound management on environmental impacts and means of preventing or reducing those impacts.

Keywords: Environment, Impact, Mitigation, Management and Road construction.

INTRODUCTION

Environmental impact assessment is a process which identifies the possible impacts of a project, policies, plans or programmes on the environment prior to the project actually taken place so that redesign of the project can minimize, prevent or compensate impacts properly and accordingly (G. Robert et al., 1996). The environmental management plan (EMP) is the procedures for achieving the objectives set out in the Environmental Policy and identified environmental performance targets for the project. It outlines the contractors approach to environmental management throughout the construction phases with the primary aim of reducing any adverse impacts from construction on local sensitive receivers (M. Ashwini Jajda et al., 2012). This project is involved a construction of road which is going to be constructed from Kalishuri to Surjomoni Union situated in Patuakhali district of Bangladesh. These areas is behind the good communication with Upazila headquarter and also other adjacent major cities. People of these two unions have to suffer for normal communication as they cannot transport in rainy season because of slippery condition of road, students have to suffer for going their educational institutions resulting poor transportation system. This is one of the most important proposed roads from Kalishuri to Surjomoni union that could be very important for people regarding to communication and development for two unions enhancing transportation, business opportunities, education, health care systems.

The objectives of this study are as follows:

To identify the significant envrionmental impacts of this road construction project

To explore the probable mitigation meausers to those significant impacts and

To develope the envrionmental management plan (EMP) of this project.

MATERIALS AND METHODS

Both primary and secondary ways were followed for completing this study. Primary data were collected from direct field survey, site observation, questionnaire survey through formal as well as informal ways. Firstly the probable environmental impacts are identified and classified in three categories namely ecological impact, physio-chemical and human interest. The secondary data were collected from various sources like Local Government and Engineering Department (LGED) of Patuakhali, Bangladesh Bureau of Statistics (BBS) 2011 census, Banglapedia, Bangladesh roads and highway department, and from different statistical reports, relevant research papers, books and many national and international journals have also been reviewed for this study.

Identified several environmental impacts of this road construction project are documented and then try to find out the possible mitigation measures based on those impacts. Environmental management plan were specified for getting sustainable outcome of this project as it's a prime concern of this study.

Location of the Project

The project is located near to the town of Patuakhali south-western Bangladesh. The proposed road will be connected two unions named Kalisuri and Surjomoni having total population 22249 and 20343 respectively (BBS, 2011).

RESULTS AND DISCUSSIONS

Analysing the likely Environmental Impacts and Mitigation Measures Ecological Impacts

Fisheries: The normal fisheries production and income from this sector will be reducing in comparison with the previous fish productivity before constructing the road. It is because due to construction of the road alongside the canals will affect the fish breeding and there availability in the internal surrounding areas. Road will prevent longitudinal and lateral migration of fishes in the flood plain.

Mitigation: Possible mitigation of this problem is to increase the internal fish and aquaculture practice which help to reduce the fish production and bring the nutrition security.

Plantation: There must be huge pressure on the existing vegetation in the projected area and most of the trees will be cut down. The attention should be given to restore and preserve the as much trees as possible within the study area.

Mitigation: To mitigate and resolve this impact some activities should be implemented for instance plantation of trees alongside the road. This sort of activities should sustain the project and will bring and conserve the biodiversity.

Physio-chemical impacts

Erosion and siltation: Erosion and siltation process will be acerbated due to construction of road alongside the canals. Siltation and erosion will be decay the project sustainability and bring new problem in some extend of the study area.

Mitigation: To overcome this problem it is essential to conduct both capital and maintenance dredging of the associated and adjacent canals.

Drainage congestion /Water logging: Drainage congestion and water logging will be increase in some location due to the regulation and proper maintenance of the drainage in the project area.

Mitigation: To overcome this problem effectively, there should be instalment of proper outlet and regulators in the project area.

Diseases and mosquito: Due to stagnant water and persistent water logging spread of different water borne diseases will be increased and mosquito breeding will be increased.

Mitigation: To mitigate the problem proper drainage system and its maintenance will be ensured.

Regional hydrology/Flooding: As the road will disrupt the normal water flow and discharge over a long area there might be change in the local and regional water system.

Mitigation: Design will be modified and adjusted that will reduce the effect and least influence to change the regional and local hydrology.

Obstruction to waste water flow: Road may obstruct the drainage of sewage and industrial waste water loading to serious pollution problem in the study area.

Mitigation: To meet the problem proper maintenance of the drainage will be ensured.

Impacts on human interest

Loss of agricultural lands: The implementation of this project will be reduce the agricultural land as it control the perennial floods so that the land that inundated by water and become cultivatable with flood water will be loose this opportunity and as result subsequent amount of agriculture land and production will be reduced.

Mitigation: Promotion of artificial irrigation and drainage system will improve the condition and production as well.

Generation of employment opportunities: Positive thing is that construction of road generates temporary employment during project implementation and permanent employment during maintenance phase

Commercial and service facilities: The road will provide benefit of fast communication, transport facilities and promotion of trade and commerce and create the employment opportunity for the local people and boost the economy.

Environmental Impact Value Estimation

By using $\mathbf{EIV} = \sum_{i=1}^{n} (V_i)\, W_i$ …… (1) Environmental impact value was estimated based on identified impacts. Estimated environmental impact value is depicted in (Table 1).

Table 1: Total environmental impact value estimation.

Environmental parameter	Total **($\sum RIV.DoI$)	Result ***(EIV) = $\sum^{n}_{i=1} (V_i)\ W_i$
Ecological impact	-5	-1
Physcio-chemical Impact	-91	
Human Interest	+95	

** RIV is relative impact value and DoI is degree of Impact.

*** EIV is environmental impact value, V_i is relative change of the environmental quality of parameters, W_i is relative importance or weight or parameter, N is total number of environmental parameters.

The Environmental Impact Value (EIV) considering both positive and negative is -1 (negative one). This shows a very low negative environmental impact on this project. Proper environmental mitigation and management plan can minimize the identified negative impacts of this road construction project. Then the project becomes environmentally acceptable. Thus it will give the clearance to go ahead with the project.

Environmental Management Plans (EMP)

The EMP of this project consists of various detailed sub plans. A set of EMP has been prepared to provide further guidance for managing certain environmental tasks and to ensure consistency in approach and quality of outcome.

Noise and vibration management measures

Construction noise levels will be managed through the implementation of the following requirements listed in Table 2.

Table 2: Noise and vibration management measures.

No.	Action	Responsibilities
Pre-Construction/Construction Phase		
	All Personnel will be informed of the importance of noise and vibration Management.	Construction Manager
	Adjacent peoples will be notified of construction activities and probable noise Emissions.	Construction Manager
	Before commences construction work (at least 20 hours), written or oral notice must be provided to the occupiers whom are likely to be affected by excessive project noise and vibration.	Construction Manager
Construction		
	The equipment used on the construction spot will be the quietest	Construction Manager

	reasonably available.	
	Works those are likely to create extreme noise or vibration levels to nearby residents will be conducted outside of normal working hours.	Construction Manager

Dust reduction and prevention management measures

The potential for dust generated from the clearing of vegetation, earthworks, spillage of soil material and vehicle movements along sealed and unsealed roads will be managed by the management measures itemized in Table 3**.**

Table 3: Dust reduction and prevention management measures.

No.	Action	Responsibilities
Pre-Construction/Construction Phase		
	All Personnel will be well-versed of the status of dust management.	Construction Manager
Construction		
	Movement of construction traffic and speed limits on unsealed surfaces will be controlled.	All Personnel
	Construction activities on windy days will be limited.	
	Dust prone areas (roads, stockpiles) will be kept moist with water.	Construction Manager

Hydrology and Drainage Management Measures

For mitigating any surface water, groundwater and drainage impacts that may occur, the following management measures listed in **Table 4** will be initiated.

Table 4: Management measures for hydrology and drainage.

No.	Action	Responsibilities
Pre-Construction		
	All Personnel will be informed of the importance of hydrology and drainage management.	Construction Manager
	A site drainage assessment will be undertaken to inform detailed design.	Construction Manager
Construction		
	All potentially contaminated storm water (sediment and hydrocarbons) will be treated prior to discharge to the environment.	Construction Manager
	Sediment basins will be examined and maintained regularly.	Field Assistant

Vegetation Management Measures

Management measures for vegetation is listed in Table 5.

Table 5: Management measures for vegetation.

No.	Action	Responsibilities
Pre-Construction		
	All Personnel will be informed of the importance of vegetation management	Construction Manager
	A Landscaping Plan will be developed.	Construction Manager
Construction		
	Cleared vegetation will not be burned within the project area.	Contractor
	Cleared native vegetation will be retained for later rehabilitation purposes.	Contractor
	Drainage or runoff from construction areas will be managed so that it does not cause erosion or sedimentation and potentially affect retained vegetation.	Contractor

Monitoring and reporting

Monitoring is an integral part of the EMP as it establishes how the project is performing against objectives and targets set in the EMP. **Table 6** Shows the monitoring responsibilities and related Authorities.

Table 6: Monitoring responsibilities and authorities.

Monitoring Item	Method	Frequency	Location	Responsibilities
Air and sound Pollution	Visual Inspection	Daily (During Construction)	Construction site and adjacent area	Site Supervisor
Water Pollution	Water Sampling	Weekly	Construction site and adjacent area	Field Assistant
Soil Pollution	Soil Sampling	Monthly	Construction site and adjacent area	Field Assistant
Agriculture	Site and Work Observation	Pre-start of works and monthly thereafter	Nearby Agricultural Land	Environment Officer

Significance of EMP

Strict implementation of Environmental Management Plan (EMP) can brings the best outcome and success of the project in term of reducing negative environmental impacts of this project.

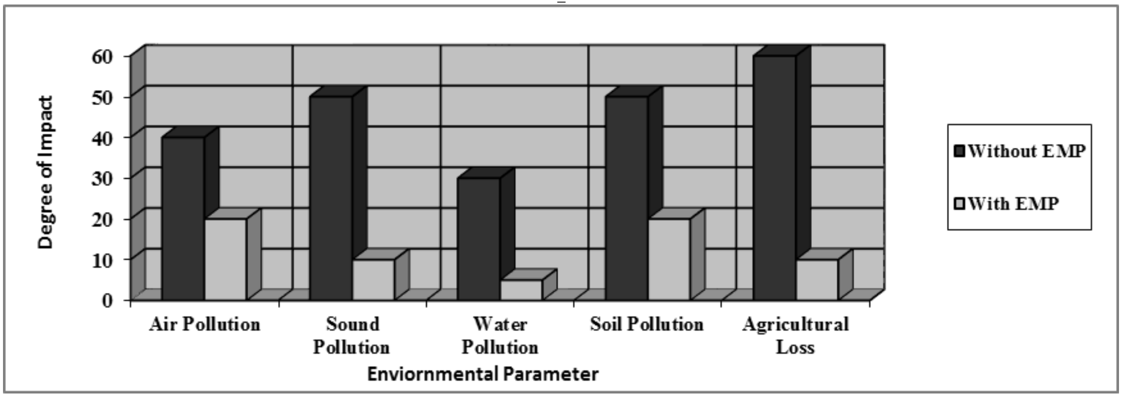

Figure 1: Significance of environmental management plan regarding with and without EMP.

CONCLUSION

This project has major impacts on air, water, soil, biodiversity. Environmental impact value estimated considering both positive and negative was -1 (negative one). These impacts can be reduced by adopting eco-friendly technologies and enhancing proper implementation of EMP. For attaining sustainable outcome of this project various environmental management plans were designed as includes noise and blasting management plan, dust reduction and prevention plan, hydrology and drainage management plan and vegetation management plan. Positive impacts such as generation of employment opportunity, improve communication system etc. can strengthen the implementation of the project. As a result people benefited as solving transportation problem of the proposed area.

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HEALTH RISKS ASSOCIATED WITH WORKERS DUE TO COAL DUST IN PIGMENT PRODUCTION FACTORIES

M. N. Alam[1*], M. S. Islam[1], M. S. Mia[1], N. F. Shemu[1] & G. C. Saha[1]

[1] Department of Civil Engineering, Dhaka University of Engineering and Technology, Gazipur- 1700, Bangladesh.
**Corresponding Author, <nazmulce@yahoo.com>*

ABSTRACT

Gazipur is one of the biggest industrially developed district in Bangladesh where a huge number of people are working in various manufacturing industries. Among these industries some are producing pigment which is used for dado of building. Due to rapid urbanization the demand of this pigment is increasing highly across the country. In the process of producing pigment, firstly the mixture of Iron fragment and Coal fragment are pulverized and by mixing them with water a paste is made. Then the paste is dried by sun light and the dried chunks are ground again. During this procedure a lot of dusts are evolved in those factories. The internal environments of those factories have not developed properly. That's why those are causing hazardous effects on health of workers and surrounding environment of the factory.

Coal is toxic because elements such as arsenic, mercury, lead, nickel, cadmium, selenium, vanadium, and copper are accumulated and concentrated within coal. Coal dust is produced by grinding and pulverizing of coal. The fine particulate nature of coal dust and its toxic constituents are readily inhaled by the workers. These lodge in the lungs, where they can travel to the heart and circulatory system. Many studies have shown that people exposed to coal dust have higher rates of health problems, including cardiopulmonary disease, chronic obstructive pulmonary disease, high blood pressure, lung disease, and kidney disease. The main aim of this study is to find out whether people working inside pigment factory are exposed to coal dust that may cause which sorts of health risks. This study also aimed to find the variation and intensity of diseases suffered by workers due to work there with respect of working time duration. In order to study the health hazards of pigment producing factory on workers, three factories were considered for this study. The workers were administered with the standard format questionnaire which was followed by personal interviews. The results indicate a visible and invisible impact on health of workers and the health related problems increases.

Long-time inhalation of coal dust will exacerbate this problem which could increase fatality of those diseases as well as mortality further in future. This study would emphasize the urgent need for prevention measures of health risks and improving the internal production mechanism of those factories.

Keywords: Coal Dust, Rapid urbanization, Hazardous effects, cardiopulmonary disease, Health risks.

INTRODUCTION

In building construction, for colouring the skirting of superstructure wall, a blackish pigment is mostly used in all over Bangladesh. Fifteen to twenty years ago this pigment was imported from India when there was no factory in Bangladesh. Then few entrepreneurs started to manufacture this pigment locally. There are five pigment production factories are identified in Gazipur city. The processes of manufacturing pigment have not been developed as required to prevent health risks of workers and ambient air pollution. Coal is a brown to black combustible material made from decayed plant matter that has been compressed by rock formations over a long period of time. It is found throughout the world and is the most abundant of the fossil fuels [1]. Coal dust is powdered form of coal. This contains toxic heavy metals including arsenic, lead, thallium, barium cadmium, chromium, mercury and nickel. The pigment is produced mainly from iron splinter, coal and chemicals. When these ingredients are pulverized huge amount of dust, gases, noise and vibration are emitted from machinery. The impacts of pigment industries are countless and adverse to health of workers when they work there without any preventive kits.

METHODOLOGY

Three factories were taken in consideration for survey. All subjects were served with a questionnaire which was made by help of doctors. Firstly some probable diseases caused by coal dust were selected and their symptoms are noted in English. Then the English disease name and symptoms were translated into Bengali to make it easily understandable to the workers. The workers working in the pigment factories were administered with the questionnaire and total of 80% of employees in pigment factories were studied. Later interviews were conducted.

The questionnaire format and interviewed questions were as follows.

WORKERS HEALTH QUESTIONNAIRE

Name :

Age :

No. of years since working in the factory :

Marital status :

1. Do you smoke tobacco?

Yes	**No**

2. If yes, how many packs per day? __________ Number of years ___________

3. Have you ever had any of the following conditions? (indicate yes or no for each)	**Yes**	**No**
a. Seizures (fits)		
b. Diabetes (sugar disease)		
c. Allergic reactions that interfere with your breathing		
d. Trouble smelling odors		
4. Have you ever had any of the following pulmonary or lung problems?	**Yes**	**No**
a. Asbestosis		
b. Chronic bronchitis more than 3 episodes in the last year		
c. Emphysema		
d. Lung cancer		
e. Silicosis		
f. Chest injuries or surgeries		
g. Asthma as an adult		
h. Pneumonia in the last month		
i. Tuberculosis (active disease)		
j. Any other lung problem		

5. Do you currently have any of these symptoms of pulmonary or Lung illness?	**YES**	**NO**
a. Shortness of breath		
b. Shortness of breath with light activity		
c. Shortness of breath with strenuous activity		
d. Cough that produces thick sputum or blood		
e. Cough lasting longer than 3 weeks		
f. Wheezing		
g. Wheezing that interferes with work		
h. Any other symptoms that may be related to lung problems:		

6. Have you ever had any of the following cardiovascular or heart problems?	**YES**	**NO**
a. Heart Attack		
b. Stroke		
c. Angina (chest pain)		
d. Heart failure		
e. Irregular heart beat		
f. Swelling in your legs or feet (not caused by walking)		
g. High blood pressure		
h. Any other heart problems:		

7. Have you ever had any of the following cardiovascular or heart symptoms?	**YES**	**NO**
a. Frequent pain or tightness in your chest		
b. In the past two years, have you noticed your heart skipping or missing a beat?		
c. Heartburn or indigestion that is not related to eating		
d. Any other symptoms that may be related to heart or circulation problems		
8. Do you currently take medication for any of the following problems?	**YES**	**NO**
a. Breathing or lung problems		
b. Heart trouble		
c. Blood pressure		
d. Seizures (fits)		

9. Do you use respirator during work.		
10. If you've used a respirator, have you ever had any kind of problem?	**YES**	**NO**
a. Eye irritation		
b. Skin allergies or rashes		
c. Anxiety		
d. General weakness or fatigue		
e. Any other problem that interferes with your use of a respirator		
11. Which health specialist do you visit most frequently?		

A) What kind of medicine do you use mostly?
12. How much a day.

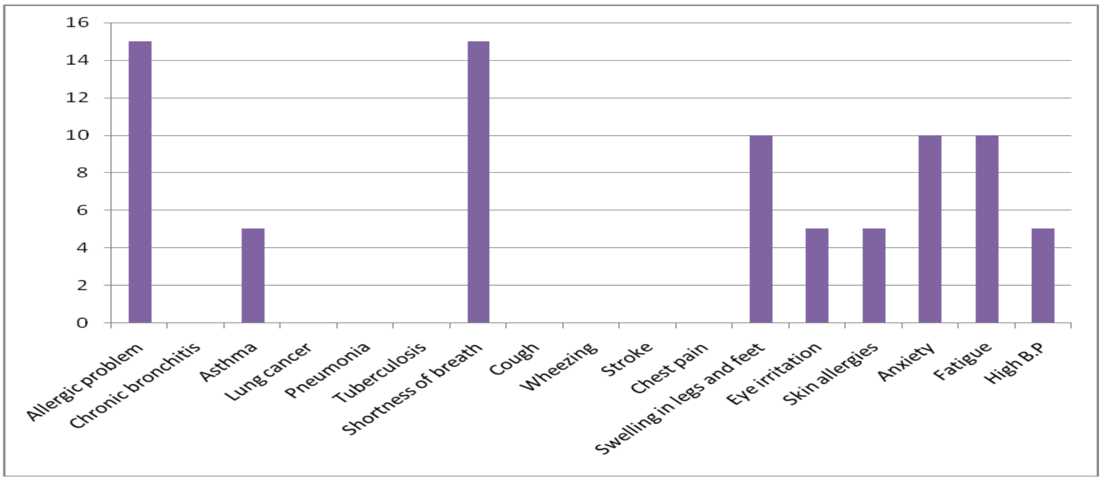
Date: _______________ Comments: ______________

Thank you for your help!

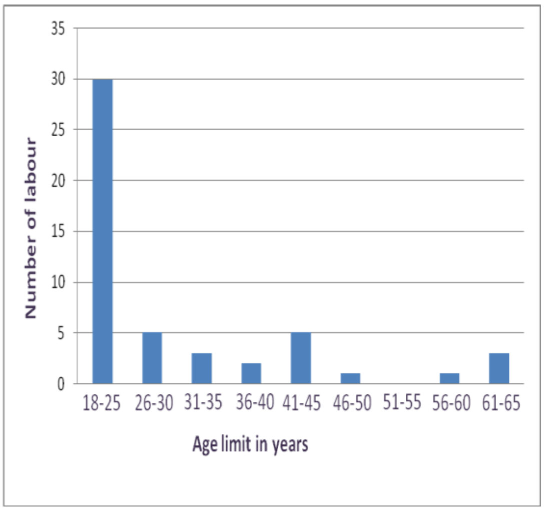
Another Interview Questions asked to workers

- Do you use any safety devices such as masks, respirator etc.
  - If, yes what?
  - If, no why?
  - Does the owner provide you with safety devices?
  - If, yes, are they functional?
  - Do you have any family history for any diseases?

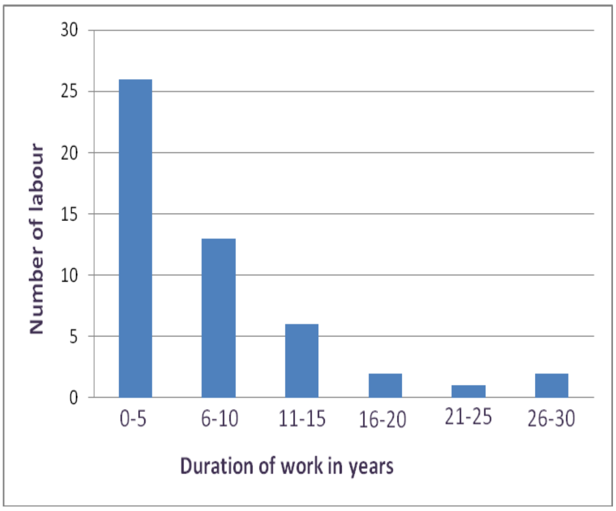
RESULTS



[Fig.1] Intensity of diseases among workers.



[Fig.2] No of worker vs. age limit graph.



[Fig.3] No of worker vs. their working duration graph.

There are seventeen kinds of diseases [Fig.1] were recorded from the labours in the pigment production in Gazipur city. These included: Allergic problem, chronic brochitis, asthma, emphysema, Lung cancer, pneumonia, tuberculosis, shortness of breath, cough, wheezing, stroke, chest pain, swelling in legs and feet ,eye irritation, skin allergies, anxiety and fatigue, High B.P. Ages of greater part of labours were within the limit 18-25 years and duration of work within 0-5 years [Fig.2] & [Fig.3].

- [Image 1] Workers in a factory working without preventive kits.

[Image 1] Young aged workers of a pigment production factory.

DISCUSSIONS

We have gathered some experience during the execution of this work. They are mentioned bellow.

- Workers were illiterate and unwilling to provide authentic information of some cases.
- Workers do not go to doctor if they suffer in allergic problem.
- The effects of working in this factory would cause fatal problem to health when worker will be old in age.
- There were no pollution control devices installed in factories.
- Workers use tube well installed in those factories but the underground water may have heavy metal due to infiltration of coal slurry.

CONCLUSION

Majority diseases were found in labors who were old in age and working for long period. But the numbers of old labors were considerably less than that of young labors. All of them were not conscious about the diseases that may occur due to inhaling coal dust. The study shows that the workers work in those factories for first few years and then change their job to anywhere. So the effect of coal dust on health could not detect as assumed from another study. If some medical tests on health were executed the accurate effect of coal dust could be determined. Another point of concerning that the infrastructures of those factories are not good. There were not enough shades and walls around the factories. For this reason the factories pollute ambient air very much. Some studies have shown that the slurry of coal dust can infiltrate into ground water and can contaminate that ground water with the presence of heavy metal. The water is needed to be tasted to know the presence of heavy metal.

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POLLUTION VS. REGULATIONS-SCREENING OF PHYSICAL AND CHEMICAL PARAMETERS NEAR REAL TIME IN TANNERY AT BEAM HOUSE, HAZARIBAGH, BANGLADESH

M. A. Hashem[1*], S. Mohsin[1] & A. Islam[1]

[1] *Department of Leather Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.*
**Corresponding Author, <mahashem96@yahoo.com>*

Physical and chemical parameters have been screened near real time at beamhouse operations in ten (10) tanneries and comply with the industrial regulations. In leather processing beamhouse operations are involved a series of chemical treatments which are produced a substantial amount of solid and liquid wastes which are discharged as green to the environment. The pH of the waste liquids was higher/lower than the standard level of industrial regulations in Bangladesh. In liming pH was 13 which was higher than the standard level. On the other hand, in pickling pH level was extremely low (pH $\leq$1.0). The level of DO for all stages in beamhouse was extremely low (<1). The total suspended solids (TSS) were 58-146 times and total dissolved solids (TDS) were 15-28 times higher than the standard level. The liquid waste containing low DO, high/low pH, high TSS and TDS are directly discharged to the environment nearby low-lying area and finally falls to the river, Buriganaga. Suspended matters get deposited on the bed of the stream and kills aquatic organisms; floating solids affect the stream's ability for self-purification by the re-generation of oxygen from the atmosphere and also it distresses to the photosynthesis activity for stream plankton and aquatic plants. Suspended solids caused the turbidity which decreases the infiltration of sunlight into the water thus reduces the photosynthesis activity of aquatic plants. Authorities should take initiative to neutralize the wastes before disposal in the environment to save the aquatic life as well as the cleaner leather production.

Keywords: Suspended solids (SS), Dissolved solids (DS), Total solids (TS), pH, Dissolved oxygen

INTRODUCTION

Worldwide leather industries appreciate for growing importance of waste minimization. Rapidly increasing population concomitantly rising environmental awareness mean that waste treatment as well as disposal practices of the past are either no longer acceptable. As a result, leather industries are faced with the dual obligation of **i**) using fewer chemicals more effectively and **ii**) improving treatment of all forms of the process residues including solid, liquid and gaseous.

Leather industries in Bangladesh are also well-known as a highly polluting industry due to generating huge amount of liquids, solid and gaseous pollutants. Due to generating excessive environmental pollution developed countries are going to discontinue leather production and some of countries already being brought to the end. In contrast, leather production is migrating from the developed countries to developing countries like Bangladesh, India because still they are not taking so much care about the environmental regulations.

In Bangladesh, out of 270 leather industries 243 are located in the western part of the capital city Dhaka at Hazaribagh covering area of 25 ha (Salam, 1998). Three sides of the industrial area are surrounded by residential areas, the western side is by flood embankment and others are scattered all over the country (Salam, 1998; Marjjuk, 2012).

Owing to generating solid, liquids and gaseous pollutants as well as causing environmental pollution; leather industries of Bangladesh also has achieved a negative impact in the society thus facing a stringent challenge. Conversely, leather and its products are one of the biggest exports earning sector to strengthen the national economy for the country. It has been reported by Export Promotion Bureau that in the year of 2011-12 Bangladesh earned US$765 million from leather sector as well as gradually demands of finished and fashionable leather products are growing all over the world (Hashem et al. 2014).

The huge amount of produced solid and liquid wastes from the leather industries are discharged directly from the tannery through the drain and deposited nearby low-lying areas and finally falls to the river Buriganga which are responsible for the pollution of the major linked rivers of Bangladesh. The liquid waste contains heavy metal like chromium (Cr). After conventional chrome tanning waste water contains chromium level is 1500-3000 mg/L (Suresh et al., 2000). Each and every steps of tannery operation different types of gaseous pollutants such as hydrogen sulphide (H_2S), ammonia (NH_3), chlorine (Cl_2), sulphur dioxide (SO_2) etc. are produced and directly merge to atmospheric environment (Das, 2000). In the conventional deliming operation yearly 3.5×10^6–12.8×10^6 m^3 ammonia is produced at Hazaribagh, Bangladesh from where a good fraction of ammonia is directly merged to atmosphere (Hashem et al., 2014).

A good number of researchers have accounted on tannery waste in Bangladesh by characterization of the parameters and their impact assessment on the environment (Das, 2000; Ahmed, 1997). In the most cases samples were analyzed after a long later of collection. Hence, there is some possibilities to change samples' original condition and may not perform as real sample results. Some of them characterized tannery waste liquors without distinguishing operation (Rouf et al., 2013) from where it is difficult to identify the specific effluent load from the specific operation in leather processing.

In this study physical and chemical parameters of tannery liquid wastes were exclusively monitored in beamhouse operation near real time. The physical parameters- total suspended solids (TSS), total dissolved solids (TDS) and chemical parameter-dissolved oxygen (DO) and pH are monitored to comply with the environmental industrial regulations in Bangladesh in beamhouse operations.

MATERIALS AND METHODS

Study area

The study area is comprised of tanneries at Hazaribagh, Dhaka, Bangladesh. Large and medium categories ten (10) tanneries were selected for collecting the liquid wastes. The selected tanneries are produced crust and finished leather all the year round from wet-salted or raw hides and skins.

Sample collection

In the **Fig. 1** shows the operational sequences for chemical treatment in beam house preparations. Each and every operation different types of chemical agents are used to obtain special type of properties or action. Liquid samples were collected just after chemical operations. After completing each chemical operation, liquids were collected into polyethylene bottles and were brought back to the laboratory for examination of physical and chemical parameters. Samples were kept in the refrigerator at 4ºC until to complete the experiment.

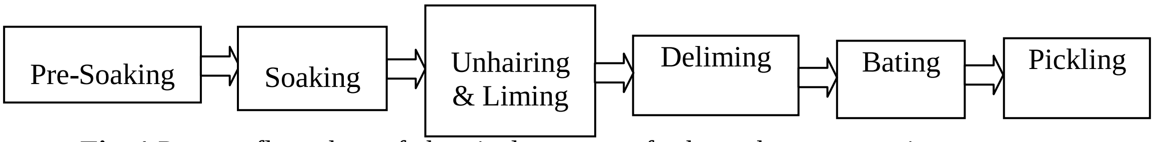

Fig. 1 Process flow chart of chemical treatment for beam house operations

METHODOLOGY

Determination of pH and Dissolved Oxygen (DO)

The pH and dissolved oxygen (DO) of the liquid wastes were measured on that day by using the pH (UPH-314, UNILAB, USA) and DO (HQ40d, HACH, USA) meter, respectively. Before measuring pH and DO both the meters were calibrated with the standard solutions.

Determination TSS and TDS

Both the total suspended solids (TSS) and total dissolved solids (TDS) were determined gravimetrically as per standard methods of APHA (APHA, 1999). A 10 mL liquid waste was passed through the glass fiber filter and dried at drying oven at 103−105°C until a constant weight is obtained.

RESULTS AND DISCUSSIONS

pH

The pH value at different operations from pre-soaking to pickling is shown in Fig. 2. The pH value were in pre-soaking (5.7–7.5), soaking (5.9–10.0), liming (13.1–13.3), deliming (7.4–9.5), bating (7.4–9.4) and pickling (1.0–1.1), respectively. It seems that pH value for liming and pickling were beyond the standard for waste from the industrial unit (pH 6–9) (ECR, 1997). On the other hand, pH values for pre-soaking, soaking, deliming and bating were closed to standard for waste from industrial unit (ECR, 1997). The high alkaline lime liquor pH (13) value was higher than the standard level. On the other hand, pH value in the pickling was extremely low (pH ≤1.0) than the standard value. After mixing, prior to discharge to the river, Buringaga from the different operations as well as from the different tanneries the pH level was 7.9. The liquid wastes which are discharged through drain to the river, Buriganga which contain sodium chloride, blood, dirt, wetting agent, bactericides, keratin protein, dissolved hide/skin proteins, unused sodium sulphide, calcium hydroxide, ammonium salts, chromium etc. are causing aquatic problem.

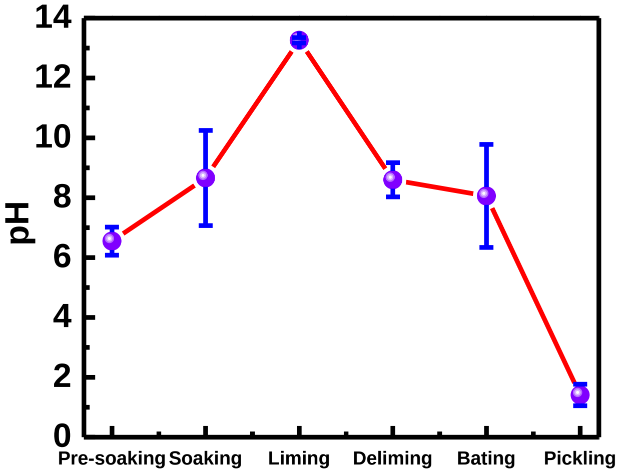

Fig. 2 pH levels at different stages in beam house operation.

Dissolved oxygen (DO)

In Fig. 3 shows the dissolved oxygen (DO) level from pre-soaking to pickling. The DO levels were extremely low (1<) and in some stages DO values were almost zero whereas the standard value for waste water discharge from the industrial unit is 4.5–8.0 mg/L (ECR, 1997). The surface water DO level is 8.5 mg/L at 24ºC (EPA, 2001). After completing tannery operations, liquid wastes are thrown directly to drain to fall the low-lying area. The liquid waste from the various operations as well as from the different tanneries finally falls to the river, Buriganga which is one of the major reasons to decrease DO level in the river, Buriganga. As a result, numbers of fishes as well as aquatic plants are disappearing due to lack of dissolved oxygen from the river, Buriganga.

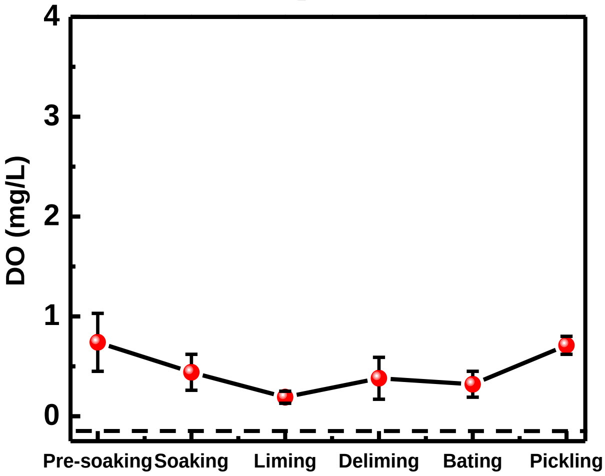

Fig. 3 Dissolved oxygen level at different operations

TSS and TDS

Total suspended solids (TSS), total dissolved solids (TDS) and total solids (TS) are shown in Fig. 4. The suspended solid as well as dissolved solid value is so high and in some cases the level was several times higher than the standard level. Especially in pre-soaking, liming, and bating the suspended solids level was 118, 206, and 100 times higher than the standard level, respectively. Suspended matter gets deposited on the bed of the stream thus kills aquatic organisms of the stream bottom. The floating solid interfere the streams ability for self-purification by re-generation of oxygen absorbing from the atmosphere. It also interfere the photosynthesis activity of the stream plankton and aquatic plants. The suspended solids are caused the turbidity which decreases to infiltrate the sunlight into water and thus reduces the photosynthesis activity of aquatic plants.

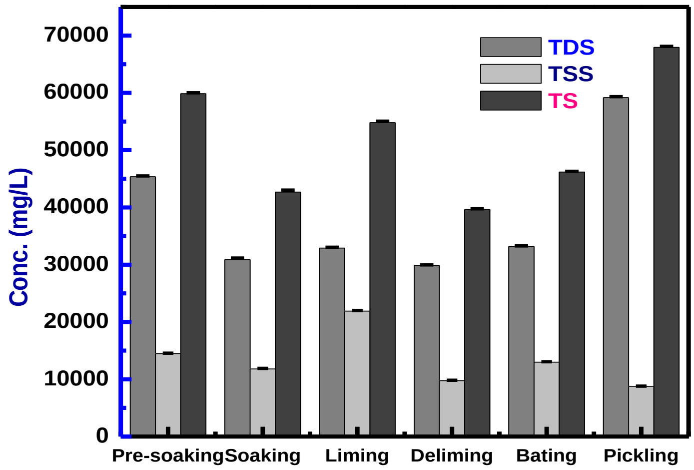

Fig. 4 TSS, TDS and TS level at beam house operations

Effect of pH, DO and suspended matters in aquatic life

The pH of water is essential for normal physiological functions of aquatic organisms, including the exchange of ions in the water and respiration. Such essential physiological process can be activated in most aquatic biota under a relatively wide pH range 6.0─9.0 (NAS, 1972). Beyond this pH ranges become stressful and potentially lethal to fish; even organisms may also be dead if pH is getting too far from their optimal range. Nevertheless low pH like in pickling (pH, below 2) containing chloride can dissolve toxic elements from the sediment or the metal tanning vessels which may be taken up by aquatic animals or plants (Addy and Elizabeth 2004). It is reported (Alabaster et al., 1980) that pH below 5.0; the productivity of aquatic ecosystems is considerably reduced which successively reduces the food supply for higher organisms. Resulting fish present would likely experience reduced numbers and/or growth rates. If the pH is higher the toxic hydroxyl ions is hypertrophy of mucus cells at the base of the gill filaments and destruction of gill and skin epithelium, with effects on the eye lens and cornea (Alabaster and Lloyd, 1980; Boyd, 1990).

Dissolved oxygen is essential for survival of aquatic livings. Low dissolved oxygen (DO) primarily results from excessive algae growth; resulting insufficient amounts of dissolved oxygen available for fish and other aquatic life. Deficiency of DO also occurs death of submerge plant.

The suspended matters can be altered taste, odor, temperature and reduces the levels of DO particularly in deeper. High total solids (TS) are highly responsible turbidity of water which causes decrease in photosynthesis process because turbidity impedes deep penetration of light in water (Muoghalu and Omocho, 2000). If total dissolved solids (TDS) are greater than 1000 mg/L is considered to be blackish. The investigated result is several times higher than the standard level. The TDS causes toxicity through increase in salinity, changes in the ionic compositions of the water and toxicity of individual ions. (Weber-Scannell and Duffy, 2007). The water quality of river, Buriganga is becoming worse due to receiving liquid wastes from tannery.

CONCLUSION

In tannery, beamhouse operations produce huge amount of waste liquids which containing extremely high total dissolved solids (TDS) and total suspended solids (TSS). The dissolved oxygen level is extremely low for living aquatic livings. The pH of the liquid waste was highly acidic and highly alkaline. The physical and chemical parameters of liquid waste were beyond the permissible level. The entire discharged waste liquids' final reservoir is the river, Buriganga which effect on aquatic life as well as ecological balance. Aquatic livings like fish are becoming disappeared due to lack of dissolved oxygen. The suspended solids are increased the turbidity in water which causes to decrease the photosynthesis process because turbidity impedes deep penetration of sunlight into water. Tannery owners as well as the concerned authorities have to neutralize the liquid wastes before discharging to the environment for cleaner leather production.

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SOLID WASTE GENERATION DURING FLESHING OPERATION FROM TANNERY AND ITS ENVIRONMENTAL IMPACT: BANGLADESH PERSPECTIVE

M. A. Hashem[1*], M. S. Nur-A-Tomal[1] & B. K. Mondal[2]

[1] *Department of Leather Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.*
[2] *Apex Tannery Limited, Dhaka-1209, Bangladesh.*
**Corresponding Author, <mahashem96@yahoo.com>*

ABSTRACT

In situ investigation has been carried out to estimate solid waste generation in fleshing operation from wet salted cow hide and goat skin from tannery. Conversion of putrescible raw hide and skin into imputrescible leather is a complex tanning technology. Fleshing is one of the most indispensable mechanical operations in beamhouse where substantial amount of solid waste is produced. Conventionally, fleshing is preferred after liming as pelt is easier to handle in swollen condition. It is estimated that yearly 20.1×10^3 MT solid waste is generated only in fleshing operation from tanneries in Bangladesh where 10.3×10^3 MT for cow hide and 9.8×10^3 MT for goat skin. Produced huge amounts of fleshings are usually kept inappropriately inside or outside of the industrial area as green. Proteinaceous substance in fleshings is hydrolysed to amino acids by proteolytic bacteria; amino acids are hydrolysed through bacteria and liberate ammonia (NH_3), hydrogen (H_2), carbon dioxide (CO_2), volatile fatty acids (VFAs) etc. which are directly merged to atmosphere. People of the tannery area are frequently inhaled the liberated gases and suffering in difficulties. During rainy season all fleshings and other solid wastes are washed away to fall low lying area and finally to the river, Buriganga. Fleshings have negative effects on soil and plant growth due to lime contains in fleshings which spoils the fertility of soil. Waste management is one of the best ways environmental friendly leather productions. Further research could be a dynamic mode to reuse of fleshings for glue, gelatin, biogas, biodiesel etc. production.

Keywords: Solid waste, Fleshings, Proteinaceous, Volatile Fatty Acids, Impact

INTRODUCTION

Leather industry plays a vital part in the economy of Bangladesh regarding gross output, value addition, manufactured export and employment (Karim, 2000). Export Promotion Bureau (EPB) reported that Bangladesh earned $980.67 million in the fiscal year of 2012-13 from the leather sector. But leather industry is facing a great challenge due to generating environment pollution (Ahmed, 2012). Leather industry has been categorized as '**red**' category industry in Bangladesh by the Department of Environment (DoE) due to generating high pollutants (Tigga et al., 2000). The 90% leather industries are located at Hazaribagh, Dhaka and after production discharged solid and liquid wastes directly through drain as green to low-lying areas and residential area without treatment (Zahid et al., 2006).

The conversion of putrescible raw hide and skin into imputrescible leather is known as tanning; tanning technology is a complex procedure comprising several technological steps (Wieczorek-Ciurowa et al., 2011). Each and every technological steps of leather making process generates significant amounts of solid and liquid waste as well as gaseous air (Anupama et al., 2013). Liquid tannery waste is highly polluted in terms of biochemical oxygen demand (BOD), chemical oxygen demand (COD), suspended solids, total nitrogen, conductivity, sulphate, sulphide, and chromium etc. The waste liquid is discharged from tannery either directly to river or to drains or canals which subsequently find their way into the river (Rahman et al., 2010). The green liquid waste (without treating) from tanneries at Hazaribagh, Dhaka is the main reason to pollute the river, Buriganga (Azom et al., 2012). Besides, tanneries release odours and other volatile organic compounds (VOCs) from biological decomposition which take place in stored raw hide/skin, liquid waste including ammonia (NH_3), hydrogen sulphide (NH_3), volatile hydrocarbons, amines and aldehydes (Famielec et al., 2011). In conventional deliming operation yearly 3.5×10^6–12.8×10^6 m^3 ammonia is produced at Hazaribagh, Bangladesh; depends on solubility of ammonia at pH (8.5–9.0) a good fraction of ammonia is merged to atmosphere (Hashem et al., 2014).

In various stages of leather process leather industry generates significant amounts of solid wastes. It is reported that conversion of every 10kg raw hide/skin into leather is produced more than 6kg of solid waste (Boopathy et al., 2013). The quality and quantity of solid wastes depends on many features (Ozgunay et al., 2007). The solid wastes consist of curing salt, raw trimmings, keratins, fleshings, shaving and buffing dust etc. Most of the solid wastes generate in pre-tanning. In Bangladesh solid wastes are dumped extensively in open places near leather industries; during rainy season all these wastes are washed away through drain to low lying area and finally fall to river, Buriganga.

The solid waste generate in fleshing operation term as ‘fleshings’. The operation involves cutting removing unwanted part from side of hide and skin. In conventional leather processing fleshing is done after liming as the hide/skin is easier to handle in swollen condition. Fleshing could be done manually or mechanically. Whatever the method it produce huge amount of solids waste. It is one of the most important operations; if flesh is not removed diffusion of tanning agents and other chemicals would be inhibited into hide/skin from the flesh side. Fleshings contain sub-cutaneous tissue, fat and flesh without removing it is impossible to manufacture good quality leather. It contains protein 5−7%, fat 4−18%, sulphide 2−4% etc. (Lupo, 2006).

In this study in situ an estimation of solid waste during fleshing operation from the wet salted cow hide and goat skin has been carried out as well as its environmental impact assessment.

MATERIALS AND METHODS

Study area

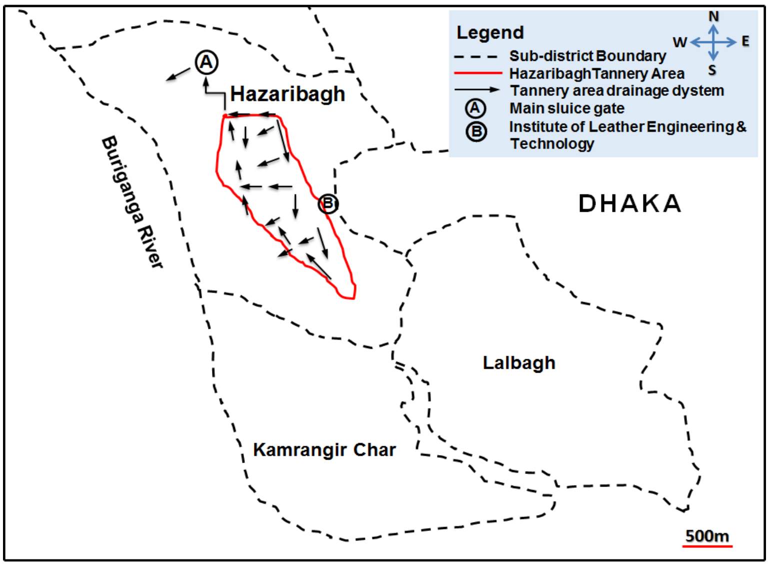

Fig. 1 The study area is located at Hazaribagh, Dhaka

The study area is located at Hazaribagh tannery area, Dhaka where 90% tanneries of Bangladesh are produced leather. One of the largest tanneries has been selected to carry out the research work. The study area is located in the Fig. 1.

METHODOLOGY

Sample and Materials

Wet salted cow hide and goat skin were collected from local raw hide market of Postogola, Dhaka, Bangladesh. Tannery machineries for processing of raw hide and skin, digital balance were used.

Methods and Analysis

The sequential operations namely raw trimming, weighing, soaking, unhairing & liming and fleshing are carried out. In Fig. 2 shows the operational steps for pre-tanning operation up to fleshing operation.

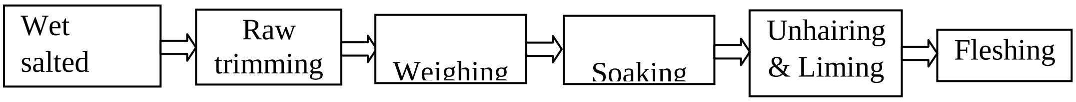

Fig. 2 Flow chart for conventional pre-tanning up to fleshing operation

Material balance analysis technique was applied i.e., weighing the samples before and after fleshing operation. A series of cow hide and goat skin were weighted before and after fleshing to estimate solid waste. Physical monitoring and interviews were made with relevant personnel at Hazaribagh tannery area to assess the environmental impact of fleshings.

RESULTS AND DISCUSSIONS

Generated Amount of fleshing

Generated amount of fleshings from the wet salted cow and goat skin is shown in Table 1 and Table 2. It is clear from the Table 1 that per kg wet salted cow hide generates 0.31kg fleshings. In case of wet salted goat skin (Table 2) fleshings generate 0.39kg.

Table 1 Generated amount of fleshings from wet salted cow hide

No.	Wt. of wet salted cow hide (kg)	Pelt wt. (kg)		Fleshings wt. (kg)	Fleshings (kg/kg)	Average wt. (kg/kg)
		Before fleshing	After fleshing			
01	22700.00	35716.80	27900.00	7816.80	0.34	
02	17000.00	25900.00	21100.00	4800.00	0.28	
03	20600.00	30894.40	25000.00	5894.40	0.29	0.31 ± 0.02
04	20200.00	31769.60	25700.00	6069.60	0.30	
05	21400.00	33656.00	27100.00	6556.00	0.31	

Table 2 Generated amount of fleshings from wet salted goat skin

No.	Wt. of wet salted goat skin (kg)	Pelt wt. (kg)		Fleshings wt. (kg)	Fleshings (kg/kg)	Average wt. (kg/kg)
		Before fleshing	After fleshing			
01	2900.00	3855.00	2700.00	1155.00	0.40	0.39 ± 0.02

02	2700.00	3802.05	2800.00	1002.05	0.37	
03	3050.00	4622.65	3300.00	1322.65	0.43	
04	3850.00	5086.10	3600.00	1486.10	0.39	
05	3550.00	4847.50	3500.00	1347.50	0.38	

Based on the FAO report in 2012, yearly 33.80 thousand tones wet salted cow hide and 24.80 thousand tones wet salted goat skin are taken for leather production in Bangladesh (FAO, 2013). It has been estimated that yearly 10.3×10^3MT fleshings from wet salted cow hide and 9.8×10^3MT fleshings from wet salted goat skin are generated during fleshing operation in leather industry of Bangladesh. Annually, the sums of the fleshings both from cow hide and goat skin is 20.1×10^3MT. It is a large amount solid waste which is discharged as green directly from the tanneries through drain/dumped nearby road side.

Effect of fleshings on human health

There is no direct effect of fleshing on human health. As fleshings contain proteinaceous substances which are hydrolysed to amino acids through proteolytic bacteria. The amino acids are further hydrolysed by bacteria which liberate ammonia (NH_3), hydrogen (H_2), carbon dioxide (CO_2), volatile fatty acids (VFAs) etc. which is directly merged to the air (Shanmugam et al., 2009). The produced VFAs help to generate toxic hydrogen sulphide (H_2S) gas from the fleshings. People of the tannery area are frequently inhaled the liberated gases and suffering in difficulties. Hydrogen sulfide can affect the nervous system, causing headaches and nausea, irritation of the skin, eyes, and respiratory tract, and at high concentration of H_2S (> 900ppm) for one minute causes instant coma and death (UNIDO, 2001). Ammonia is irritating to the eyes, respiratory tract and skin (USDHHS, 2004). Carbon dioxide is believed to be a possible aspiration and a simple asphyxiate.

Effect of fleshings in atmosphere

The gaseous forms of H_2S, NH_3 VFAs are produced from fleshings are directly merged to atmosphere and they have negative effect. The so called volatile fatty acids (VFAs) such as acetic acid, propanoic acid and butanoic acid which are responsible to provide H-atom in the atmosphere (Atkinson, 1989). The fluxes of H_2S led to toxic levels of H_2S at atmosphere by the atmospheric photochemical reaction. Also, ozone shield destroys and increase the greenhouse methane gas (Kump et al., 2005). The exposed hydrogen sulphide burns with blue flame to atmosphere and produces sulphur dioxide (SO_2) (OSHA, 2005); SO_2 is less hazardous than H_2S. The sulphur dioxide is the major precursors of acid rain; it reacts with atmospheric oxygen and water vapor to produce sulfuric acid, the so-called acid rain. Acid rain has the negative effect on soils, lakes and streams; it accelerates the corrosion of buildings and monuments and reduces the visibility (Padhan and Kumar, 2013).

Ammonia once is emitted to atmosphere it could undergo conversion to NH_4^+ aerosol due to its highly reactive nature and quickly deposited near to the sources of emission (McCulloch et al., 1998). The conversion of ammonia (NH_3) to ammonium ion (NH_4^+) in aerosol or in clouds is depending on the concentration of acids in atmosphere.

Effect of fleshings on aquatic life

During rainy season dumped fleshings and other solid wastes are washed away to fall low lying area and finally to the river, Buriganga. Fleshings contain lime and sodium sulphide cause to increase the pH, COD, BOD, total solids (TS) and decrease the dissolved oxygen (DO) of the aquatic body. As a result aquatic effluence is triggered by the fleshings.

Effect of fleshings on plant growth

Fleshings have high pH value because of lime which spoils fertility of soil. Fleshings contain sulfide which is toxic to various organisms inhibiting the top layer of soils including plants (Olson, 2012). Sulfide is a soil phytotoxin; inhibit the growth of plants (Lamers et al., 2013). Due to its toxic effect on soil many plants are disappeared in the tannery area as well as adjacent area where the sulphide containing liquid wastes are discharged.

CONCLUSION

The study revealed that huge amount of solid waste is generated in fleshing operation from the tannery at Hazaribagh, Bangladesh. Fleshings have the negative effect including air, water and land. Tannery has two options i) proper management of the fleshings i.e., collection of fats, proteins for byproduct before disposal to the environment by means of reduction of environmental impact and ii) by omitting the fleshing operation in leather production. The second one is not possible because it is an essential operation for manufacturing of good quality leather. Leather engineers, researchers, tannery authority should optimize the proper management of fleshings for environmental friendly leather production.

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ID: EE 011

STUDY ON ENVIRONMENTAL IMPACT ASSESSMENT (EIA) OF PROPOSED LEBUKHALI BRIDGE CONSTRUCTION PROJECT IN PATUAKHALI, BANGLADESH

Md. Shafiqul Islam[1]*, Tania Yeasmin[1] & Md. Shamsuzzoha[2]

[1] *Department of Geography and Environment, Dhaka University, Dhaka-1000, Bangladesh.*
[2] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <islamshafiquldm@gmail.com>*

ABSTRACT

This study focuses the possible impact assessment on environment of proposed Lebukhali Bridge construction project which area is located in southern coastal part of Bangladesh. The main purposes of the study are to identify and analyze the potential environmental impacts, suitable alternatives and preparing environmental mitigation plan of this project. Both primary and secondary data were collected in doing this research. Primary data, such as the opinion from Union parishad offices, local NGOs, local people has been collected through in depth interview. Secondary data, such as statistics and reports on the parameter of environmental impacts, equation of Environmental Impact Value (EIV), procedure of Environmental Management Plan (EMP), was collected from past study reports, books and journals etc. The major environmental impact would be air pollution, water pollution and waste siltation, river erosion, migration, loss of Agricultural land etc. Environmental impact assessment is designed on the basis of ecological, physico-chemical and human interest. Environmental management plan prepared to minimize and control of negative impacts during pre-construction, construction and operation/management stages for its sustainability. The environmental impact value was estimated +2 (Positive two) shows the acceptance of this project. This bridge will help to mitigate the transportation problem as well as increase the socio-economic development of southern coastal region of Bangladesh.

Keywords: Environmental Impact Assessment (EIA), EIV, Mitigation and Bridge construction.

INTRODUCTION

Environmental Impact Assessment is a very crucial part of any construction. It is conducted for identifying the impacts both positive and negative for any construction. EIA helps us to know the bad circumstances at a very early stage. Those will helps to rethink about the project or take precautionary steps earlier. Especially EIA is necessary before construction a bridge because it can pose great threat to the areas near side the river. Government under took the plan for making bridge on Paira river because of poor communication network with the southern coastal part of Bangladesh. Coastal part is known as a great tourist area especially for Kuakata Sea beach and also attacks tourist attraction. But the river Paira has become a physical barrier for this region (Ahmed et al., 1995). This physical barrier is seen as an impediment to economic development and social unity. The construction of Lebukhali Bridge will establish a permanent link with the southern coastal part of the country. This paper provide Environmental Impact Assessment (EIA) report of Lebukhali Bridge Project to provide potential negative and positive environmental impacts of the project. The Lebukhali Bridge site is well connected with rest of the country by road and water transports. But the transportation system is not developed and it consumes a lot of time. Moreover, road communication is quite slow because of interruptions in ferry Ghats. For this The Government of Bangladesh (GoB) through their nodal agency - Roads and Highways Department (RHD) within Ministry of Communication (MoC) has been planning for construction of the Lebukhali Bridge for a long time. RHD had prepared a Development Project Proposal (DPP) for a 1450.9 m long Dual Carriage way bridge with the main bridge being in Pre-Stressed Concrete (PSC) box girder together with 2.75 km approach roads at an estimated investment cost of BDTK 4,461.002 million (US$ 65M) in November 2010 (Kundu et al., 2011). It is hoped that this project will be proved as one of the successful project of Bangladesh Government. It will permanently solve the transportation problem of southern part with the country. The objectives of this study as follows (a) to identify and analyze the potential environmental impacts and EIV (b) to explore suitable alternatives and (c) preparing environmental mitigation plan of this project.

MATERIALS AND METHODS

Data were collected from primary and secondary sources. Primary data were collected by means of semi-structured questioonaire survey, site observation and key informants interview method. Secondary data were collected from Bangladesh bureau of statistics (BBS), bridge construction authorities, construction manager, project manager, chief engineer of this bridge construction project, Local government and engineering department (LGED) and from relevant articles. Firstly envrionmental parameters were divided into three catagories namely ecological, physico-chemical and human interest. Then the envrionmental impacts were identified according to those catagories and documented each impacts as range from 0 to ±5. Envrionmetal impacts and mitigation measures for this bridge construction project have been identified according to pre-construction, during construction impact and post construction impacts.

Quantification of environmental impact

Changes of environmental parameters consider as (a) Severe (+5 or -5) (b) Higher (+4 or -4) (c) Moderate (+3 or -3) (d) Low (+2 or -2) (e) Very Low (+1 or -1) (f) No change (0)

Method of Assessment

Environmental impact value (EIV) are estimated and calculated by using mentioned below equation as follows.

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

Where, V_i = Relative change of the environmental quality of parameters, W_i= Relative importance or weight or parameter, N = total number of environmental parameters.

Project Location

Lebukhali Bridge is located over Paira River near Patuakhali district. Lebukhali Bridge Project (LBP) at Paira River is situated at 189 km on Dhaka-Mawa-Bhanga-Barisal-Patuakhali Road (N8), which is at 26 km on Barisal-Patuakhali Road. The Latitude is 22°30.07′ N and Longitude is 90°17.77′ E. The main development objective of the project is to establish a permanent road crossing over the Paira River, replacing the existing unreliable and unsafe ferries. This would eradicate an important hindrance in the development of southern coastal region of Bangladesh. The implementation of the project will require the acquisition of 74 ha of land. Almost half of the land to be acquired is required for the construction of river training works. Over 10 hectors are estimated to be needed temporarily for the construction operations (Kundu et al., 2011).

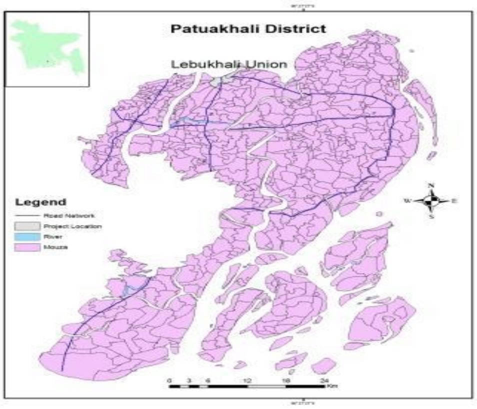


Figure 1: Google Earth Location of Proposed Lebukhali Bridge (Left) and Location of project in Patuakhali District in GIS Map (Right)

RESULTS AND DISCUSSIONS

Environmental impacts and its value estimation

Table 1: Probable environmental impact and calculation of Environmental impact value

Environmental Parameters			RIV*	DoI**	Individual EIV***
I	Ecological Parameters	Loss of Fish Habitat	35	-2	-70
		Loss of Vegetation	21	0	0
		Ecosystem Destruction	29	-1	-29
		Plantation	15	+1	+15
Total Ecological Parameter Value					**-84**
II	Physico-Chemical Parameters	Water Logging	14	-1	-14
		Erosion and Siltation	20	0	0
		Regional Hydrology Change	6	0	0
		Water Pollution	18	-1	-18
		Sound Pollution	30	-2	-60
		River Excavation	12	+1	+12
Total Physico-Chemical Parameter Value					**-80**
III	Human Interest Parameters	Land Use Change	5	-1	-5
		Loss of Agricultural Land	13	-1	-13
		Flood Protection	7	+2	+14
		Road Communication	30	+3	+90
		Employment Opportunity	20	+3	+60
		Migration	10	-1	-10

		Economic Development	15	+2	+30
Total Human Interest Parameter Value					**+166**
Total Environmental Impact Value: $EIV = \sum_{i=1}^{n} (Vi)Wi$ **= (-84-80+166)= +2**					

Source: Based on expert opinion and filed survey, 2013

Note: RIV* is relative impact value, DoI** is degree of impact and EIV*** is environmental impact value

The EIV represents that the project is environmentally acceptable and it gives the clearance to go ahead with the project. On the basis of this result we can say that if we can take precautionary measures to eliminate the negative impacts of this project then this project will be succeeded and people get benefits of the project.

Mitigation Measures

For reducing negative impacts in this project, mitigation measures are considered in three phase's namely pre-construction stage, construction stage and operation & maintenance stage.

Table 2: Possible Mitigation Measures

Project Phase	**Potential Environmental Impacts**	**Proposed Mitigation Measures**
Pre-Construction Stage	Land Acquisition for project	Compensation according to resettlement action Plans
	Loss of vegetation coverage	Enhancing tree plantation programme
	Loss of 45 ha of agricultural land and crops	Agricultural development plan Compensation for loss of land, crops and stock families
Construction Stage	Loss of fish ponds and wetlands	Full compensation for loss of fish Construction of fish sanctuary
	Impacts from land and river transport of materials	Construct and maintain temporary road bypasses
	Water pollution	Create the alteration way to pass waste water
	Sound pollution	Using environmental friendly technology
Operation and Maintenance Stage	Dredging and Maintenance	Project authority should properly maintain
	Oil spills from bridge	Oil gutters
	Waterlogging because of raised alignment	Adequate drainage systems (underpasses, gutters)

Environmental management Plan

A suitable number of environmental management plans is set up for getting sustainable outcome of this bridge construction project. Those plans comprises (a) Sustainable waste management plan (b) Dust management by water sprayer/watering (c) Water management plan (d) Hazardous materials and management plan (e) Top Soil Stripping, storage and reuse (f) Bioderversity management plan (g) Community environment management plan (h) Land acquisition and resettlement (i) Provision of cross drains (j) Construction camp and yard facilities (k) Engagement of environmental specialists

Compensation and enhancement

Compensation and enhancement for this project includes (a) Paira (river) protected sanctuary (b) Tree plantation measures (c) Public health and occupational safety action plan (d) Income and livelihood restoration plan (e) Development of resettlement plan (f) Safeguard alternative income for displaced people

Economic Analysis

Economic analysis of the project is based on a relative study scrutinizing what would arise with and without the project at the local, regional, and national levels, without numerous effects.

Table 3: Scheduling of the Action plan

Scheduling and Reporting												
Actions	Year-1				Year-2				Year-3			
	Q_1	Q_2	Q_3	Q_4	Q_1	Q_2	Q_3	Q_4	Q_1	Q_2	Q_3	Q_4
Mitigation Measures												
Resettlement action plan												
Fish Sanctuary												
Tree Plantation Program												
Capital and Maintenance Dredging												
Installation of Outlets												
Awareness Raising												
Agriculture development measures												
Monitoring												
Pollution (water, sound, air)												
Siltation and erosion												
Bridge Monitoring												

Note: Q_1 is first quarter, Q_2 is second quarter, Q_3 is third quarter and Q_4 is fourth quarter in a year. (Here, one quarter refers the four month in a year like Q_1)

Alternatives

Beforehand the choice of the planned project, some alternatives were considered and these comprise;

Construction of a cable stayed bridge in the same site

Cable stayed bridge decrease span number. It is well for room of river and its flow.

Improving the existing ferry service (Number of ferry and Ghat)**

At present two ferry (one ferry is active and another one have technical problem) service existing there. It can be consider the alternative suggestion by improving ferry service instead of bridge.

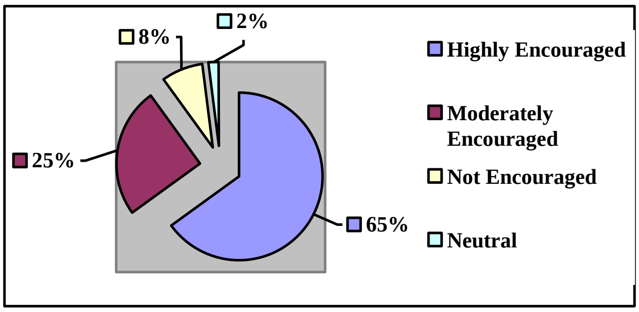
Figure 2: Existing ferry service of the project location

Public Consultation

During the prefeasibility and feasibility study a series of consultations were organized and these were stepped up during detailed design. Public consultation can be involves regarding this project work encompasses as (a) Key Government and Union Stakeholders (b) Consultations with NGOs and CBOs (c) Village-and/or Ward-Level Consultations (d) consultation with expert groups (e) Individual and Group Discussions including FGD. The prime objective of public consultation is to share information on the project and to get feedback on significant social and environmental issues to be considered in the social assessment and environmental impact assessment.

Public Acceptance

In spite of having some negative impacts of this project, people of the project area including associated areas, vehicles drivers, passengers, and small traders are very much positive to get a bridge on this location. Besides that some land owners, small traders within this location, a little number of fishermen shows reversed opinion for constructing this bridge.



Source: Author calculation based on field survey 2014 (Total Respondents N=50)

Figure 3: Public Acceptance of the project

CONCLUSION

An attempt has been taken from this study to identify the environmental impacts related to Lebukhali Bridge project. The major negative impacts are loss of fish habitat, ecosystem destruction, sound pollution, erosion and siltation, loss of agricultural land, migration etc. Some of the impacts are permanent and some can easily handle by taking measures during construction stage and operational stage. It should be remembered that the expected benefits resulting from this project by far outweigh the negative impacts which has led to realization of Lebukhali Bridge. The major positive impact of this project includes plantation, road communication, employment opportunity, economic development etc. Those positive impacts are added the value of the acceptance of this project. Also the Environmental Impact Value (EIV) is +2 (positive two) which gives the clearance of this project to be started. From this assessment it can be declared that the project may be treated as the development pile for the coastal region of Bangladesh inspite of some negative impacts that can be easily reduced by taking proper environmental management plan and planned mitigation measures.

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RELIABILITY ANALYSIS OF URBAN RAINWATER HARVESTING: A CASE STUDY OF DHAKA CITY

Md. Rezaul Karim[1], Md. Zobair Ibne Bashar[1*] & Khondokar Kamrujjaman Nabil[1]

[1] *Department of Civil and Environmental Engineering, Islamic University of Technology, Board Bazar, Gazipur-1704, Bangladesh.*
**Corresponding Author, <shukh@iut-dhaka.edu>*

ABSTRACT

Non-conventional water resources must be introduced to partially offset the increasing water demand. This paper investigates the applicability and reliability of rainwater harvesting (RWH) systems to meet the daily water demand in the multistoried residential buildings in combination with the conventional water supply systems. With the aim of developing a comprehensive tool to assess the reliability of the RWH systems a daily water balance model was developed considering daily rainfall, catchment area, runoff losses and tank volume. Three different climatic conditions i.e. wet, average and dry years were chosen by analysing historical 20-year daily rainfall data. Typical residential buildings of plot size 2.5-5 katha (168 to 335 m^2) with an average of 5 residents in each apartment were considered for the study. According to the results about 15-25% reliability can be achieved under wet climatic conditions and for catchment sizes varying from 140- 200 m^2 a volume of 250-550 kL of rainwater can be harvested each year. Several reliability curves have been presented for two different scenarios under three climatic conditions which indicate that reliability does not increase significantly beyond the tank volume of 30 m^3 and it decreases with the increasing water demand.

Keywords: Rainwater harvesting, reliability, water balance model, water supply, Dhaka

INTRODUCTION

Rainwater harvesting (RWH) is a decentralized practice that provides both water supply and runoff reduction benefits that are often difficult to assess. This technique is relevant in areas with sufficient rainfall for collection but experiencing water shortage due to either limited availability of conventional water resources or due to high water demand. The government of People's Republic of Bangladesh is planning on amending the Building Construction Rules 2008 to incorporate the system of rainwater harvesting mandatory for all new houses in Dhaka Metropolitan area. According to the new proposed BNBC, every building proposed for constructing on plots having extent of 300 square meter or above shall have the facilities for conserving and harvesting rainwater. But a very few or insignificant number of studies have been carried out to investigate the applicability, reliability and efficiency of the rainwater harvesting system in Dhaka city.

Rooftop rainwater harvesting has received an increased attention as a potential alternative water supply source both in the coastal and arsenic affected rural areas of Bangladesh. Karim et al. (2013) investigated the reliability of household based rainwater harvesting in the coastal areas of Bangladesh and found that the maximum reliability that can be achieved under average climatic condition varies from 70% to 90% and the reliability does not increase significantly beyond the tank volume 3000 L. This study also revealed that significant amount of water is lost as spilled water for smaller tank sizes and the potential uses of rainwater can be increased by capturing this huge spilled water if a proper combination of storage tank capacity, catchment (roof) area and rainwater demand is used. A study by Rahman et al. (2014) to assess the sustainability of RWH system for Dhaka city found that 11% water can be saved annually in a building having a roof area of 1850 sft and this volume of rainwater can serve a building with 60 people for about 1.5 months in a year without the help of traditional water supply. This study also showed that with an 1850 sft catchment area, about 69,026 gallons of water can be harvested over one year. This volume of water can reduce the potable water demand and approximately 100 kWh electricity can also be saved as well if rainwater capturing system is introduced in the residential buildings of the city. Optimization of the rainwater tank size is the most widely studied topic of RWH. Notable researchers have conducted on the relationship between rainwater tank sizing and water savings. Imteaz et al. (2011a) developed a daily water balance model to assess the reliability of the rainwater tanks in Melbourne. This study found 100% reliability for a two-people household scenario with a roof size of 150-300 m^2 having a tank size of 5,000-10,000 L but for a four-people household scenario it is not possible to achieve 100% reliability even with a roof size of 300 m^2 and a tank size of 10,000L. Ghisi et al. (2007) investigated the water savings potential from rainwater harvesting systems of Brazil and found the average potential for potable water savings of 12-79% per year for the cities analysed. A good number of software and simulation based models were also developed to help assessing the reliability and tank size of RWH system. eTank is a decision support tool for optimizing rainwater tank size, developed by Imteaz et al. (2011b). To evaluate both the water supply and runoff reduction benefits Sample et al. (2013) developed a model that simulates a single RWH system in Richmond, Virginia, using storage volume, roof area, irrigated area, an indoor non-potable demand and a storage dewatering goals independent design variables.

This study focuses on the reliability of RWH in the multistoried residential buildings of Dhaka city. This study examines how much reliable rainwater is in terms of yearly volume and the number of day’s rainwater is able to completely satisfy the daily water demand. The study also shows the economic savings that can be achieved if rainwater harvesting system is integrated with the conventional water supply. Despite positive outcome from many studies, there remains a global community reluctance to adopt rainwater harvesting on a wider scale. Part of the reason for this reluctance can be attributed to lack of information about the effectiveness of a rainwater harvesting system and the optimum storage size required to satisfy the performance requirements under the specific site conditions (Imteaz et al., 2011a). The research presented in this paper is undertaken in the light of current knowledge gaps to access the economic benefits of a RWH system in Dhaka to provide guidance to water authorities to enhance the acceptance of a RWH system.

METHODOLOGY

Considering daily rainfall, contributing catchment (roof) area, losses due leakage, spillage and evaporation and storage (tank) volume a daily water balance model was developed. The prime input value in this model was the daily rainfall amount for three different years (Dry, Average and Wet year). The daily runoff volume was calculated by multiplying the rainfall amount with the contributing roof area and runoff coefficient. For this study the runoff coefficient was considered as 0.9 to account for several losses (leakage, spilling and evaporation).

Generated runoff was diverted to the existing underground storage tank. This model uses the existing underground storage tank as the rainwater storage tank. So, no construction of additional rainwater storage tank is required which eventually leads to economy. Available storage capacity was compared with the accumulated daily runoff. If accumulated runoff is greater than available storage volume, excess (spilled) water was deducted from the accumulated runoff. Amount of water use(s) is deducted from the daily accumulated/stored runoff amount, if sufficient amount of water is available in the storage. In a situation, when sufficient amount of water is not available in the storage, model will assume that the remaining water demand is supplied from the conventional/town water supply.

The model calculates daily rainwater use, daily water storage in the tank, daily spilled water volume and daily town/supply water use. In addition, the model calculates accumulated annual rainwater use, accumulated annual spilled water volume and accumulated annual town water use. The water balance equation for the study is given by:

$$S_t = V_t + S_{t-1} – D \quad (1)$$

$$S_t = 0, \text{ when } S_t < 0 \quad (2)$$

$S_t = C$, when $S_t > C$ (3)

Where, S_t is the cumulative water stored in the tank (L) after the end of the t^{th} day, V_t is the harvested rainwater (L) on t^{th} day, S_{t-1} is the storage in the tank (L) at the beginning of t^{th} day, D is the daily water demand (L) and C is the capacity of the underground tank (L). The volume of spilled water and town water used equation is given by:

$SW = V_t + S_{t-1} - D - C$ for $V_t + S_{t-1} - D > C$ (4)

$TW = D - V_t - S_{t-1}$ for $V_t + S_{t-1} < D$ (5)

In this study two types of reliability was calculated. Time based reliability is calculated using the following equation:

$$R_{e(t)} = \frac{N-U}{N} \times 100 \qquad (6)$$

Where, $R_{e\,(t)}$ is the reliability of the tank to be able to supply intended demand (%), U is the number of days in a year the tank was unable to meet the demand and N is the total number of days in a particular year.

Volumetric reliability is given by:

$$R_{e(v)} = \frac{\text{Volume of rainwater supplied in a year}}{\text{Total demand in a year}} \times 100 \qquad (7)$$

DATA

In this study, 20 year (1988-2007) rainfall data was considered and all the calculations were performed for three different climatic conditions (i.e. wet, average and dry years). Years corresponding to maximum and minimum annual rainfall were considered wet year and dry year respectively. The average year was considered the year having annual rainfall close to the average annual rainfall over 20 years' period. Selected years and corresponding annual rainfall amounts are shown in Table 1.

Table 1. Annual rainfall for Dry, Average and Wet year

Year	Annual rainfall (mm)
Dry year (1992)	1169
Average year (1990)	2103
Wet year (2007)	2885

As this study is concerned with the rainwater harvesting in the multistoried residential buildings of Dhaka city having a plot size 2.5 to 5 katha i.e. 1800 to 3600 sft., the catchment area was considered as 60-70% of the plot size. The reliability calculations were performed for the catchment sizes ranging from 1080 sft to 2160 sft and tank sizes varying from 30 to 50 m^3. A daily water demand ranging from 120 lpcd to 180 lpcd has been assumed based on the estimations of Ahmed and Rahman (2000), Bangladesh water utilities data book (2006-2007), BNBC and DWASA.

Based on the daily water balance model a software was developed using Visual Studio that calculates the reliability and at the same time it also shows the economic savings. The software requires catchment size, water demand, percentage of runoff loss and tank capacity as input data.

RESULTS AND DISCUSSIONS

Reliability curves were developed under three different climatic conditions (wet, average and dry year) for two different scenarios. For a plot size of 5 katha (3600 sft) a catchment area of 2160 sft and for a 2.5 katha (1800 sft) a catchment area of 1512 sft has been considered. It has been assumed that 50 people live in a 6 storied residential building having a plot size of 3600 sft and and 30 people live in a 6 storied residential building having a plot size of 1800 sft.

Figure 1 shows the reliability relationships with the tank volume for different catchment sizes. For both the cases reliability increases up to a tank volume of 30 m^3 for the wet year and beyond that reliability does not increase with the increasing tank volume. But in case of average and dry conditions tank size have no significant impact on reliability. It also shows that the reliability for a 1512 sft catchment area is greater by about 3-4% than the reliability of a 2160 sft catchment area. Although for both the cases reliability is insignificant for the dry climatic condition.

Volumetric reliability i.e. the percentage of water savings with varying tank volume for two different scenarios have been shown in fig. 2. The effect of tank size on the volumetric reliability is same as the time based reliability. It can be observed that the value of volumetric reliability is higher than the time base reliability for the same scenarios. Where the time based reliability varies from 3-17% for a catchment size of 1512 sft, volumetric reliability varies from about 10-24% for the same catchment size. This is primarily because of the water demand considered for this study includes domestic water use such as drinking, toilet, kitchen sink, bath, washing machine, wash basin, outside use, shower, dishwasher, leaks etc. which is very high for a city like Dhaka in compared to the other areas of Bangladesh. Whereas, the number of days when the rainfall runoff exceeded this demand is less.

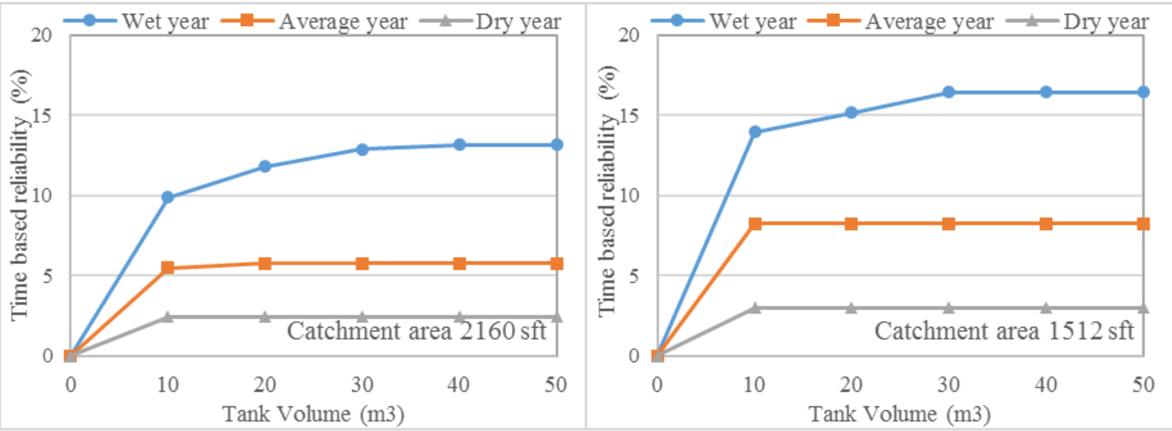

Figure 1. Reliability curves for a building with a water demand of 140 lpcd for different catchment sizes.

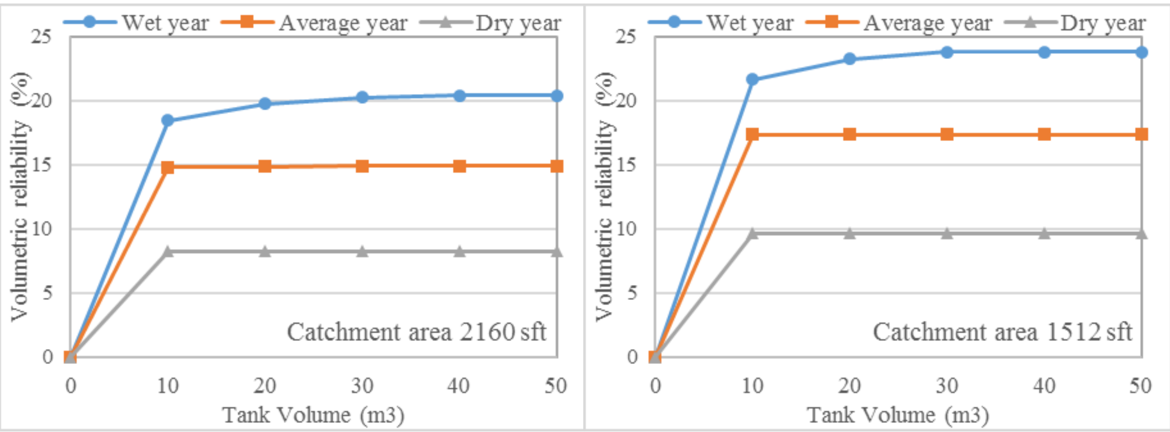

Figure 2. Volumetric reliability for a building with a water demand of 140 lpcd for different catchment sizes.

Figure 3 shows the reliability curves with varying daily water demand for two different scenarios. For both the cases reliability decreases significantly with the increasing water demand. Reliability decreases by about 5-10% for an increase in water demand of about 60 lpcd, which signifies that the rainwater harvesting system becomes inefficient with a high water demand.

Water and economic savings for a sample scenario is shown in Table 2. The water price considered for this study is BDT 8 per kL (DWASA water price). This monetary saving only includes the water price, not the savings associated with energy consumption reduction and other savings.

Table 2. Water & Monetary savings form a RWH system for a catchment of 2160 sft & tank volume of 40 m^3

Climatic condition	Water savings (kL)	Economic savings (BDT)
Wet year	521	4170

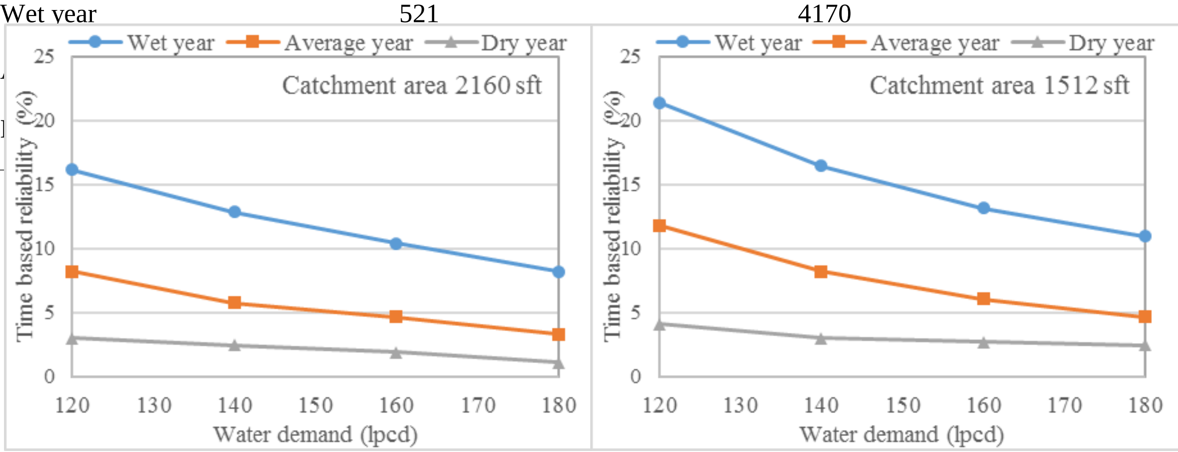

Figure 3. Reliability for a building with a tank volume of 30 m^3 for different catchment areas.

CONCLUSIONS

This research investigated the reliability and water savings for the multistoried residential buildings of Dhaka city under three different climatic conditions. The study revealed that the current underground tank sizes of the residential buildings of the city are sufficient to prevent the overflow of the harvested rainwater and about 250-550 kL of rainwater can be harvested per year from a catchment area of 2160 sft if RWH system is combined with the conventional water supply.

For a reliable and sustainable rainwater harvesting system, reliability analysis should be performed to find out the optimum tank volume. Tank capacity is the key factor to maximize the rainwater storage. There are numerous optimum solutions with different combinations of storage volumes, roof sizes and rainwater demand. The proposed model can be used to predict the reliability and water savings with reasonable accuracy.

The present analysis indicates that a maximum of 25% reliability can be achieved if sufficient rainfall is available throughout the year. Though the reliability and the economic savings found is not much greater, it will have a significant impact if rainwater harvesting system is practised at a larger scale as it will reduce a certain portion of the pressure from the conventional water supply and it will also play a significant role in alleviating the water clogging problem.

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APPLICATION OF FLOATING TREATMENT WETLANDS FOR LAKE WATER TREATMENT IN BANGLADESH

T. Sarker[1*], M.Rahman[2] & A.I.Khan[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka-1208, Bangladesh.*
[2] *Department of Water Resource Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
**Corresponding Author <tannisarker92@gmail.com>*

ABSTRACT

This paper reports on the pollutant removal efficiencies of constructed floating wetlands in treating polluted lake water. A rectangular baffled steel tank was used to replicate a lake on a pilot scale. A floating mat was used as a media for the growth of microbial community. The system used for this purpose was a rectangular airtight PVC platform on which a mat was set up where a variety of plants was allowed to grow while their roots were submerged in water. The mat was constructed using locally available materials. Polluted lake water from Gulshan Lake in Dhaka was dosed into the tank containing floating wetland. Pollutant concentrations in influent lake water and subsequent effluent from the system were tested over a period of 8 weeks, in two phases of 4 weeks. The parameters tested were pH, Eh, Biochemical oxygen demand (BOD_5), *Escherichia coli* and turbidity. The results of this study provided evidence supporting the effectiveness of floating treatment wetlands as a low cost, energy efficient technology for polishing polluted open water bodies in Bangladesh.

Keywords: constructed wetland, surface flow wetland, media, pollutant removal rate, efficiency

INTRODUCTION

The contents of polluted lake water vary from time to time depending on a variety of factors ranging from the level of influent loading to local average weather. Some common contaminants of lake water in are suspended solids, biodegradable organic, pathogens, nutrients, refractory organics, heavy metals, dissolved inorganics, pharmaceuticals, radiological, toxins, emulsions, macro solids, animals, and gases. In Dhaka, massive amount of these contaminants are found in its' lakes, which can be attributed to indiscriminate pollution of the environment and a lack of implication of environmental protection laws.

Constructed wetlands are engineered wetlands that have saturated or unsaturated substrates, emergent/floating/submergent vegetation, and a large variety of microbial communities that are purposely built for pollution control. (Saeed and Sun, 2012)The objectives of this research were to investigate the effects of Floating Treatment Wetland and Surface Flow wetland on the environmental parameters (pH and Redox Potential) of and to investigate the performance of Floating Treatment Wetland and Surface Flow wetland in the removal of BOD, *Escherichia Coli* and Turbidity from lake water.

STUDY AREA

Gulshan Lake is the northernmost lake in a chain of water bodies in Dhaka, suffering from highly significant pollution. Gulshan Lake with an area of about 100 ha and is located at 23º48' N and 90º25' E of Dhaka city. The length of the lake is 3.8km which covers an area of 0.0160 km2. It has an average depth of 2.5 m and a volume of 12 ×105 m^3. The lake has inlets through which it is connected with some old river channel and is, therefore, affected by flood water during peak flooding seasons. Many drains and gullies discharge into the lake. Previous study revealed that among the heavy metals only Pb concentration exceeded the standard level during the monsoon, otherwise concentrations of all other four heavy metals (Cd, Cr, Cu and Ni) exceeded the standard level of drinking, fishing and surface water as setup by WHO, GOB, USEPA, DOE and FWPCA, for the summer period.

METHODOLOGY

CONFIGURATION OF THE PILOT SCALE WETLAND SYSTEM

A rectangular baffled steel tank was used to replicate a lake on a pilot scale. A floating mat was used as a media for the growth of microbial community. The system used for this purpose was a rectangular airtight PVC platform on which a mat was set up where a variety of plants was allowed to grow while their roots were submerged in water. The surface of the water was divided into three sections; mid-section containing the floating wetland. Effluent from this tank entered a matured, naturally formed wetland system in a plastic tank. In the first part, emergent plants (primarily Oriental Sword) were planted. It was constructed in such a way, so as to facilitate the growth of roots underwater. The natural wetland part was previously constructed wetlands which had grown over time in a plastic tank. The tanks were connected using water valves and plastic pipes. Flow could be controlled using the water valves fitted into both tanks.

CONFIGURATION OF THE STEEL TANK

The steel tank is of rectangular shape of dimensions length 2.5m, width 1m and height 1.1m. It has 3 internal baffles which are spaced at 0.61m c/c. It also consists of 4 openings which have been equipped with water valves; the upper two valves have been used for dosing and sample collection purposes. The FTW was placed in between the two farthest baffles. The steel tank was filled up to a depth of 0.2m with a mixture of sand, gravel and clay.

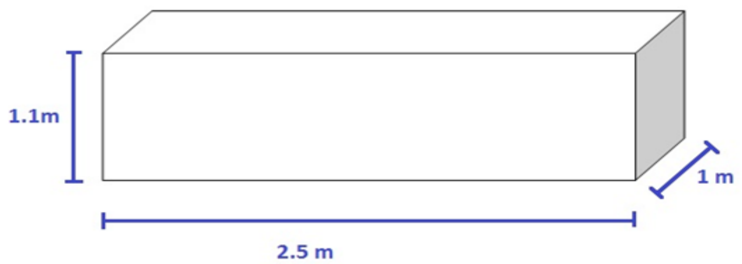


Fig1. Dimensions of the steel tank containing the floating mat system

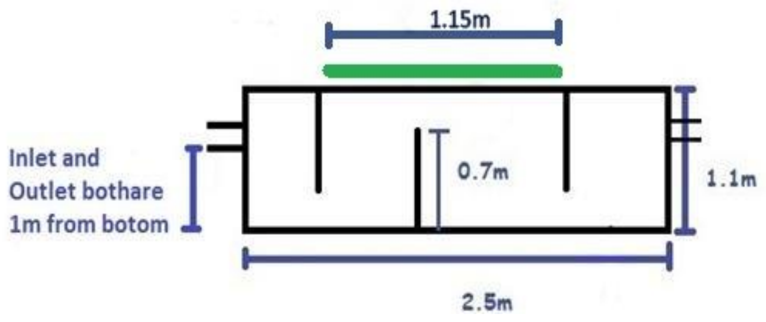


Fig2.Internal configuration of the steel tank containing the floating mat system.

CONFIGURATION OF THE FLOATING MAT

The floating mat's primary structural element is an infrastructure of four hi-performance PVC pipes of 4 inch diameter, interconnected with four PVC angles and made airtight with industrial strength plastic adhesive. The dimensions of the mat were of length 1.15m and of width 0.85m. The mat was constructed using nylon nets and ropes, jute ropes, dried grass and galvanized iron wires, all of these integrated to form a bed to support the weight of soil and plants.

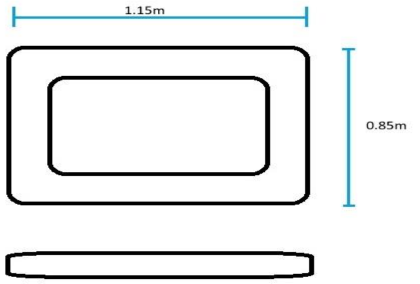
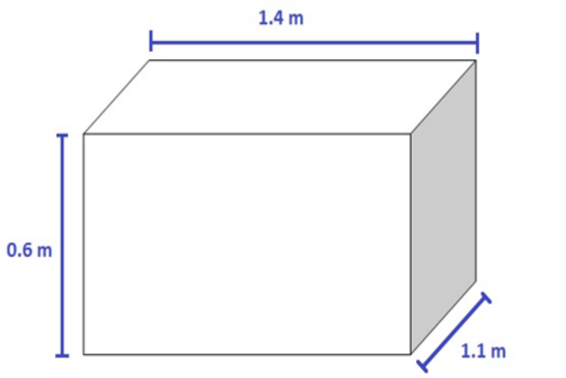


Fig3. Dimensions of the floating mat containing the FTW system

Fig4.Initially constructed PVC platform for infrastructural and buoyant support

CONFIGURATION OF THE PLASTIC TANK

The plastic tank containing the SF system had the dimensions of length 1.4m, width 1.1m and height of 0.6m. A mixture of sand, gravel and clay were used to fill it up to 2.5m. This soil served as the base on which the various types of plants can grow. A matt of depth 0.12 m was set up to act as a media. A vivid visual of the internal configuration is shown in figure 7



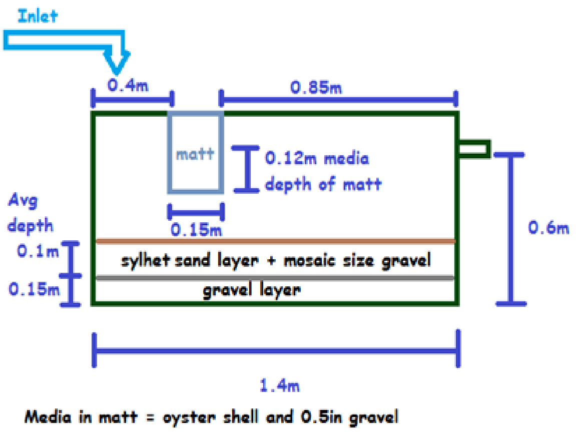


Fig 7- Dimensions of the plastic tank tank containing the natural wetland system

Fig 8- Internal configuration of the plastic Containing the natural wetland

Fig 9.Plastic tank containing the natural wetland

Fig 10.floating mat section after maturing.

PLANTS

Plants used in the system were collected from local water bodies and nurseries. Plants that survived till the end of phase 2 are ***Dracaena sanderiana***(Lucky Bamboo),***Lamnaceae***(Duckweeds),***Phragmites Austalis***(Phragmite)**,*Salvininia*(**salvinia**),*Cyperus alternifolius, Hydro cotyleranunculoides***(pennywort), ***Pisitastratiotes***(Water lettuce).

HYDRAULIC LOADING

Raw lake water collected from Gulshan Lake in Dhaka was dosed into the system in two phases. The 1st phase consisted of 4 weeks. For four days per week, 50 liters was dosed into the tank containing FTW and 12 liters was dosed separately into the natural wetland system. A period of 30 days water-lock was then observed before the start of the 2nd phase. In the 2nd phase, 75 liters of raw lake water was dosed directly into the tank containing floating wetland, 4 days per week for 4 weeks.

ARRANGEMENT AND SAMPLING

After dosing in the wetland system, the effluent was collected in a sterilized container from the outlet of both the steel tank and the plastic tank. The flow of water was regulated using manual valves. The collected samples were tested for the following parameters: pH, oxidation reduction potential, biochemical oxygen demand (BOD_5), *Escherichia coli* and turbidity. Dosing, subsequent sample collection and testing were conducted in two phases of 4 weeks each. A water lock period of 30 days was observed between week 4 and week 5. In phase 1, the FTW and SF system were separately dosed and did not have any connection between them. In phase 2, the FTW and SF system were connected together.

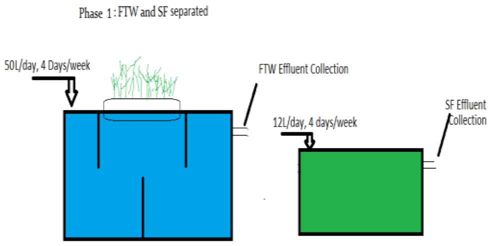
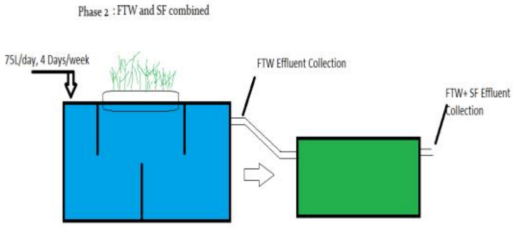


Fig11. Arrangement of the two Systems in phase 1(week 1-4)

Fig 12-.Arrangement of the systems in phase 2(week 5-8) two



TABLES

The removal performances of the FTW and SF systems have been summarized in the series of tables below. In phase 1, the SF and FTW systems were separated, independently dosed. In phase 2 both the systems were combined to achieve better performances.

Table 1- Overall removal performance for FTW in the first phase (Dosing: 50l/day, 4 days/week) 4 weeks Table 2- Overall removal performance for SF in the first phase. (Dosing: 50l/day, 4 days/week) 4weeks)

Table 1					Table 2				
Parameter	Unit	Raw Water	Effluent of Floating Wetland	Percent removal	Parameter	Unit	Raw Water	Effluent of SF Wetland	Percent removal
pH	-	7.16	7.01	-	pH	-	7.16	7.27	-
Eh	mV	116.0	145.75	-	Eh	mV	116.0	157.25	-
BOD	mg/L	195.0	44.0	77.43%	BOD	mg/L	195.0	21.75	88.84%
E Coli	CFU/100 ml	12427.5	3025	75.66%	E Coli	CFU/100ml	12427.5	1375	88.94%
Turbidity	FTU	53.69	11.78	78.05%	Turbidity	FTU	53.69	0.17	99.69%

Table 3- Overall removal performance for FTW in the first phase (Dosing: 75L, 4 days/week, 4 weeks)

Parameter	Unit	Raw Water	Effluent of Floating Wetland	Percent removal	Effluent of Surface Flow Wetland	Percent removal
pH	-	6.83	6.55	-	6.99	-
Eh	mV	111.25	170.75	-	145.5	-
BOD	mg/L	140	26.75	80.89%	28.5	79.64%
E Coli	CFU/100ml	13918.75	1075	92.28%	600	95.69%
Turbidity	FTU	116.26	6.34	94.55%	0	100%

RESULTS & DISCUSSION

Results have been represented in the tables 1, 2& 3. The Floating Treatment Wetland and SF were able to achieve impressive removal rates of BOD, Escherichia coli and Turbidity individually in both phase 1 and 2. They also demonstrated their capability of maintaining the pH of water within desirable range, and increasing the average ORP of lake water.

CONCLUSION

Overall the results of this study demonstrates the possibility of Floating Treatment Wetlands as a low cost, energy efficient and waste water treatment technology for polishing polluted open water bodies in Bangladesh. Further research on this type of wetland system is required over a longer testing period and extended range of parameters including Nitrogen, COD and DO.

ACKNOWLEDGEMENT

We would like to express our heartfelt gratitude towards our honorable supervisor Dr. Tanveer Ferdous Saeed, Assistant Professor, Department of Civil Engineering, Ahsanullah University of Science & Technology, for his valuable suggestions, instructions, encouragement, co-operation and guidance to culminate this project.

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Thesis

Integrated Floating Treatment and Surface Flow Wetlands for the Treatment of Lake Water in Dhaka City.

ID: EE 015

ARSENIC CONTAMINATION OF GROUNDWATER IN BANGLADESH: A CASE STUDY IN BRAHMANBARIA DISTRICT

S. Pk.[1*], M. A. H. Kafi[1], M. S. Hossain[1] & A. K. chowdhury[1]

[1] *Institute of Water Modelling, Dhaka, Bangladesh.*
**Corresponding Author, <pk_shubhra@yahoo.com>*

ABSTRACT

Arsenic contamination in groundwater in the Padma-Meghna fluvial plain is a major concern in relation to safe drinking water provision in Bangladesh. The most obvious alternative of surface water sources carries the risk of serious microbiological pollution and necessitates treatment facilities often exceeding local financial resources and operational capacity. Department of Public Health Engineering (DPHE), Bangladesh and British Geological Survey (BGS), United Kingdom conducted a study in 1998-1999 and identified 61 district of Bangladesh as arsenic affected. This research has been carried out focusing Brahmanbaria district where 38% groundwater was identified as arsenic contaminated by BGS in 1998. Secondary data have been collected from DPHE and analyzed to know previous arsenic contamination records. To investigate present arsenic contamination status thirty water samples have been collected from various villages of nine upazillas varying tube-wells depth from 15 to 200 meters. The samples are tested at Environmental Engineering laboratory of Bangladesh University of Engineering & Technology (BUET) using Graphite Furnace Atomic Absorption Spectrometry (GFAAS). To compare present and past arsenic contamination records, spatial distribution maps are prepared using Geographical Information System (GIS) showing classified areas for three different arsenic levels. The shallow tube-wells are heavily arsenic affected and the areal extent of contamination becomes almost double in last fifteen years whereas still now deep tube-wells are free from this contamination.

Keywords: Arsenic, Brahmanbaria, BGS, BUET, DPHE, GFAAS, GIS

INTRODUCTION

The practice of drinking groundwater was started in early 1970s and various aid agencies especially UNICEF and WHO spent a substantial amount of funds on sinking tube-wells in Bangladesh with the aim of providing safe drinking water for the country (Mukherjee, 2001). Primarily few number of shallow tube-wells were sunk but present today the number is estimated to be anywhere from 11–18 million (Nahar, 2009). Now a day’s tube-well water reaches 97% of the rural population and constitutes the backbone of the rural water supply in Bangladesh (NGO Forum, 2004). This made a significant contribution in decreasing infant mortality from 151 to 83 per thousand from 1960 to 1996 by reducing water borne epidemics (UNICEF, 1998). Since 1993, the success of the UNICEF sponsored tube-well sinking policy has become a matter of anxiety because of the manifestation of chronic arsenic toxicity among the population resulting from prolonged consumption of arsenic contaminated groundwater (Chowdhury et al., 1999) and has emerged as a serious threat to public health in the country (Hossain, 2002). This treat has emerged as a new burden in recent years along with flood and drought in the land of natural disasters (Safiuddin et al., 2011). According to the Department of Public Health Engineering (DPHE), Bangladesh and British Geological Survey (BGS), United Kingdom, 61 districts out of 64 in Bangladesh and approximately 27% of the shallow tube-wells are containing arsenic concentration above the maximum permissible value of Bangladesh drinking water standard (0.05 mg/l) (DPHE, 1999).

Arsenic is not only affected drinking water but also contaminated irrigation wells those are extracting from the shallow aquifer, were installed across the country along with shallow tube-wells for domestic water supply. A total of 924,023 shallow tube-wells and 23,434 deep tube-wells were used for irrigation in Bangladesh during the 2003 dry season, and groundwater irrigation covered about 75% of the total irrigated area (BADC, 2003). These irrigation facilities boosted agricultural production and enabling Bangladesh to become self-sufficient in food even though the population nearly tripled over the last four decades. Dry-season rice called Boro, totally cultivated by irrigation water and produced about 49% of the total rice production in 2003 (MoA, 2004). Thus, the quality of irrigation water should be safe. A study on Arsenic in rice and rice straw in Bangladesh was conducted by S. Chakma and others in 2012 where they found that the concentrations of arsenic in rice and rice straw were 0.235 ± 0.014 mg/l and 1.149 ± 0.119 mg/l respectively (Chakma et al., 2012). Both the values are greater than FAO recommended maximum permissible arsenic concentration in food grains. This arsenic affected food grains may affect human and other animal’s health.

In early 1990s naturally occurring arsenic contaminated water was first detected in Bangladesh. The Department of Public Health Engineering (DPHE) first detected arsenic in the groundwater of Bangladesh in 1993 and the issue came in limelight in 1995 (Fazal et al., 2001; Smith, 1997). British Geological Survey (BGS), School of Environmental Studies (SOES) of Jadavpur University, Dhaka Community Hospital (DCH), Bangladesh University of Engineering & Technology (BUET), many educational institutes and government and nongovernment organizations (NGOs) investigated arsenic contamination problem of Bangladesh (Safiuddin et al., 2011). All these studies reported that, almost all districts of Bangladesh are arsenic affected and a huge population exposed to arsenic contamination. In 1998, British Geological Survey (BGS) collected 2022 water samples from 41 arsenic-affected districts and prepared a list of percentage of tube-wells those arsenic concentrations above 0.05 mg/l (BGS, 1998). In this list, the percentage of arsenic contaminated tube-wells in Brahmanbaria district was 38 which were higher by 3 of average percentage 35 of 41 districts (BGS, 1998). This study first detected Brahmanbaria district as a severe arsenic affected area of Bangladesh. After that Kinniburgh and Kosmus in 2002 and Escobar and others in 2006 conducted study on arsenic contamination in the south and east parts of Bangladesh and reported that, more than 60% tube-wells is severely affected in these areas (Kinniburgh, 2002; Escobar et al., 2006). This paper shows present arsenic contamination status in both shallow and deep tube-wells in Brahmanbaria district and spatial distribution of contamination in shallow and deep tube-wells over the whole study area.

STUDY AREA

This study is carried out at Brahmanbaria district which is located at the east-central region of Bangladesh under Chittagong division and lies between 23°29'/N to 24°16'/N latitudes and 90°39'/E to 91°39'/E longitudes (Fig. 1). It is divided into nine upazillas (sub-district) with a total area of 1927.11 km² (LGED, 2014). Brahmanbaria is bounded by Kishoreganj District and Habiganj District on the north, Comilla District on the south, Habiganj District and Tripura State, India on the east and Meghna River, Kishoreganj District, Narsingdi District and Narayanganj District on the west (LGED, 2014) (Fig. 1). Geologically this area is located in the Meghna Deltaic Plain (MDP) comprising course grained channel-fills deposit and fine grained over-bank deposits but the north eastern part of the study area is generally flat and occurring is relatively higher level than surrounding floodplain which is Chandina Deltaic Plain (CDP) (Bark, 1977). The sediments of the CDP are composed of silt, silty loam, silty clay belonging to the chandina formation. The chandina formation is overlain by the Meghna alluvium and underlain by the Pleistocene Madhupur clay and Pliocene dupi tila formation. The study area is belongs to the aquifer of Late Pleistocene coarse sands, gravels and cobbles of the Titas and Brahmaputra Meghna-fans and basal fan delta gravels along the incised Brahmaputra channel (BGS and DPHE, 2001).

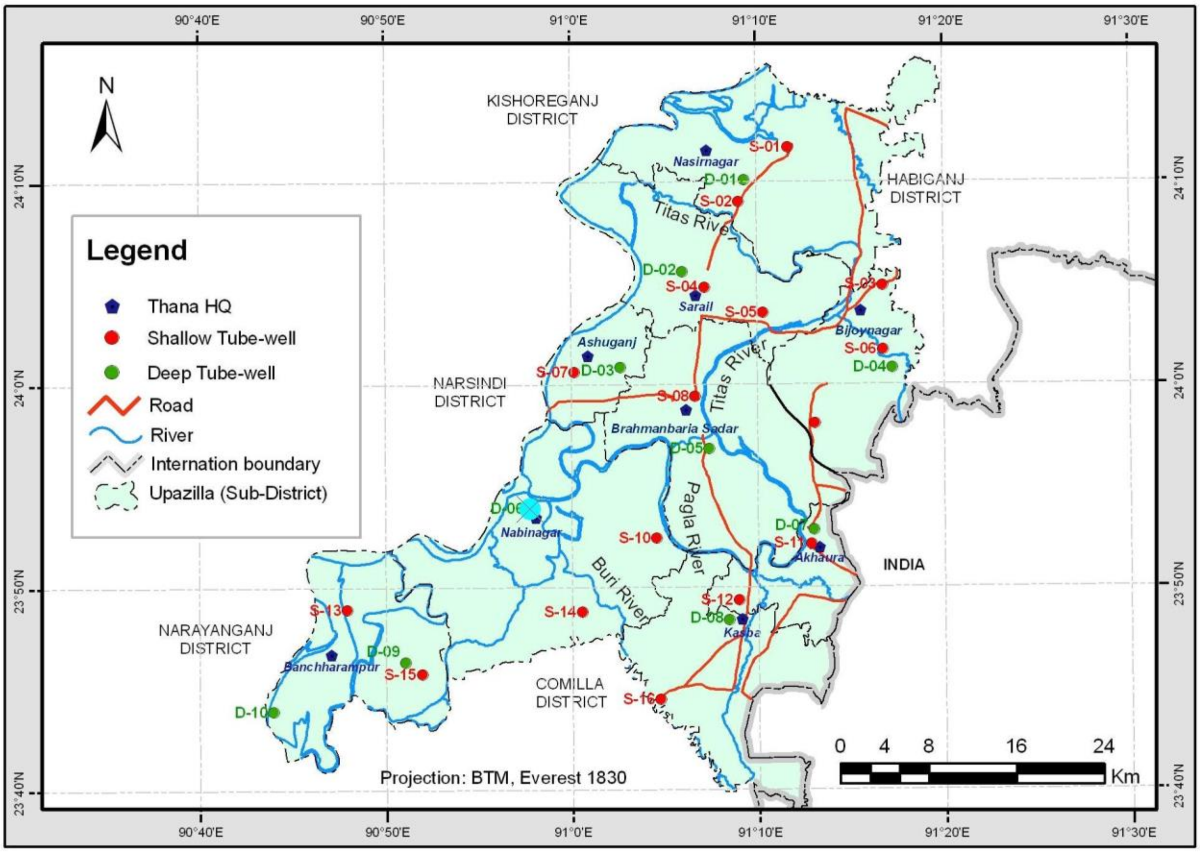

Figure 2 Base map of the study area showing collected sample locations

DATA COLLECTION AND STUDY METHODOLOGY

Secondary data collection and analysis

Department of Public Health Engineering (DPHE) in association with British Geological Survey (BGS) carried out a huge study over 61 districts of Bangladesh in 1998 to 1999 and tested many tube-wells to identify their safety against arsenic contamination (DPHE, 1999). For this study, the data of numbers of total tube-wells and arsenic affected tube-wells in every village of Brahmanbaria district were collected from DPHE, Brahmanbaria, Bangladesh. Then the data were screened according to union and upazilla and finally the percentages of arsenic contaminated tube-wells in every upazilla of Brahmanbaria district were calculated.

Procedure for water sample collection

Thirty water samples were collected from different villages of nine upazillas of Brahmanbaria district during the month of January, 2013 (Fig. 1). Among thirty, twenty samples were collected from shallow tube-wells and rest of all from deep tube-wells. For each sampling location, three samples were collected for chemical analysis in one liter plastic bottle. The samples were collected at running condition of the tube-wells and not before than five minutes from the starting of pumping. The collected water samples were tightly sealed as early as possible to avoid exposure to air and stored in standard temperature. Water sampling techniques were followed as outlined by Hunt and Wilson (1986) and APHA (1989) (Hunt, 1986; APHA, 1989). All the chemical analyses were conducted at the Environmental Engineering Laboratory of Bangladesh University of Engineering & Technology (BUET), Dhaka, Bangladesh.

Method of chemical analysis of the samples

The samples were tested using Graphite Furnace Atomic Absorption Spectrometry (GFAAS) which also known as Electrothermal Atomic Absorption Spectrometry (ETAAS). Test procedure and all necessary precautions were done according to APHA (1989) specifications (APHA, 1989). To show spatial distribution of arsenic contamination over the whole study area two maps have been prepared for shallow tube-well samples and deep tube-well samples using Geographical Information System (GIS) applying Inverse Distance Weightage (IDW) interpolation method.

RESULT AND DISCUSSIONS

Previous Arsenic contamination records

Analyzing secondary data it is observed that, the average percentage of arsenic contaminated tube-wells in Brahmanbaria district was 30 in 1998-1999. The percentages of arsenic contaminated tube-wells in nine upazillas of Brahmanbaria district is shown in Fig. 2. Bancharampur upazilla shows highest percentage of arsenic contamination whereas Akhaura demonstrates the lowest. More than 40 percent tube-wells were arsenic affected in Brahmanbaria Sadar, Nabinagar, Sarail and Bancharampur upazillas whereas the percentages were less than 20 in Akhaura, Nasirnagar, Ashuganj, Bijaynagar and Kasba upazillas.

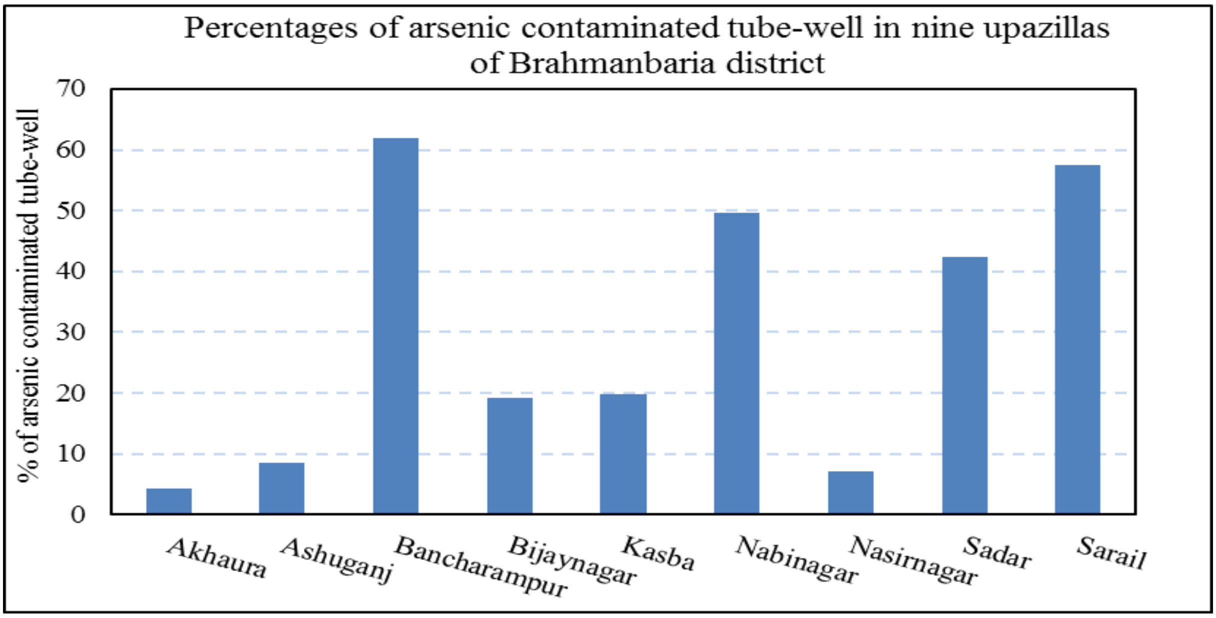

Figure 2 previous arsenic contamination records in nine upazillas of Brahmanbaria district

Present Arsenic contamination status

The spatial distribution maps show the spatial distribution of arsenic contamination over the whole study area for shallow and deep tube-well samples. The spatial distribution is classified into three ranges; the values are lower than WHO recommended highest permissible value (< 0.01 mg/l), the values lie between WHO and Bangladesh standard's maximum permissible values (0.01-0.05) and the values are greater than Bangladesh standard's highest allowable value.

Fig. 3 (a) describes the spatial distribution of arsenic contamination in shallow tube-wells over the whole study area. The map demonstrates the arsenic levels in groundwater of shallow aquifers of Bancharampur, Nabinagar, Ashuganj, Sarail, Bijoynagar and Nasirnagar upazillas overdo allowable value of Bangladesh standard whereas Akhaura upazilla is free from contamination but arsenic level exceeds WHO recommended maximum permissible value. On the other hand maximum area of Brahmanbaria Sadar upazilla is affected by arsenic but in Kasba upazilla opposite scenario is observed.

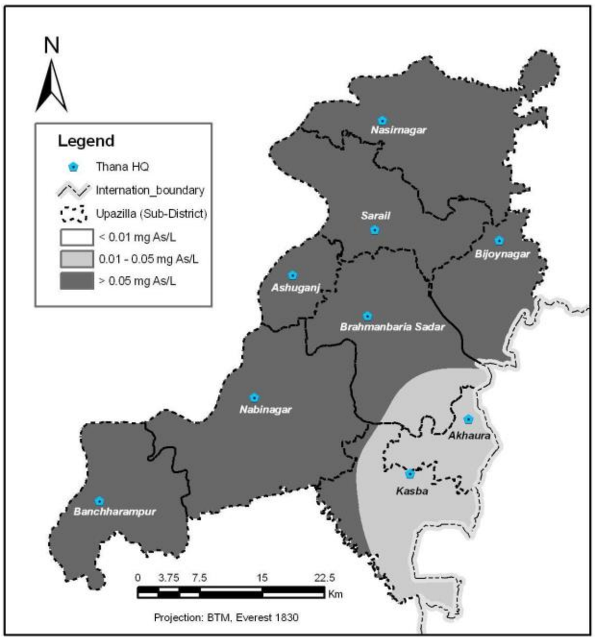

Figure 3 (a) Spatial distribution map for shallow tube-wells

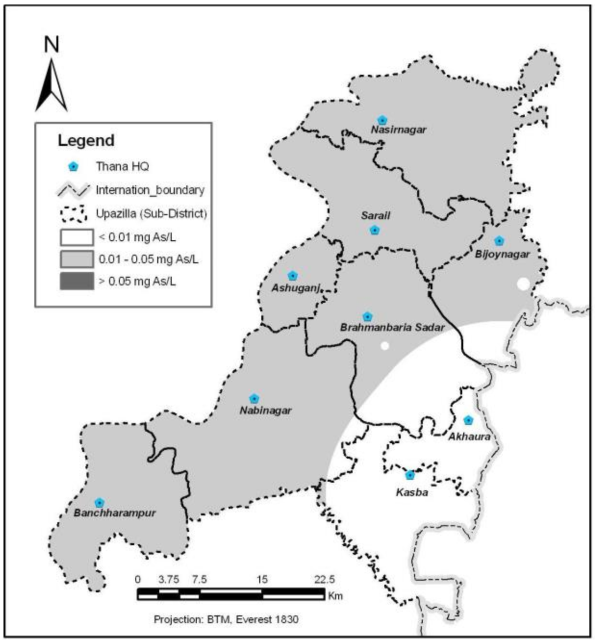

Figure 3 (b) Spatial distribution map for deep tube-wells

Another spatial distribution map for deep tube-wells is shown in Fig. 3 (b). It shows deep tube-wells of Brahmanbaria district are free from arsenic contamination but in most of the areas arsenic levels exceed WHO recommended highest permissible value. Akhaura, Kosba and some areas of Brahmanbaria Sadar, Nabinagar and Bijoynagar upazillas are completely free from arsenic contamination whereas Bancharampur, Ashuganj, Sarail and Nasirnagar upazillas are affected by arsenic but the levels do not exceed national maximum permissible limit yet.

CONCLUSION

Comparing present and previous arsenic contamination status it can be concluded that, the arsenic level in shallow tube-wells increasing day by day and the figure becomes double in fifteen years. Shallow tube-wells are highly affected by arsenic over the whole Brahmanbaria district except Ahkaura, Kasba and some little area of Brahmanbaria sadar upazillas. On the contrary, still now deep tube-wells are free from arsenic contamination. Deep tube-wells can be an alternative source for safe drinking water in this area but how long it will be safe it is a question to all. People have to find an efficient alternative source of drinking water which has no probability of arsenic contamination in future. Surface water especially river water can be the best source of drinking water in this area, because three rivers such as Titas, Pagla and Buri pass over Brahmanbaria district and carrying water all over the year. To use surface water, treatment plants have to install which is costly and not possible to do individually. On the other hand rainwater harvesting can be another option to supply safe drinking water in this area and anyone can practice it individually at home.

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E-WASTE RECYCLING PRACTICES IN BANGLADESH (A CASE STUDY ON CHITTAGONG CITY)

Md. Ashraful Islam[1*], Kutub Uddin Chisty[1] & Md. Shahjalal Misuk[1]

[1] *Department of Urban & Regional Planning, Chittagong University of Engineering & Technology, Chittagong, Bangladesh.*
**Corresponding Author, < ai_rony@yahoo.com>*

ABSTRACT

The exponential growth in the field of telecommunication and information technology is leading towards the development of new electronic appliances. Electronic waste, e-scrap, or Electronic-disposal, waste electrical and electronic equipment (WEEE) describes discarded electrical or electronic devices. There is a lack of consensus to resale, reuse, and refurbishing industries, or only to a product that cannot be used for its intended purpose. Informal processing of electronic waste in developing countries may cause serious health and pollution problems, though these countries are also most likely to reuse and repair electronics. An estimated 50 million tons of E-waste are produced each year. According to a report by UNEP titled, Recycling from E-Waste to Resources," the amount of e-waste being produced including mobile phones and computers could rise by as much as 500 percent over the next decade in some countries. This study aimed at examining the current practice of recycling e-waste in the informal sectors of Bangladesh and identifying the problems towards development of a system of hazard-free e-waste recycling in the country especially in Chittagong. Findings of the study on the informal recycling sector revealed that, the main source of e-waste is ship breakage electronic products in Chittagong. Salient features of these recycling markets include inefficient recycling process, ignorance regarding hazards of e-waste, higher profit extraction and no government control or monitoring. Recommendations such as, mass scale awareness program, development of legislation and infrastructure for hazard-free recycling, organizing and extending producer responsibility, and providing incentives, were provided to develop the e-waste handling capacity of the country.

Keywords: E-waste, recycle, Electronic, disposal, awareness.

INTRODUCTION

In recent years, as a result of fast growing access to technology and the rapid growth of the economy, a market has emerged for computers, electrics and home appliances. Electronic waste defined as secondary computers, electronics device, mobile phones, and other entertainment items such as television, refrigerators, whether sold or discarded by their original owners. According to the global trend in Bangladesh also the market for electronic goods in having exponential growth due to rising disposable income and increasing demand for the latest electronics products. A large proportion of waste generation in our country comprise E-waste. In Bangladesh generally e waste includes cell phone, television, telephone, washing machine, air conditioners, electronic toys, etc. But in addition to this waste electronic products such as printer, compass, light, radio, horn, etc. generated from ship breakage industry also constitutes a significant quantity of E-waste in Bangladesh. According to BEMMA Bangladesh consumes around 3.2 million of electronics products each year. Of this amount only 20 to 30 percent is recycled and the rest of the waste is released in to landfills, rivers, drains lakes, canals, open spaces which are very hazardous for the health and environment. Chemical such as lead, mercury, copper found in computer scene and TVs and berylliums in motherboards are poisonous. It can lead to fatal disease like cancer kidney failure and damage the environment through soil water pollution (Sinha at al. 2007). Presently there is no specify laws and ordinance for e-waste management and recycling in Bangladesh. There is no formal plants to recycle e-waste in a hazard free system. Most of this electronic products are recycle by the informal sector that's mainly located in Dhaka and Chittagong city. The main objective of the study is to identify the current e-waste recycling process in Bangladesh mainly in Chittagong. There also to detect the sources, e waste hotspots, present map of recycling process, understanding the awareness and problems. This research report has been written based primary data as well as based on the secondary Sources of information. This report will also give a recommendations for way forward.

CURRENT STATUS OF E-WASTE

Quantity

According to a study more than 500 thousand computers were in use in 2004 and this number has been growing 11 percent annually (Hossain 2007). If the figure of 500 thousands were taken as the base line that many pc would contains approximately 30646 tons of waste in 2010 containing deadly plastics lead mercury etc. the quantity of e-waste to be generated has been estimated by two method (Sinha et al 2007) the first method market supply method A, assume that the average life time of an electronic products is approximately five years and after that these are discard. And the second method market supply method B assume that all the product are not disposed at the same time rather they are disposed in varying quantities over different years . Here weighted average method is used to show the disposal trend. We consider for pcs growth rate is 11.4 percent (Hossain, 2004) and cell phone 100 percent growth rate annually (Pervez at al, 2007).

Table 1: Estimation of PCs and Cell phones.

Year	Personal Computers weights(in tons)			Cell phone Weight(in tons)		
	Method A	Method B	Average	Method A	Method B	Average
2010	33402	27890	30646	5134	5648	5391
2011	36502	33402	34952	10270	10270	10270
2012	39604	32232	35918	15404	15404	15404
2013	42704	35516	39110	20539	20196	20368

Note: 1. Weight of PCs is derived 27.2 kg/PC (Sinha et al, 2007).

2. Weight of Cell phones is derived .0792 kg/Cell phone (Pervez at al , 2007).

Current situation

Bangladesh is a signatory to the Basel convention on trans-boundary movements of hazardous waste. Currently there has no specific regulation & rules dealing with e-waste management. The ministry of environment and forest (Moef) is in process of formulating the rules on e-waste management. As the reported low level of knowledge on hazardous item found in e waste they showed a very positive attitude towards development of hazard free e-waste management system. The aggregate value of the willingness to pay (WTP) for the e-waste management was BDT 1.13 billion (USD 16.16 million) (Ahmed, 2010).

E WASTE RECYCLING IN CHITTAGONG

Chittagong the Port city which generates the high quantity of e waste due to existence of ship breaking industry and other heavy industry. The recycling process of the informal sector in Chittagong most of second hand electronic products are purchase by recycling shop owner from auction. The auction held in the vatiary area of the Chittagong city.

Recycling flow

The recycling process of the E-waste followed by all the market mentioned is almost similar. The shop owners buy the old electronics products from the auction held in shipyard. This auction accumulates various items salvaged the end of life ship as scarp. After buying and taking this delivery they clean and repair the electronic products. The repaired useable products are sold to retailer, and wholesalers. The non-recoverable item are also valuable as include metal such as iron steel, bronze cables etc. Then these are sold to scrap dealers. According to the shop owner and workers almost all the purchase item are either sold by repairing or sold as scrap. A very small quantity is thrown away as a waste. The recycling process is shown in Fig 1.

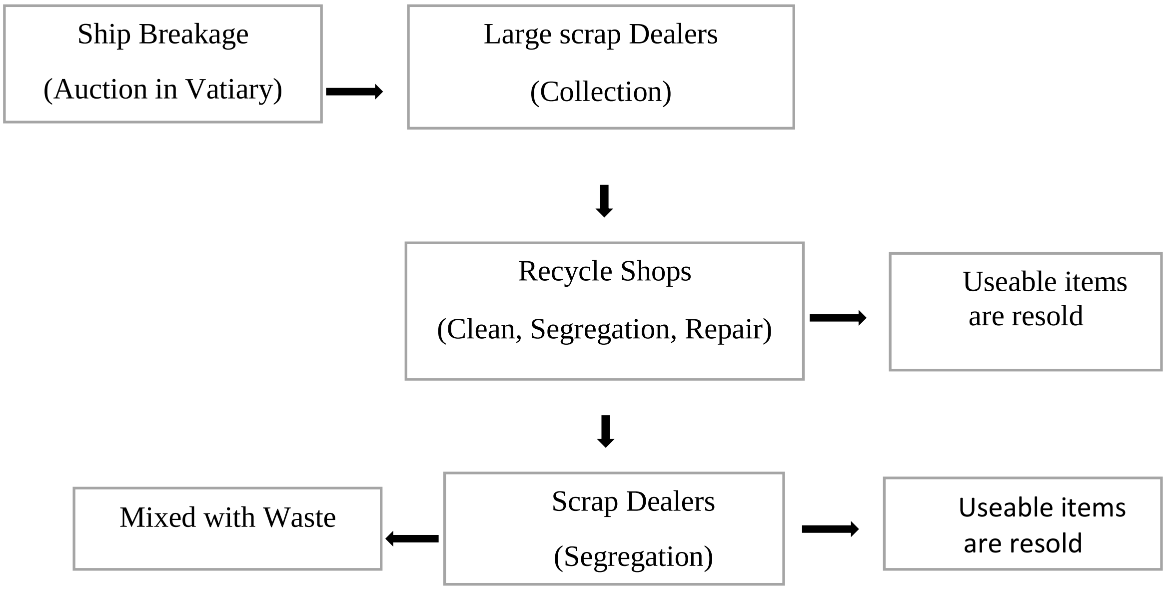
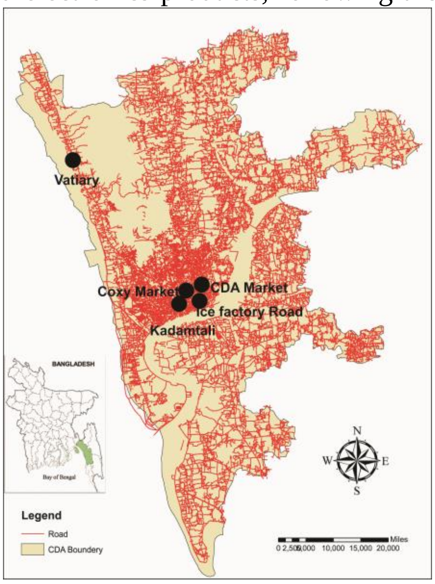


Fig 1: E-waste recycling flow in Chittagong

E-waste hotspots

There are different areas in Chittagong that handles second hand electronics products, following are the key areas dealing with e waste recycling among them.

CDA market
Coxy market
Ice factory road
Vatiary
Kadamtali



CDA Market Chittagong.

CDA market area is one of the biggest markets in Chittagong which handling electronic second hand products. This informal market has around 50 shops. They buy the electronic products from auction in the vatiary shipyard resulting from ship breakage. It handles electronics products such as auto pilot, printer and ship navigation related products. After buying this product from the auction they clean and repair this products. From the shop owners it was revealed that the shop can salvages 50 percent of the purchase products. These products sells to buyer at home and abroad. The rest of non-recoverable item such as steel iron cable are sold as scrap. They think that their current recycling process is safe and do not agree that these electronic product contain toxic materials which can be hazardous for human health and environment.

Table 2: Description of the products handle

E-Product	Avg. weight(KG)	Purchase price(BDT)	Selling price(BDT)
Auto pilot	170	100,000	115,000-130,000
Printer	5	15,000-20,000	20,000-25,000
Gyno Compass	60	80,000-150,000	100,000-180,000
Repeater	10	4,000-6,000	8,000-10,000
AIS	3	40,000	60,000

Coxy market, Chittagong

Coxy market area is another small second hand electronic products handling market in Chittagong. This informal market has around 15 shops. They buy the product from auction. This market is not as CDA market and has mainly fridge and air conditioner. They also flow the common procedure of recycling shops. According to the shop owners they can salvage approximately 90 percent of the purchased products and sell these to retail buyers. The rest of the non-recoverable item are sold as scrap. The owners of these shops are in business for relatively shorter period of time and invest BDT 100 thousand. They also think their recycling process is safe. As these electronic products are originally produced in Europe having higher safety standards and believe that’s these do not contain any toxic materials.

Table 2: Description of the products handle

E-Product	Avg. weight(KG)	Purchase price(BDT)	Selling price(BDT)
Fridge	35-50	5000-7000	8000-10,000
Air conditioner	40	10,000-12,000	12,000-16,000

Ice factory road, Chittagong

The market locate in ice factory area is a medium size market. Here mostly buy the product from auction in shipyard from ship breakage. It deals with electronic products such as generator hydraulic pump, panel board, and compressor. After buying this product from the auction they clean and repair this products. The shops can salvage approximately 50-60 percent of the purchased products and sell these to local wholesale market. The rest of the non-recoverable item are sold as scrap. The owners of these shops are in business for relatively moderate period of time and the capital of this market about 100-1000 lakhs. They think their recycling process is not safe.

Table 2: Description of the products handle

E-Product	Avg. weight(KG)	Purchase price(BDT)	Selling price(BDT)
Generator	1-3tons	10,00,000-50,00,000	15,00,000-75,00,000
Hydraulic Pump	1 ton	30,000-40,000	40,000-50,000
Compressor	500-1000kg	4,00,000-10,00,000	5,00,000-12,00,000

Vaiary, Chittagong

Vatiary is another big electronic product recycling market. It's located inside naval base. It has 80 shops. This market has wide variety of electronic products ranging from different types of light to horn, radio, television etc. after buying this product from the auction in the shipyard they clean and repair this products. The shops can salvage approximately 70-80 percent of the purchased products. The rest of the huge non-recoverable item are sold as scrap. In this market large capital investment approximately 40 lakhs having income of BDT 10 lakhs. They think that their current recycling process is safe and do not agree that these electronic product contain toxic materials.

Table 2: Description of the products handle

E-Product	Avg. weight(KG)	Purchase price(BDT)	Selling price(BDT)
Charge light	40-60	15000-25000	20000-40,000
Deck Light	5-12	2000-3000	3000-5000
Navigation Light	5	3000-5000	4000-7000
Radio	4	2000-4000	3000-5000
Television	10-12	8000-10,000	10,000-12,000
Wind Direction Meter	8	12,000-14,000	18,000-22,000

Kadamtali, Chittagong

Kadamtali is another electronic product recycling market in Chittagong having approximately 20 shops. Is market has fan, printer, washing machine, IPS etc. electronic products. According to the shop owners they can salvage approximately 50 percent of the purchased products and sell these to retail buyers. The rest of the non-recoverable item such as cable plastics etc. are sold as scrap. They do not agree that these electronic product contain toxic materials.

Table 2: Description of the products handle

E-Product	Avg. weight(KG)	Purchase price(BDT)	Selling price(BDT)
Wall fan	1.5	120-150	200-250
Table Fan	2	180-200	300
Television	8	5000-6000	8000-9000
Fridge	15-40	8000-10,000	12,000-13,000
Washing Machine	10-12	2000-3000	4000-5000
Printer	5	3000-4000	4000-6000
Computer	12-15	8000-10,000	12,000-14,000
IPS	5	3000-4000	6000-7000

RECOMMENDATION

To ensure hazard free recycling of e waste Comprehensive and sustainable laws are needed. Which will be based on polluter pay principle, government should enact rules for e waste management and handling.

There should be initiated a large scale awareness programme for all the stakeholder to enhance their understanding regarding the danger of e-waste.

In our country waste are not separated before disposal. Initiatives should be taken to separates garbage in to burnable, non-burnable and e-waste. This will help to segregate waste easily and isolate e waste which will in turn increase recovery by reducing wastage.

Collection point refurbishing and recycling center should be establish in urban and rural areas. Government can also play an important role by providing intensive.

This is an important to establish to an e-waste treat plant. This will be public private partnership. Producer should registered with the recycling agencies and treatment plant for paying the cost. Treatment cost might be share by both producer and consumers

Producer's responsibility should be extended that they will ensure hazard free disposal. They should be responsible for the product after their useful life and pay the cost to the recycling agency. This will encourage redesign and reduce the use of toxic materials by developing alternative materials and development of environment friendly technology

Registration of producers should be establish to make them more accountable.

Government should think of providing incentives for being of environmental friendly and encourage the use of cleaner material technology in recycling. Incentives in the form of tax exemption might be provide to the producers using toxic free materials

CONCLUSION

While the problem of e-waste were widely discussed and it is to help policymakers with appropriate policy instrument. This study attempted to take modest attempt to investigate the recycling sector in Bangladesh.

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Our sincere thanks to Shop owner of the study area for their support for conducting this study. This could not have been possible without the support and willing participation of selected stockholders, and civil societies.

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WATER POLLUTION STATUS OF KARNAPHULI RIVER NEARBY CHITTAGONG CITY CORPORATION AREA

I. Haider[1*], A. Kadir[1], R. A. Mullick[1] & S. K. Palit[1]

[1] Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong - 4349, Bangladesh.
**Corresponding Author, <imran.haider.082@gmail.com>*

ABSTRACT

Chittagong, the second largest city of Bangladesh, possesses one of the most potential rivers Karnaphuli in its heart. This important river, during its course, serves the city in lots of ways but receives untreated pollutant loadings from the metropolitan area and neighbouring villages in reply. The paper presents an attempt to evaluate the water quality of the river to comprehend the state of pollution that can jeopardize the lives of aquatic species and other human uses of water. The water samples have been collected from different points of the river to find the physical and chemical quality of the river water. Water quality of the collected samples have been immediately tested in the environmental laboratory of Chittagong University of Engineering and Technology, CUET. The results found have been compared with the standards suggested for inland surface water in the Environment Conservation Rules (ECR), 1997 and quality criteria for water by Environmental Protection Agency (EPA). The comparison shows that the river near Chaktai and Rajakhali Dissolved Oxygen level is too low and also other parameters neither satisfies the criteria for aquatic lives nor can be used for irrigation or other human activities. Also at the other points of Karnaphuli the water quality is alarmingly poor that represents the water pollution. Study shows that pollution of Karnaphuli River water is a great concern where authorities need to pay attention by restricting and treating all kinds of harmful disposal into the river.

Keywords: Karnaphuli River, Water pollution, Water quality, Dissolved Oxygen, Aquatic life, Irrigation.

INTRODUCTION

Karnaphuli is not only the largest river of the port city Chittagong but also one of the most important rivers of Bangladesh. Being originated from the Lusai hills at Mizoram in India this river travels south-west to the Bay of Bengal through Chittagong hill tracks and Chittagong city for about 270 km (Encyclopaedia Britannica). It possesses about 14000 square kilometers of catchment area including all of its allied streams. This coastal river accommodates the Chittagong district and city corporation area with more than 700 numbers of small and heavy industries and the main seaport of the country. Karnaphuli river contributes in export-import industry, fish industry and hydro-electric power generation for our country, drinking water supply for Chittagong city and navigation for water route, employment of many fishermen, boatmen, health of the general city dwellers and a significant prospect of tourism in its 88 kilometers of course in Bangladeshi border. Chittagong city is expanding and developing with the addition of population, industrial activities, agricultural practices and other infrastructures. As a result a large amount of industrial effluents, municipal wastes and oil and grease are discharged into the Karnaphuli River without any treatment through different canals and drainage systems.

From Kalurghat to the estuary at Patenga the Chittagong city is in contact with this river. In this region there are 20 canals (khals) which dispose the wastewater and storm water into Karnaphuli. This adds a large volume of organic and inorganic substances, which changes the chemical characteristics of the surface water by producing toxic substances and ultimately pollutes the Karnaphuli River. As

Karnaphuli is a tidal river, it experiences high tide and low tide twice a day and thus frequently mixes with the saline sea water from Bay of Bengal. At Sadarghat area salinity values found varying from 0.1 gm/l at low tide to 0.2 gm/l at high tide during monsoon and during dry season 2.5 gm/l at low tide to 16.5 gm/l at high tide (Chittagong Port Authority). According to DoE 2001, alkalinity level at the Kaptai barrage , Karnaphuli paper mill in Chondroghona, the Shah Amanat bridge near the Chittagong city and TSP plant situated at the end of the Karnaphuli exceeds the environmental quality standard for Bangladesh as prescribed by the World Health Organization. In an environmental study near the Kalurghat industrial area Cr, Cu, Zn, Pb contamination values of Karnaphuli River were found such that it was unsafe for fish culture, livestock and agriculture (Majid et. al, 2003). According to Alam, 2013 the value of Dissolved Oxygen (DO) in Karnaphuli River ranges between 2.40 mg/l and 2.88 mg/l, salinity ranges between 0.01 ppt and 0.05 ppt and PH value ranges between 7.5 and 8.5. Alam, 2013 concluded that the condition of the river was critical due to decreasing trend of DO values. A green organization, Paribesh Bachao Andolon, found the DO value varying between 3.37 mg/l to 6.37 mg/l (The Independent, 2014). Even though the diurnal tidal cycle is minimizing the pollution of river water, the quality of Karnaphuli River water is deteriorating. Condition of the river gets more severe while accidents like occurrence of oil spilling happens. For example, in 30th July 2013, 100,000 liters of furnace oil spilled over into the river water (Prothom Alo, 2013). Such occurrences and the continuous pollution of the river affect the phytoplankton and zooplankton which adversely affects the lives of fishes, shrimps and prawns. Water experts and scientists apprehend that if the river pollution continues unrestricted, Karnaphuli will become devoid of fish and other aquatic resources in the near future.

This initiative has been taken to assess the present condition of Karnaphuli River near Chittagong City Corporation area by analyzing the water quality from different location of the river along with physical survey. The main objective of this project is to measure the water quality of Karnaphuli River, evaluating the pollution condition due to canal discharges and thus finding the possible impact of pollution regarding the criteria for aquatic lives and other uses of inland surface water.

METHODOLOGY

To evaluate the Karnaphuli River water pollution, physical investigation has been conducted to the study area. Previous state of the river was evaluated by collecting and analysing the secondary data. Canals that meet the river from adjacent city or area have been taken as the observation locations. Then sampling locations have been selected and sample water has been collected from those locations. Collected water has been tested to evaluate the water quality and then compared to the standard values for aquatic lives and other inland surface water uses.

Study Area

Karnaphuli River receives the maximum pollutant loading from the Chittagong city that is adjacent to the segment from Kalurghat to estuary at Patenga. This segment of the river has been taken as the study area for physical investigation. The study has been conducted on pre-monsoon dry period at March 2014 to April 2014. To assess the river water quality, 9 sampling location in the river have been selected as the sampling location based on the important canal discharge. The sampling locations and the map of study area are shown in Fig. 1 and described in Table 1.

Table 1: Locations of Water Sample Collection for Quality Testing

No	Khal Name	Latitude	Longitude	Distance from Estuary (km)	Avg. River width (m)
1	Khal 15	22°14’21”	91°49’23”	3.05	722
2	Gupta Khal	22°16’21”	91°48’46”	8.86	517
3	TSP Khal	22°16’50”	91°47’57”	10.51	416

4	Saltgola Khal	22°18’03”	91°47’57”	12.88	571
5	Laitta Khal	22°19’22”	91°50’35”	18.17	686
6	Fishery Khal	22°19’23”	91°50’43”	18.39	570
7	Rajakhali Khal	22°19’25”	91°50’51”	18.63	537
8	Chaktai Khal	22°19’30”	91°51’08”	19.14	497
9	Shikalbaha Khal	22°19’36”	91°51’29”	19.78	688

Sample collection dates: 27/03/2014 and 08/04/2014; Source: Google Earth, 2014

Fig. 1: Map of Karnaphuli River

Fig. 2: Severely Polluted Area

INVESTIGATION

Physical Investigation

Due to lack of proper sanitation and waste collection facilities, sewage and solid wastes from all over the city find their way into the Karnaphuli River through 5 canals. The collected solid wastes are dumped in two dumping grounds adjacent to the Karnaphuli River estuary that causes pollution. The pollution of river water near the urban area is 4 to 10 times more polluted than the countryside locations (Ahmed and Rahman, 2000).

Considerable amount of blood and viscera of about 400 slaughter animals from Feringi Bazar and Dewan Hat slaughter houses find their way into the river Karnaphuli. Sometimes dead animals are directly thrown in the river.

Heavy metals such as mercury enters the Karnaphuli river by Chittagong Chemical Complex (CCC) and Karnaphuli Paper Mills (KPM), lead enters from the oil refinery, chromium from tanneries, cadmium from dying and painting and arsenic from Urea Fertilizer Factory.

The width of Karnaphuli River is reduced by a great measure due to land grabbing. Siltation is occurring in the river bed, which is reducing the depth of river and causing difficulties in navigation at low tide period.

More than 1000 number of ships and 40–50 oil tankers in Chittagong port are handled annually. Besides, numerous river craft, launches and steamers also play along water ways and discharge waste oil spillage, bilge washing, into the water and create pollution in the marine environment. They affect coastal fisheries in both qualitatively and quantitatively.

Experimental Investigation

Sample Collection

From each location water samples have been collected from two points in both high tide and low tide condition. The two points are the confluence of canal and the river and middle of the river along the confluence point. Samples have been collected in plastic bottles with screw cap from 0.1meter depth manually to avoid debris. New or well washed bottles have been used before collection to make sure that it is completely free from any undesirable materials.

Sample Testing and Analysis

The collected water samples have been immediately brought to the environmental laboratory of Chittagong University of Engineering and Technology to test. Each sample has been tested for 5days Biochemical Oxygen Demand (BOD_5), dissolved Oxygen (DO), alkalinity, PH and total dissolved solid (TDS). The laboratory test results are then compared to the standard values specified by Environment Conservation Rules, 1997 and EPA for aquatic lives and inland surface water use which are given in Table 2.

Table 2: Water quality criteria and standards for aquatic life and inland surface water use

Quality Parameter	Standard for Aquatic life	Inland Surface water Use Standard			
		Drinking water supply	Recreational Activity	Irrigation	Various process and cooling industry
BOD_5	<3 mg/l	<2 mg/l	<3 mg/l	<10 mg/l	<10 mg/l
DO	5 mg/l	>6 mg/l	>5 mg/l	>5 mg/l	>5 mg/l
PH	6.5 to 9	6.5 to 8.5	6.5 to 8.5	6.5 to 8.5	6.5 to 8.5
Alkalinity	>20 mg/l	-	-	-	<500 mg/l
TDS	Varies with fishes	<250 mg/l	-	Varies	-

Source: ECR, 1997 and EPA, 1976

RESULT AND DISCUSSION

Laboratory test results of collected samples from various locations of Karnaphuli River are depicted in Table 3. It is found that the DO values ranging from minimum 1.4 mg/l at the confluence of Chaktai khal during low tide to 8.0 mg/l at the middle of the river near TSP and Saltgola Khal during high tide. BOD_5 values were found to be ranging from maximum 2.2 mg/l during low tide to minimum 0.2 mg/l during high tide at several locations. When BOD_5 loadings increase, DO values decreases as the bacteria in the water consume the available Oxygen (sawyer et al., 2003). Since DO concentration is less in some locations of Karnaphuli River (shown in Fig. 2), some species of fish and other aquatic organisms may not survive.

Table 3: Water Quality Parameters and Values of Karnaphuli River

At the time of Low Tide										
Khal Name	BOD_5 mg/l		DO mg/l		Alkalinity mg/l		PH		TDS mg/l	
	Con	Mid	Con	Mid	Con	Mid	Con	Mid	Con	Mid
Khal 15	0.6	1.8	6.0	8.0	130.0	180.0	7.05	7.15	1786	2300
Gupta Khal	0.2	1.0	6.6	6.8	145.0	120.0	7.00	6.98	1652	2130
TSP Khal	0.6	2.2	5.0	8.2	150.0	170.0	7.00	6.50	1608	2050
Saltgola Khal	1.8	1.6	6.4	7.6	125.0	150.0	6.40	6.20	1570	2025
Laitta Khal	1.2	1.0	2.6	6.0	75.00	85.0	7.15	7.10	1390	1256
Fishery Khal	0.8	0.6	4.8	5.0	60.00	55.0	7.10	7.00	1124	1200
Rajakhali Khal	0.4	0.5	3.6	2.0	90.00	98.0	7.30	7.20	900	1210
Chaktai khal	0.4	0.8	1.4	8.0	110.0	57.0	7.65	7.50	790	840
Shikalbaha Khal	1.0	1.0	6.0	7.0	48.00	40.0	7.67	7.47	560	600
At the time of High Tide										
Khal 15	0.2	1.0	7.8	7.0	110	125.0	7.05	7.25	17600	22610
Gupta Khal	0.2	1.4	7.4	7.0	120	110.0	7.13	7.00	17354	20470
TSP khal	0.2	2.0	5.8	8.0	120	115.0	7.26	6.50	14760	18300
Saltgola Khal	2.0	1.0	6.0	8.0	110	110.0	7.43	6.00	11361	17800
Laitta Khal	1.0	1.4	7.8	5.2	53	47.0	7.05	7.05	9568	9173
Fishery Khal	0.8	0.8	4.2	5.2	43	41.0	7.05	7.05	9232	8594
Rajakhali Khal	1.6	1.0	5.6	5.8	46	43.0	7.90	7.32	8206	8523
Chaktai Khal	0.8	0.6	5.0	7.8	51	45.0	7.90	7.85	9109	8396
Shikalbaha Khal	0.6	0.2	6.0	6.0	50	50.0	7.60	7.43	6913	5783

Reports shows that from 66 species of fresh water, 59 species of mixed water and 15 species of migratory fish 20 to 25 species of fresh water and 10 species of mixed water species have become extinct (The Independent, 2014 & Daily Observer, 2014). Moreover, inland surface water uses like use for recreational activity, irrigation or various process and cooling industries should be sidestepped too as DO concentration does not meet the minimum criteria (ECR, 1997) and EPA water quality criteria. Fishery biologists have determined that for a healthy and diverse fish population DO concentration should be minimum 5mg/l (EPA, 1976). The DO variation at different locations of the river is shown in Fig. 4.

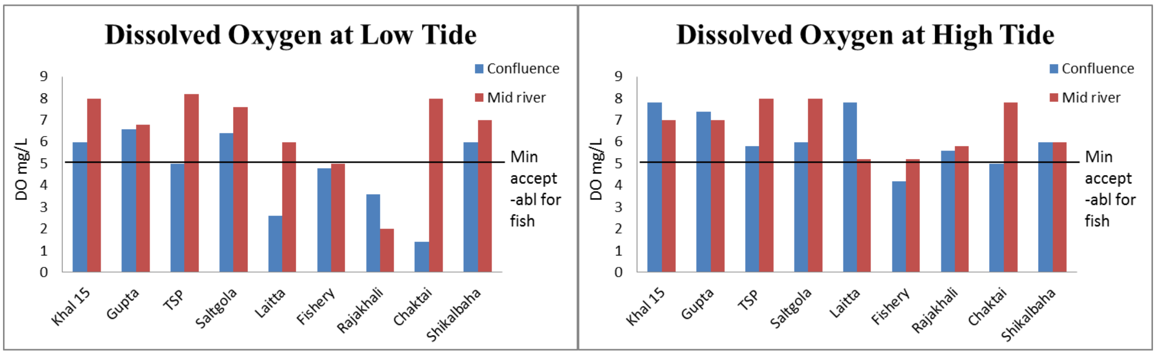

Fig. 4: DO Concentration in Karnaphuli River

The concentration of alkalinity varies from 40 mg/l to 180 mg/l. Values shows that near the estuary alkalinity are generally of greater values. Water bodies should contain alkalinity concentration more than 100 mg/l for highly productive diversity (Rahman, 1992). But the river near Chaktai and nearby other 3 canals the alkalinity concentration is relatively low that denotes polluted water.

Total Dissolved Solid concentration value shows that, it increases to the downstream due to more dissolved salts and substances. At the nearest location towards estuary, Khal 15, TDS value varies from maximum 22610 mg/l during high tide to 1786 mg/l during low tide.

CONCLUSION

Obtained values of water quality parameters show that at the canal confluence points most of the samples do not satisfy the criteria suggested by ECR, 1997 and EPA. Though tidal effect and self-purification mechanism recovers the condition at mid-river, pollution state is still considerably alarming. But in few locations certain parameter values are within the standard limit. To recover the soundness of aquatic environment of the Karnaphuli River for aquatic organism as well as fisheries, it is needed to raise awareness regarding the water quality problems through different form of education, monitoring and research. The authority concerned should pay attention on this and take necessary steps to prevent the pollution of Karnaphuli River.

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EVALUATING THE EFFECTIVENESS OF RETENTION TANK ON IRON REMOVAL EFFICIENCY, A CASE STUDY OF CUET IRP

Sabbir. Ahmmed[1*], Md. Ibrahim Khalil[1] & Sudip Kumar Pal[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong - 4349, Bangladesh.*
**Corresponding Author, <sabbir_ce@yahoo.com>*

ABSTRACT

Iron Concentration found in ground water often exceeds the limit with an order of magnitude for drinking water quality. Conventional design of groundwater treatment plant requires several steps to bring iron concentration down the limiting value. Due to the presence of high iron concentration in pumped water, the removal efficiency at various treatment stages continuously dropped down within a short period, thus increase frequency of regular maintenance. In order to increase the removal efficiency and decrease the pressure on maintenance, a retention tank has been proposed as a case study which receives and retains water initially after pumping for several hours using CUET IRP. It is seen that for retention time of 2, 3 and 4 hours iron concentration is reduced by 19%, 33% and 45.6% respectively. Optimizing the cost and removal efficiency, three hour retention time is found acceptable. The water is then kept three hour in retention tank, passed through two filters (RSF & SSF) and found water with little or nil iron concentration. In addition decrease of filtration rate compared to existing filtration rate is minimum. It has been found that addition of retention tank not only reduce the iron concentration but also keep system running hassle free, thus to avoid frequent backwashing of filter and regular maintenance with existing treatment system.

Keywords: Iron concentration, Retention tank, Removal efficiency, Maintenance.

INTRODUCTION

The intensive use of ground water is increasing day by day in Bangladesh since last four decades. In recent time, it is even found as solo source, as surface water is seriously polluted due to the inexistence of waste water treatment plant. The ground water mostly contains iron, for which typical iron removal plant (IRP) exists to take care of excess concentration above the limit set by the guidelines. The iron present in ground water is found in two phases, e.g. in suspended form and in dissolved form. When iron is present in suspended form, then there is a possibility of increase filter clogging with conventional IRP which in most of the cases designed based on dissolved portion of iron rather than suspended portion. The scenarios of CUET IRP falls with suspended form of iron in ground water, which clogs filter bed very frequently even after regular backwashing preceded by aeration, flocculation and sedimentation. To keep the system running for a long period with minimum maintenance, it is therefore anticipated that the suspended form of iron could be better trapped earlier providing a pre-sedimentation tank/retention tank. This study evaluates the effectiveness of retention tank at IRP with modified treatment train compared with present system so as to avoid frequent backwashing and maintenance without compromising quality improvement.

METHODOLOGY

Study Area

In CUET campus water is supplied by its own authority. The source of water is only ground water. The total area of CUET campus is 161 acres where groundwater is stored into the central water tank and then supplied to the different needed zones. But last several years due to the increasing water demand the reserve tank capacity of present IRP is not adequate and clogging of filter bed contributing severe water loss. At present CUET has approximately 6000 population and demand of water is 750 m^3/day. The main source of water supply in CUET is ground water.Water that is pumped out by a 40 HP and 30 HP pump and then allowed to pass through different stages such as aeration, flocculation, sedimentation and filtration processes. After that the filtered water reaches to the reserve tank from where it is pumped to the overhead tank and then supplied. It is found that the water of this area has high iron concentration. The high concentration of iron clog the filter media very rapidly and due to the clogging of filter media frequent backwashing is required (Ahmed and Rahman, 2000).

Field Visit

To know the present condition of iron removal plant of CUET, field survey was carried out at the area. The present treatment process of the study site was as following.

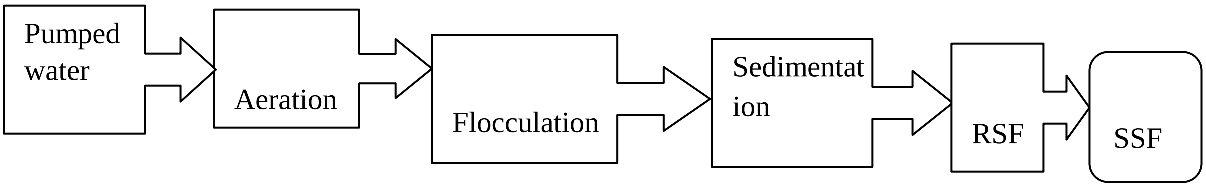

Fig.1 Present treatment process of CUET IRP.

Sample Collection

From each stage of IRP sample was collected. Water samples were collected in clean washed plastic bottles. Sample collected 5 days by turns and necessary test was conducted.

Sample Testing and Analysis

Collected Sample was immediately taken to the laboratory and tested to evaluate the iron concentration and compared with the standard value for drinking purpose. Water was kept for 2, 3 and 4 hours and iron concentration was measured and then was filtered. The filtrated water was tested again to measure iron concentration to evaluate the effectiveness of retention tank. The water quality parameters were measured before and after filtration.

Fig.2: Collection of water sample from aeration chamber at CUET IRP, kept in laboratory and tested in the laboratory.

Filter preparation

Two filters were made for the treatment of pump water and the water in different time interval. Two bucket was used for the preparation of filters (RSF & SSF). The locally available material was used to design Rapid Sand Filter (RSF) and Slow Sand Filter (SSF). The design procedure are as follow:

Rapid sand filter

A bucket of 30 cm height with diameter of 31 cm and 23.5 cm at top and bottom respectfully is need to prepare RSF. The filter sand media is supported on base material consisting of graded gravel layers.

The different sizes of gravel free from clay, dire, vegetables and organic matter are used **(**Das et al., 2007). The size of gravel used is presented in Table 1:

Table 1: Grade size and depth of layer in rapid sand filter

Layer	**Depth**	**Grade Size**	**Sieve Size**
Top most layer	6 cm	20-50 mm	3/2″ -3/4″
Intermediate layer	6 cm	12-20 mm	3/4-″3/8″
Intermediate layer	6 cm	6-12 mm	3/8″ - #4
Bottom layer	6 cm	2-6 m	#4 - #8


Fig. 3 Preparation stages of rapid sand filter.

Slow sand filter

In slow sand filtration, water is allowed through a bed of fine sand which retains most of the impurities present in water. The filter sand media is supported on base material consist of sand. Sylhet sand was used to achieve more filtration rate. F.M of the sand was 2.70. The different layers are composed of different materials as shown in Table 2.

Table 2: Grade size and depth of layer in slow sand filter

Layer	**Depth**	**F.M / Size**
Top Layer (Sand)	16 cm	2.70
Bottom Layer (Gravel)	5 cm	2-6 mm


Fig. 4 Preparation stages of slow sand filter

Analytical Technique

Iron concentration is measured at Environmental Engineering Laboratory at CUET by titration method. Sample preparation, quality control and quality assurance were made as per guideline of testing iron in water (Ahmed and Rahman, 2000). Volume of retention tank was designed based on cut off retention time.

RESULTS AND DISCUSSIONS

Iron occurs in underground water as soluble (Fe++) form and it becomes as an insoluble (Fe+++) form when it comes in contact with air (Ahmed and Rahman, 2000). The insoluble iron sediments at the bottom of the container and the water thus found as less concentration of iron. Therefore, concentration of iron decreases with time. The graph below shows the same characteristics.

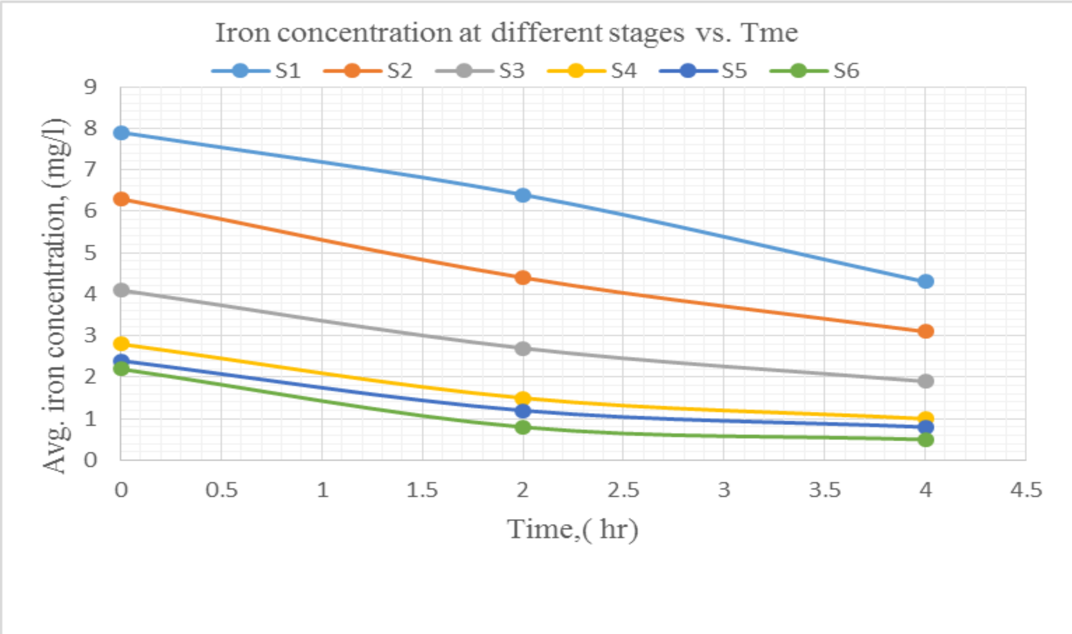

Fig. 6 Reduction of iron concentration at different stages with time interval.

Note: S_1 = Pumped water, S_2 = Water sample after aeration process, S_3 = Water sample after flocculation process, $\mathbf{S_4}$= Water sample after sedimentation process, S_5= Water sample after passing RSF, S_6 = Water sample after passing SSF.

As seen in Fig. 7, iron concentration in pumped water (S_1) is found as 7.9 mg/l, after two hour 6.4 mg/l and after four hour 4.3 mg/l during collection, two hours after collection and four hours after collection respectively. Similarly, the iron concentration in water after aeration process (S_2) is found as 6.3, 4.4 and 3.1 mg/l during collection, two hours after collection and four hours after collection respectively. The iron concentration in water after flocculation process (S_3) is found as 4.1, 2.7 and 1.9 mg/l during collection, two hours after collection and four hours after collection respectively. The iron concentration in water after sedimentation process (S_4) is found as 2.8, 1.5 and 1.0 mg/l during collection, two hours after collection and four hours after collection respectively. The iron concentration in water after passing RSF (S_5) is found as 2.4, 1.2 and 0.8 mg/l during collection, two hours after collection and four hours after collection respectively. The iron concentration in water after passing SSF (S_6) is found as 2.2, 0.8 and 0.5 mg/l during collection, two hours after collection and four hours after collection respectively. The result suggests that the iron concentration is decreasing with time.

Table 3: Percentage reduction of iron concentration with time for different stages of IRP at CUET.

Time	S_1 (%)	S_2 (%)	S_3 (%)	S_4 (%)	S_5 (%)	S_6 (%)
After 2 Hr.	19.0	30.1	35.7	46.4	50.0	63.6
After 4 Hr.	45.6	50.8	54.8	67.9	45.0	77.3

Results found from Table 3, indicates that iron reduces more when water is kept for a few hours. Therefore, the pump water was kept for several hours, measured percent reduction of iron concentration after different hours and after filtration to determine effective retention time. The modified design flow diagram with retention tank is identified as seen in fig.7.

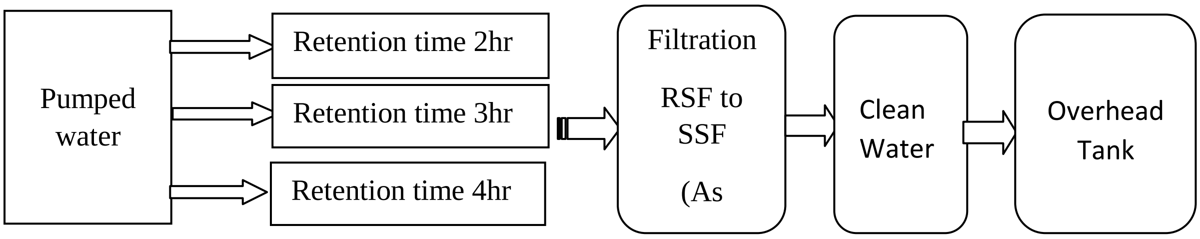

Fig.7 Modified treatment train for IRP at CUET.

Filtration process involved three techniques .These are passing water through RSF only, SSF only and passing the water through RSF and then SSF. It was found that last one is more effective.

Table 4: Percent reduction of iron concentration after several hours & after filtration.

Iron Concentration (ppm)	Percent reduction (pumped water – water after several hours)	Percent reduction (water after several hours - filtrated water)
Pumped water= 15.0 ppm After 2 hours= 11.8 ppm Passed through R.S.F only = 4.8 ppm Passed through S.S.F only = 2.6 ppm Passed through R.S.F to S.S.F = 1.4 ppm	21.3%	90.7%
Pumped water= 15.0 ppm After 3 hours= 9.2 ppm Passed through R.S.F only =2.6 ppm Passed through S.S.F only = 0.4 ppm Passed through R.S.F to S.S.F = 0 ppm	38.7%	100%

The results as obtained from Table 4, suggest that if the water is kept for 3 hours and then only filtered through RSF and SSF, it becomes fully iron free. Hence, the effective retention time is 3 hours.

Size of retention tank

Based on the Calculation considering future expansion and development of CUET it is found that demand reaches 750 m^3/day. Considering this demand and cut off retention time as identified in this study, the size of retention time is 26′*26′*5′.

CONCLUSION AND RECOMENDATION

Iron concentration in water pumped at CUET IRP found from 8 to 16 ppm. Iron concentration measured at different stages of treatment and the concentrations of the treated water was found even more than allowable limit (BECR-1997).To improve efficiency of present IRP, a pre-sedimentation tank/retention tank of size 26′*26′*5′ is to be proposed with present treatment system and it found effective. Based on findings, a revised treatment system is proposed for CUET IRP and it is hoped that the system will work with similar background elsewhere.

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ENVIRONMENTAL IMPACT ASSESSMENT OF BRICK CHIPS CONSTRUCTION

Md. Shahrior Alam[1]*, Syed Ishtiaq Ahmad[1]

[1] *Department of Civil Engineering, Bangladesh University Of Engineering and Technology, Dhaka-1000, Bangladesh.*
**Corresponding Author, < shahrioralam@yahoo.com>*

ABSTRACT

Aggregate is commonly considered as the most important inert filler in concrete mixes, which forms 60 to 75 percent of the volume. In Bangladesh, brick aggregates are easily available and much cheaper than crushed stone aggregate. This paper will show the environmental impact of a construction building using brick chips .Here mainly the emission of carbon dioxide (CO_2) is taken into consideration. The impact of global warming due to climate change is adverse for a river surrounded country like Bangladesh. This paper categorically estimates the carbon dioxide (CO_2) emission and energy consumption from typically building plans for its construction life which hopefully helps us in guiding the reduction of carbon dioxide emission from a building. This necessitates the using of alternate materials to achieve environment friendly green building with lesser emission of greenhouse gas like carbon dioxide (CO_2).

Keywords: Global warming, carbon dioxide (CO_2), energy consumption, cost analysis, brick chips.

INTRODUCTION

In Bangladesh, bricks are the predominant building material in urban areas. The manufacture of bricks is an energy-intensive activity. All developed countries and some developing countries have shifted away from traditional low-efficiency manufacturing processes to modem high-efficiency ones. They have also become a significant building material in the rural areas. High prices or scarcity of alternative building materials, such as stones, iron sheets, wood, bamboo, and straw are very rapidly increasing the demand for bricks. In Bangladesh, about legal 6000 brick kilns are in operation, producing about 17.5billion bricks per year. With increasing demand for bricks, more and more paddy fields are being converted to brick-fields, thus putting tremendous pressure on the already scarce agricultural land of the country.

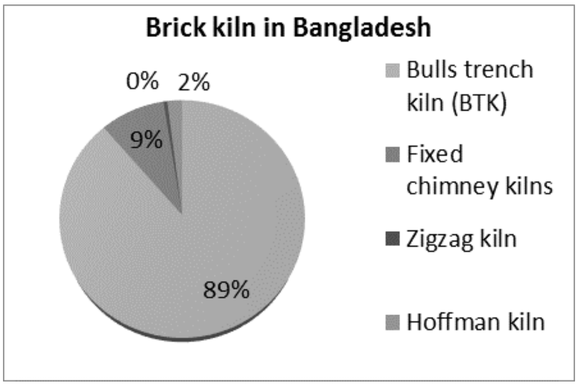

Fig.1: Percentage of Brick Kiln in Bangladesh.

METHODOLOGY

Life cycle assessment is increasingly being used to determine the environmental impacts of building and construction projects. The initial impact of a building on the environment results from the energy and other resources consumed in its construction. In this research, detail construction data are collected and analyzed from six storied residential building project situated in Dhaka, Bangladesh. The life cycle analysis (LCA) of building materials show that the energy requirement and CO_2 emission is mainly by two ways: active & passive. Here only active way is calculated. Table 1 shows the summary of LCA of building materials with the reasons of CO_2 emission & energy consumption from the preparation, transportation to the site and use of these materials.

Table 3: Reasons of CO_2 Emission & Energy Consumption of Brick

Sl. No.	Description of Construction Items	Sources of CO_2 Emission & Energy Consumption				
		Wood Cutting	Burning of Wood/Gas/Coal	Electricity for Machine Operation	Plant Operation	Fuel Burning for Transportation
	Brick					
i)	Cutting, Carrying & Mixing of Earth			√		√
ii)	Molding Works				√	
iii)	Burning Sources:					
	Wood	√	√			
	Gas		√			
	Coal		√			
iv)	Kiln Operation & Maintenance				√	
v)	Brick Transportation to Construction Site					√

Table2:CO_2 Emission of Brick Production

Parameter	Unit	Bull's trench kiln	Fixed chimney	Zigzag kiln	Hoffman
Fuel		coal	coal	coal	gas
CO_2 emission(Ton)	Ton per lacs brick	55.4 Ton	47.51 Ton	47.51 Ton	32 Ton

Table3:CO_2 Emission & Energy Consumption for Residential Building at Dhaka, Bangladesh

(Construction phase only)

Sl. No.	Item Description	Material requirement	Standard Value Per Unit		CO_2 Emission (Ton)	Energy Consumption (GJ)
			CO2 Emission (Ton)	Energy Consumption (GJ)		
1.	Cement(Bags)	7398	0.0194	0.0935	143.53	691.71
2.	Brick (Nos.)	845222	0.00054	0.00575	456.8	4870.6
4.	Sand (Cft)	18174	0.00138	0.02346	25	426.36
5.	Rebar(Kg)	133000	0.0000624	0.001365	8.3	181.545
6.	Glass (Kg)	3500	0.0013	0.0184	4.55	64.4
7.	Lime(Ton)	3	0.47	5.69	1.41	17.1
	Total				639.59 Ton	6251.72 GJ

Table 4: Total cost analysis of Building

Sl. No.	Item Description	Project-2	Per Unit cost (maximum price)	Amount of cost (taka)	
			Cost (Taka)	Project Value	Total Project value
			Construction Materials (construction phase)		
1.	Cement(Bags)	7398	500	3699000	25162120 TK
2.	Brick (Nos.)	845222	10	8452220	
3.	Sand (Cft)	18174	100	1817400	
4.	Rebar(Kg)	133000	80	10640000	
5.	Glass (Kg)	3500	150	525000	
6.	Lime(Ton)	3	9500	28500	

RESULTS AND DISCUSSIONS

Table 2 shows the total CO_2 emission of brick production of different kiln. Brick is very energy extensive material and it emits CO_2 with high rate. Bull's trench kiln is emitted around 56 Ton per lacs of bricks. Fixed chimney is emitted around 48 Ton same as Zigzag kiln and Hoffman is emitted 32 ton which is more energy efficient but very few of them are available in Bangladesh. Energy source like coal is emitted high CO_2 and gas as a source of energy is emitted comparatively less. There is a need to use more modern technology in burning clay bricks. Table 2 shows that building using only bricks about 639.59 Tons CO_2 is emitted and energy consumption is about 6251.72 GJ from a six storied building in construction phase of 5400sft. So per sft of CO_2 emission is 0.02 Ton, energy consumption is 0.2 GJ and cost is around 800 Tk per sft. It is high time to use proper technology and make sure environments will not degrade.

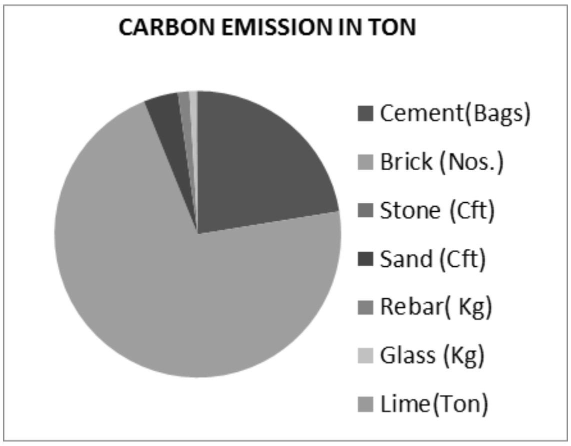

Fig.2: Total carbon emission of the Building.

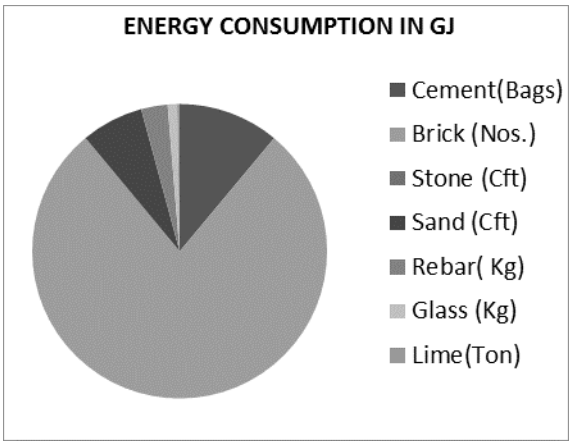

Fig.3: Total carbon emission of the Building.

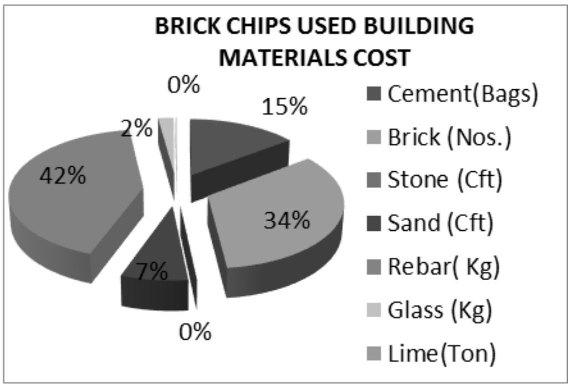

Fig.4: Details of building materials cost.

CONCLUSION

Concrete is the most common construction material used in the world. Strength of concrete is commonly considered to be most valuable properties but sustainability also important properties and using high energy consumed materials will degrade the environment. Cement is the principal ingredient in concrete. Producing one Ton of cement results in the emission of approximately one Ton of CO2 is created by fuel combustion and the calcinations of raw material. Cement manufacturing is a source of greenhouse gas emissions, accounting for approximately 7% to 8% of CO2 globally. So it is important to calculate the total CO2 of concrete industries because of their high emission of CO2 .We need a sustainable concrete industries and a sustainable industrial growth will influence the cement and concrete industry in many respects. So it is important to develop green concrete materials like green brick, green cement etc.

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TEMPORAL VARIATION OF BIOMASS CONCENTRATION IN THE BANGLADESH SUNDARBANS USING REMOTE SENSING TECHNIQUES

Ahsan Azhar Shopan[1*], G.M. Tarekul Islam[1], A.K.M. Saiful Islam[1], Md. Munsur Rahman[1], Attila N. Lázár[2] & Craig Hutton[3]

[1] *Institute of Water and Flood Management, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
[2] *Faculty of Engineering and the Environment, University of Southampton, Southampton, United Kingdom.*
[3] *GeoData Institute, University of Southampton, Southampton, United Kingdom.*
**Corresponding Author, <ahsan.shopan@gmail.com>*

ABSTRACT

The Sundarbans is the largest deltaic mangrove forest in the world. Formed at estuarine phase of the Ganges-Brahmaputra river system, the forest covers approximately 10,000 km2, of which about 6,000 km2 are in Bangladesh and the rest are in India. It plays an important role in the national economy and the demand of its resources is increasing rapidly along with the population. This paper assesses the change in biomass concentration of the Sundarbans in Bangladesh from 1980 to 2009. The biomass concentration was assessed by estimating the normalized differential vegetation index (NDVI) as there is a correlation between biomass concentration and the NDVI. The study used four Landsat satellite images covering the entire Bangladesh Sundarbans, all acquired in the month of January in 1980, 1989, 2000 and 2009 respectively. It has been found that the mean NDVI value of the mangrove forest increased continuously during 1980-2000 from 0.31 to 0.56. However, in 2009, the mean NDVI value dropped to 0.52. The top 5% of NDVI values ranged between 0.39-0.43, 0.57-0.59, 0.69-0.72 and 0.59-0.61 in 1980, 1989, 2000 and 2009 respectively. Again, the bottom 5% of NDVI values ranged between 0.10-0.20, 0.17-0.35, 0.03-0.32 and 0.16-0.34 in the respective years. These results indicate that till 2000 the biomass concentration of the Sundarbans was increasing continuously but during 2000-2009 the concentration reduced most likely due to massive deforestation. Such deforestation may cause an imbalance in the mangrove ecosystem resulting to the extinction of endangered species of the Sundarbans.

Keywords: Sundarbans, Biomass Concentration, NDVI, Landsat, Remote Sensing

INTRODUCTION

Mangrove ecosystems dominate the coastal wetlands of tropical and subtropical regions throughout the world. They provide various ecological and economical ecosystem services. At the same time, mangroves belong to the most threatened and vulnerable ecosystems worldwide and experienced a dramatic decline during the last half century. International programs, such as the Ramsar Convention on Wetlands or the Kyoto Protocol, underscore the importance of immediate protection measures and conservation activities to prevent the further loss of mangroves (Kuenzer et al., 2011).

The *Sundarbans* is the largest deltaic mangrove forest in the world. Formed at estuarine phase of the Ganges-Brahmaputra river system, the forest covers approximately 10,000 km^2, of which about 6,000 km^2 are in Bangladesh and the rest are in India. The *Sundarbans* is a complex ecosystem intersected by a complex network of tidal waterways, mudflats and small islands of salt-tolerant mangrove forests (Hussain and Acharya, 1994). It plays an important role in the economy of the south-western region of Bangladesh as well as in the national economy. In addition to traditional forest produce like timber, fuel wood, pulpwood etc., large scale harvest of non-wood forest products such as thatching materials, honey, bees-wax, fish, crustacean and mollusc resources of the forest takes place regularly (FAO, 1995). The vegetated tidal lands of the *Sundarbans* also function as an essential habitat, produce nutrients and purify water. The forest also traps nutrient and sediment, acts as a storm barrier, shore stabilizer and energy storage unit. However, the demand of its resources is increasing rapidly along with the population and thus, as other mangroves, the *Sundarbans*, is also under threat.

The trophic relationships between mangroves and coastal ecosystem can be characterized by the biomass and productivity of the mangrove forests (Hess et al., 1990). Biomass is biological material derived from living, or recently living organisms. In the context of biomass for energy this is often used to mean plant based material. However, biomass and productivity data of mangrove forests are scarce, mainly because of the difficulties associated with field measurements. In this context, remote-sensing techniques have demonstrated a high potential to detect, identify, map, and monitor mangrove conditions and changes, which is reflected by the large number of scientific papers published on this topic.

Remote sensing from satellites is economically competitive with other forms of data collection, such as aerial photography, especially where low or moderate resolutions are adequate. Broad swath widths and the advent of high-resolution systems enable frequent repeat coverage of targets. Systems can also collect data over denied or hazardous areas without interruption. Satellite sensors not only observe the earth in visible light, but also in the infrared region and with microwaves. Spatial resolution ranges from more than one kilometer to less than one meter (Karmaker, 2006).

This paper assesses the change in biomass concentration of the *Sundarbans* in Bangladesh from 1980 to 2009 using remote sensing techniques. The scope of the study is limited to the Bangladesh part of the *Sundarbans* only, which is situated between 21°40’00’’N to 22°30’00’’N and 89°00’00”E to 89°50’00”E. It mainly belongs to 3 districts of the Khulna division of Bangladesh namely: Satkhira, Khulna and Bagerhat. Howerver, a minor part of it also belongs to the Barguna district of Barisal division.

METHODOLOGY

In this study, the biomass concentration was assessed by estimating the normalized differential vegetation index (NDVI) as there is a correlation between biomass concentration and the NDVI. NDVI is an index of plant “greenness” or photosynthetic activity, and is one of the most commonly used vegetation indices. By taking the ratio of red and near infrared bands from a remotely-sensed image, an index of vegetation “greenness” can be defined. Vegetation indices are based on the observation that different surfaces reflect different types of light differently. Photosynthetically active vegetation, in particular, absorbs most of the red light that hits it while reflecting much of the near infrared light. Vegetation that is dead or stressed reflects more red light and less near infrared light. Likewise, non-vegetated surfaces have a much more even reflectance across the light spectrum. NDVI is calculated on a per-pixel basis as the normalized difference between the red and near infrared bands from an image using the following equation:

$$NDVI = \frac{(NIR - RED)}{(NIR + RED)} \qquad (1)$$

Where, NIR is the near infrared band value for a cell and RED is the red band value for the cell. Many factors affect NDVI values like plant photosynthetic activity, total plant cover, biomass, plant and soil moisture, and plant stress. Because of this, NDVI is correlated with many ecosystem attributes. Also, because it is a ratio of two bands, NDVI helps compensate for differences both in illumination within an image due to slope and aspect, and differences between images due things like time of day or season when the images were acquired. Thus, vegetation indices like NDVI make it possible to compare images over time to look for ecologically significant changes.

The study used four Landsat satellite images covering the entire Bangladesh *Sundarbans*, acquired in 1980, 1989, 2000 and 2009 respectively. The images have been downloaded from webpage of United

States Geological Survey (www.earthexplorer.usgs.gov). These images are taken at four tiles: Path of 137 with Row of 44, Path of 137 with Row of 45, Path of 138 with Row of 44 and Path of 138 with Row of 45, which cover the entire study area. The images represent dry season of Bangladesh as all of them have been captured in the month of January. It is assumed that temporal changes of water bodies remain insignificant over this period.

Properties of the images are presented in Table 1. It should be mentioned that the TM (Thematic Mapper) sensor has a spatial resolution of 30 m for the visible, near-IR, and mid-IR wavelengths and a spatial resolution of 120 m for the thermal-IR band. The ETM+ (Enhanced Thematic Mapper Plus) has spectral bands which are similar to those of TM, except that the thermal band (band 6) has an improved resolution of 60 m. However, all TM/ETM+ images are now resampled to 30 m resolution by the production system. On the other hand, the downloaded MSS images had a spatial resolution of 60 m which were resampled to 30 m during analysis for convenience of the study.

Table 1: Properties of Landsat satellite images

Image No.	Acquisition Date	Satellite	Sensor	Spatial Resolution (m)
1	January, 1980	Landsat 3	MSS	60
2	January, 1989	Landsat 4	TM	30
3	January, 2000	Landsat 7	ETM+	30
4	January, 2009	Landsat 5	TM	30

The images were analyzed using open-source remote sensing software ILWIS 3.4. Firstly, the images were manually digitized along the border of the *Sundarbans* to separate it from the whole image. It was then classified into several categories such as mangroves, mudflats, and waterways using maximum likelihood classification. The waterways and mudflats were then excluded from the images and the mangroves area was further analyzed to determine the changes in biomass concentration of the *Sundarbans* over the period of 29 years by preparing NDVI maps of the *Sundarbans* for 1980, 1989, 2000 and 2009 and compare them to determine the intensity of deforestation at different parts of the Bangladesh *Sundarbans*.

RESULTS AND DISCUSSIONS

The NDVI maps of the *Sundarbans* at different years are presented below (Figure 1). Minimum, maximum, mean, median and mode values of NDVI are compared in Figure 2. Table 2 also shows the standard deviation, range and skewness of the distribution of the NDVI values. It has been found that the mean NDVI value of the mangrove forest increased continuously during 1980-2000 from 0.31 to 0.56. However, in 2009, the mean NDVI value dropped to 0.52. Again, the maximum NDVI value increased from 0.43 to 0.72 over the period of 1980-2000 but reduced to 0.61 in 2009. However, the minimum NDVI value changed inconsistently throughout the period. These results indicate that till 2000 the biomass concentration of the *Sundarbans* was increasing continuously but during 2000-2009 the concentration reduced most likely due to massive deforestation. From the comparative spatial analysis of the NDVI maps of 2000 and 2009, it has been found that the southwest region of the Bangladesh *Sundarbans* is affected the most in terms of biomass degradation. Figure 3 shows the areas where NDVI value has decreased by more than 0.15 from 2000 to 2009.

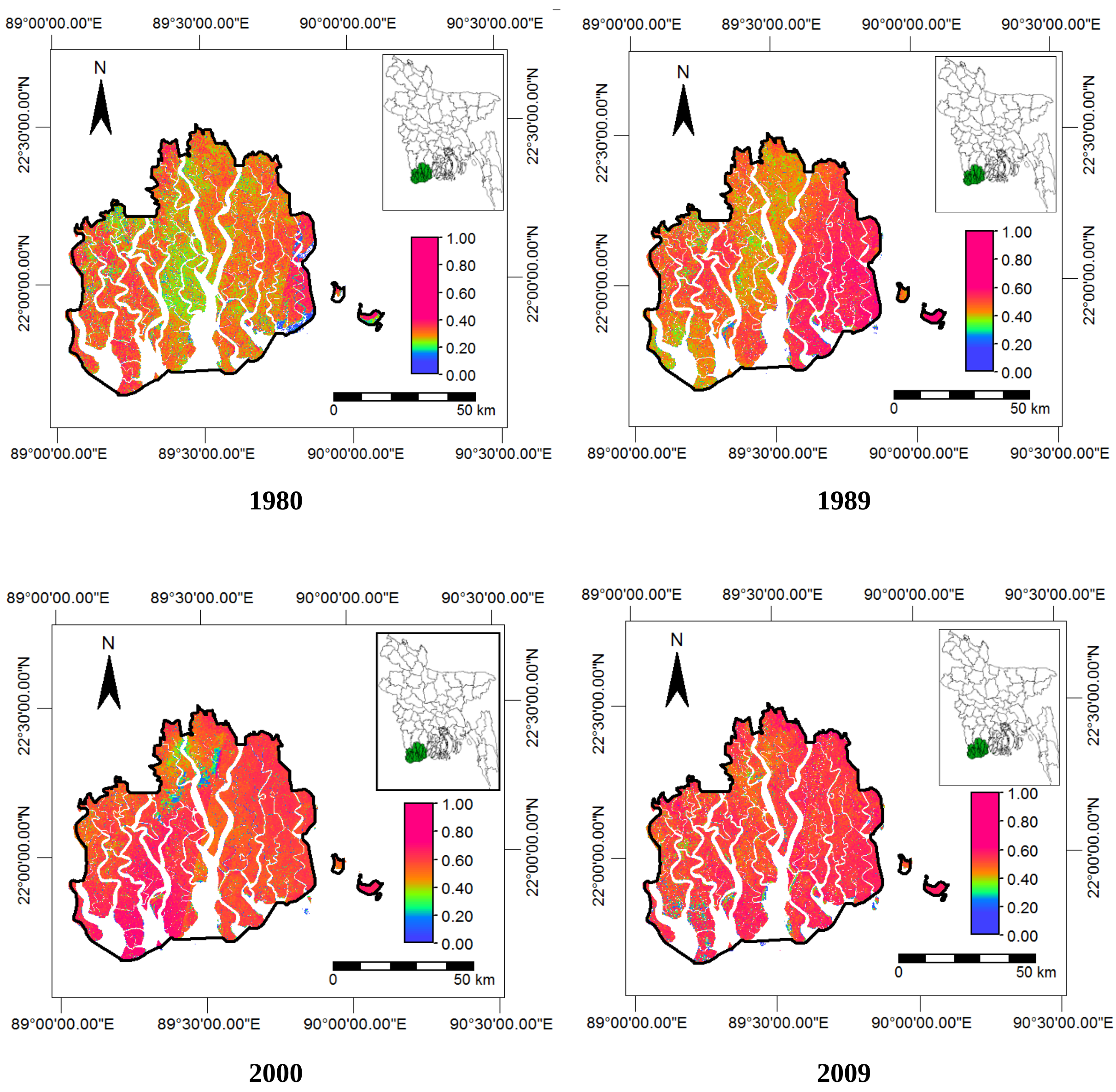

Figure 1: NDVI maps of the *Sundarbans* at different years

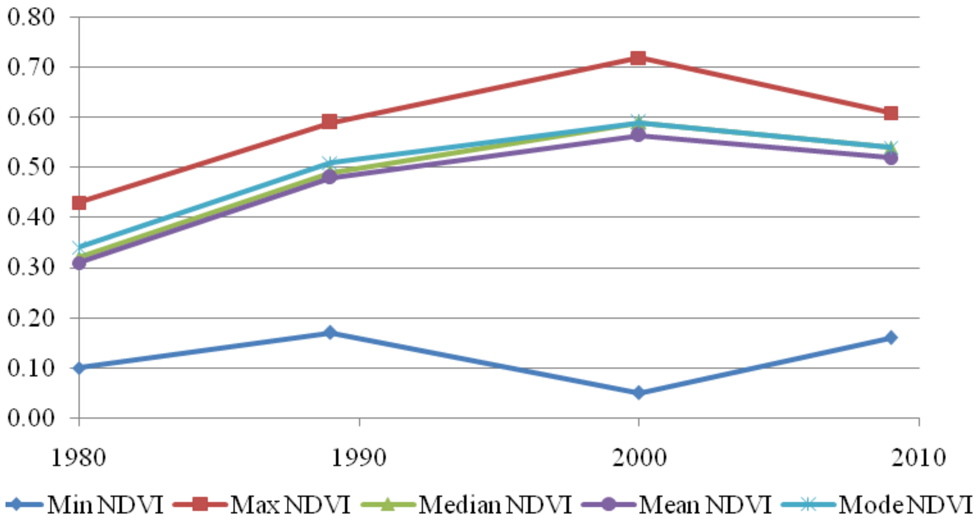

Figure 2: Comparison of NDVI values at different years

Table 2: NDVI value properties at different years

NDVI	**1980**	**1989**	**2000**	**2009**
Min	0.10	0.17	0.05	0.16
Max	0.43	0.59	0.72	0.61
Median	0.32	0.49	0.59	0.54
Mean	0.31	0.48	0.56	0.52
Mode	0.34	0.51	0.59	0.54
Std Dev	0.01	0.01	0.01	0.02
Range	0.33	0.42	0.67	0.45
Skewness	Negative	Negative	Negative	Negative

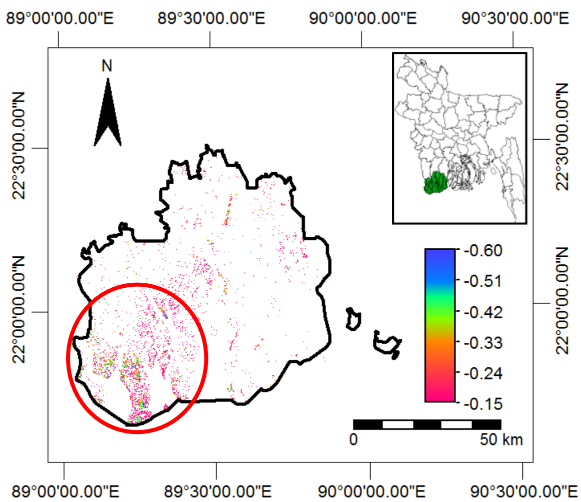

Figure 3: Biomass degradation at the Bangladesh *Sundarbans* during 2000-2009

CONCLUSION

Forest degradation has become a serious issue, especially in developing countries. In 2000, the total area of degraded forest in 77 countries was estimated at 800 million hectares (ha), 500 million ha of which had changed from primary to secondary vegetation (ITTO, 2002). Among other impacts, the process of forest degradation represents a significant proportion of greenhouse gas emissions. Thus, there is an urgent need to measure and analyze it, in order to design action to reverse the process. NDVI, despite of its limitations has been proved to be very useful to measure forest degradation. This study clearly indicates that the biomass concentration at the Bangladesh *Sundarbans* has decreased significantly since 2000 most likely due to massive deforestation. Such deforestation may cause an imbalance in the mangrove ecosystem resulting to the extinction of endangered species of the *Sundarbans*. Thus, extensive studies must be carried out to determine the exact causes of biomass degradation at the *Sundarbans* as this study does not focus on this subject. Moreover, latest images can be analyzed to further verify the findings of the study.

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STUDY ON THE WATER CONSUMPTION BEHAVIOR IN KHULNA CITY

S. M. Moniruzzaman[1], M. H. Mishuk[1*], Q.S. Sayeed[2] & Md. H. Rahaman[1]

[1] *Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.*
[2] *Department of Civil Engineering, Stamford University Bangladesh, Dhaka -1217, Bangladesh.*
**Corresponding Author, <mehedihasanmishuk@yahoo.com>*

ABSTRACT

Water is indispensable for all the living being in the world. Sufficient quantity and required quality of water is a major concern for drinking and other purposes. Khulna is the third largest city in Bangladesh. Inadequate quantity and poor quality of water is a major problem in Khulna city. The present water supply system of Khulna Water Supply and Sewerage Authority (KWASA) is not appropriate for the city dwellers. There is neither supply of water for all the people from KWASA nor have sufficient number of safe, salt-free and reliable sources of water in Khulna. This study investigates the present water consumption behavior in Khulna city. A questionnaire survey was conducted in the study area of Khulna city for collecting necessary data. The questionnaire survey form was prepared based on some important characteristics of the city dwellers. A total number of 200 persons were interviewed in Khulna City Corporation (KCC) area. The main objectives of this study are to investigate different available sources of water, survey the trend of the city dwellers for selection of water sources for different household chores, investigate the diseases and inconvenience they face while using the available water in Khulna. The arranged and sorted out data were analyzed by Service Product for Statistical Solution (SPSS) software to establish relationships among dependent and independent variables by linear regression, cross tabulation and correlation analysis. Finally, a bunch of suggestions was provided to fulfil the water supply necessity of the city.

Keywords: Water Consumption, questionnaire survey, linear regression, cross tabulation

INTRODUCTION

Domestic fresh water is a fundamental requirement for human welfare and economical activities. Human survival and welfare generally depend on regular availability and control of water. The paradox of community water supply in developing countries is that everyone has access to water supply, however in fact many people do not. They may access to water, but only a large walking distance, in too little volume or of poor quality. A lot of effort is made in the world to change this situation; however is this situation really changing?

The water consumption pattern is a very important parameter generally used in estimation of water consumption in a certain periphery. The amount of consuming water needed daily, depends on various factors such as temperature, gender, region, community pattern, stage of production, health, etc. Information on the water consumption pattern of a population is essential in order to estimate the risk of adverse health effects attributable to any water contaminant. It is also crucial in developing a safe water supply program for a population. As a common characteristic of the urban areas of Bangladesh, accessibility to the suitable quantity and quality of water is a major problem in the city areas of Khulna. Unplanned and improper withdrawal of water is the main reason of the shortage of water. These shortages become more acute during the dry seasons. The ground water withdrawal and recharge can be increased, approaching the potential limits by creating additional storage through increased during dry season (Ahmed and Rahman, 2000). People in these areas are in such condition that they neither have a supply of water from Khulna Water Supply and Sewerage Authority

(KWASA), nor have sufficient number of safe and reliable sources of water in their surroundings. There is also less availability of sources that provides salt-free water in Khulna city areas (Mohsin, 2007).

The greater Khulna region is very close to the Bay of Bengal. Due to the geological position of Khulna, the water in the ground exhibits high salinity. There is a scarcity of water even for drinking and in the city areas of Khulna people are sometimes bound to use the saline water for drinking and household purposes as the non-saline water they get from surrounding sources is not sufficient to meet their demand. The consumptive use of saline water causes lots of inconvenience to the users. The augmentation of salt water in the surface waters increases the abstraction of groundwater, which then becomes vulnerable to a risk that salt water will be drawn into the aquifer (Md. M. Rahman & A. K. Bhattacharya, 2006). There are some people who are in better economic condition and well educated are using water from alternative sources for drinking, whereas a lot of people are beneath the poverty line and some people are not well aware of health and hygiene are using the water they can collect from the nearby water sources. It is a matter of great concern that there is no water supply in the city areas of Khulna. That is why the water consumption pattern analysis is necessary to find the quality and quantity of water people are using for different household purposes.

The social status, economic conditions, level of education, age and experience of people are the factors affecting the selection of sources for water. Availability of fresh water is another factor that has significant influence on it. Factors affecting higher probabilities of a respondent being primarily a bottle water drinker included: higher income, unpleasant taste experiences with tap water, non French-speaking and being a male with children in one's household (Dupont et al., 2010). The objectives of the study are:

1. To investigate of different available sources of water in the city areas of Khulna.

2. To survey the local people about their selection of water sources for different household uses.

3. To investigate about the disease and the inconvenience they face while using the available water.

4. To find out the correlation between the awareness of people while choosing a source of water and the occupation, experience, authority, maturity, gender, socioeconomic condition, level of education of local people using the SPSS (Statistical Package for the Social Sciences)

5. To analyze the change of trends in the selection of water sources with the changes of the influencing factors.

METHODOLOGY

The methodology of this study includes:

1. Collection of Information: Information has been collected about the existing condition of sources of water in the areas of KCC, the required information was provided by Khulna WASA.

2. Preparation of a Questionnaire Survey Form: A questionnaire survey form has been prepared, including all the inquiries required for the analysis of this study.

3. Questionnaire Survey: Surveys of local people have been performed to collect the required data. The survey was performed based on the factors influencing people's selection of water sources. 200 participants were surveyed from the study areas.

4. Data Sorting: The collected data have been arranged and sorted out for the analysis.

5. Analysis: The analysis for determining the correlations have been done by SPSS 16.0 software through the linear regression analysis method. The correlation matrix is formed by a Bivariate correlation method with Pearson correlation coefficients.

Study area

The study area is very important in this study. This study is not based on a particular area in Khulna city. The areas are selected from the different wards, so that it focuses on the whole city area. The research work is to be carried out in Khulna city as shown in Table 1.

Table 1: Study area of Khulna city

Ward No	Name of the Area
16	Sonadanga, Nurnagar
18	Banergati
20	Sheikhpara
22	Munshipara
24	Nirala Residential Area, Bagmara
26	Baniakhamar, Bosupara
29	Haze Mohsin Road
30	Totpara Hospital Road, Khan Jahan Ali Road.

Factors influencing people's selection of water sources

Although the human mind and the choice are unique, there must be some factors that have an influence on the selection of water sources for different uses. Availability of proper water source come in the list. The factors are age, gender, occupation, economic condition, authority, family members, level of education.

RESULTS AND DISCUSSION

Regression analysis in drinking water sources

The output of the model (SPSS software) for drinking water sources are shown in Table 2 and Table 3.

Table 2: Model summary of drinking water sources

Model	**r**	**r^2**	**Adjusted R square**	**Std. Error of the Estimate**
1	0.479[a]	0.229	0.210	0.70392

predictors: (constant), physician cost (PC), detergent (D), color (C), family members (FM), education(E)

Coefficient of Correlation, r = 0.479

Coefficient of determination, $\mathbf{r}^2 = 0.229$

Table 3: Coefficients of drinking water sources

Model		Unstandardized Coefficient		t	Sig.
		B	Std. Error		
1	(constant)	2.162	0.386	5.603	0.000
	Family members	-0.327	0.073	-4.461	0.000
	Detergent	0.244	0.108	2.250	0.026
	Color	0.536	0.103	5.191	0.000

Education	-0.150	0.049	-3.097	0.002
Physician cost	0.165	0.072	2.278	0.024

dependent variable: drinking

The equation formed by the co-efficient is given by

Drinking water source = 2.16-0.327*(FM) +0.244* (D) +0.536* (C) -0.150*(E)+0.165*(PC) (1)

The results from the Eq. (1) will near 2. In the most cases, as 1 represents the water source is a tap water and 2 denotes the water source is a tube well. For a particular household for which the participant's family members is 2-4 (1), detergent (1), color (1), education graduate(6), physician cost 100-500(1); the result of drinking water source becomes 1.878 which means the household uses a tube-well water for drinking.

Regression analysis in main water sources

The output for main water sources are shown in Table 4 and Table 5.

Table 4: Model summary of main water sources

Model	**R**	**r^2**	**Adjusted R square**	**Std. Error of the Estimate**
1	0.597^a	0.357	0.340	0.33732

Predictors: (constant), Detergent(D), Color(C), Family members(FM), Diseases(Di), Red(R)
Coefficient of Correlation, r = 0.597

Coefficient of determination, $\mathbf{r}^2 = 0.357$

Table 5: Coefficients of main water sources

Model		Unstandardized Coefficient		t	Sig.
		B	Std. Error		
1	(constant)	1.952	0.157	12.457	0.000
	Family members	0.134	0.034	3.918	0.000
	Detergent	-0.410	0.051	-7.967	0.000
	Red	0.069	0.017	3.975	0.000
	Diseases	-0.048	0.015	-3.250	0.001
	Color	-0.116	0.051	-2.292	0.023

Dependent Variable: main sources

The equation formed by the coefficient is given by

Main water source = 1.952+-0.134* (FM) -0.410*(D)+0.69*(R)-0.048* (Di) +0.116*(C) (2)

The results from the Eq. (2) will represent whether the household selects a tube-well water source or tap water as their main source. If the value of the equation is closes to 1, it will indicate the tube-well and, if the value is close to 2 it will indicate tap water as their main sources. For a particular household for which the participant's family members 2-4 (1), detergent (1), red (1), diseases (1), color (1); the result for main water source becomes 1.341 that is closer to 1 which means the main water source is tube-well.

Percentile selection about consuming water

The selection of different water sources is shown in Table 6.

Table 6:Selection of water sources

Water Sources	Drinking		Cooking		Bathing and Washing	
	Frequency, f	Percentage,%	Frequency,f	Percentage,%	Frequency,f	Percentage,%
Tube-well	159	79.5	120	60	138	69
Filtrate water	40	20	0	0	0	0
Bottle water	1	0.5	0	0	0	0
Pond/river	0	0	3	1.5	9	4.5
Tap water	0	0	77	38.5	53	26.5

The selection of medicine and physician cost per year for different diseases are shown in Table 7.

Table 7: Selection of medicine and physician cost per year for suffering diseases

Diseases	Frequency, f	Percentages, %	Average cost per year (TK)
No diseases	95	47.5	
Dysentery	100	50	1100
Diarrhea	5	2.5	

Education and drinking cross tabulation

Table 8: Variation of drinking water source with education level

Count		Drinking			Total
		Tap Water	Tube well Water	Filtrated water	
Education	Only signature	4	5	0	9
	Primary	1	5	3	9
	SSC	0	28	10	38
	HSC	0	41	9	50
	Graduate	0	79	15	94
Total		5	158	37	200

The variation of drinking water sources with education level is shown in Table 8. Some survey participants below SSC use tap water for drinking purposes. But the educated people (HSC and graduate) use tube well water and filtrated water for the drinking purposes. The more education level is increased, the more people are interested in using filtrated water as shown in Table 8.

Correlation matrix of dependent and independent variables

The relationship between dependent and independent variables are shown in Table 9. The Correlation between two variables represents how the variables are related to each other and in which way one variable will respond to any change of the other. The optimistic value of correlation is 1. If correlation is greater than 0.5 then the relation between two variables is considered better.

Table 9: Correlation matrix

Variable	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1. Age	.081														
2. Gender	.41	1													
3. Occupation	-.574	.402	1												
4. Economic condition	.176	.193	-.236	1											
5. Family members	.071	-.051	-.10	-.193	1										
6. Education	.151	-.076	.089	-.222	-.308	1									
7. Drinking water source	-.115	.209	.164	.033	-.292	.018	1								
8. Cooking water source	-.058	.163	.019	-.097	-.145	.004	.18	1							
9. washing water source	.11	.177	.044	-.104	-.081	.159	.149	.687	1						
10. Amount of drinking water	.037	.059	-.207	.052	.137	.111	-.202	.038	-.01	1					
11. Taste	-.254	.100	.079	-.002	.251	-.216	-.146	-.173	-.19	.121	1				
12. Regularity	.041	-.017	-.030	.067	.065	.054	-.114	-.098	-.04	-.21	.394	1			
13. All seasons	-.074	-.132	.161	-.11	-.023	.065	-.097	-.002	-.01	.032	.103	.068	1		
14. Diseases	-.074	-.025	-.270	.195	.198	-.175	-.110	.054	.14	.116	-.10	-.16	-.37	1	
15. Satisfaction	.087	-.158	.105	-.038	-.008	.150	.008	.054	.05	-.11	.20	.083	.149	-.40	1

CONCLUSIONS AND RECOMMENDATIONS

The major conclusions and recommendations drawn from the study are as follows:

For drinking purposes, 79.5% people use tube-well water, 20% use filtrate water, 0.5% people use bottle water and for cooking, 60% people use tube-well water, 38% people use tap water.
For bathing and washing purposes, 69% people use tube-well water, 26.5% use tap water.

By using water, 50% people suffer from dysentery, 2.5% people from diarrhea and others are safe. Each family spends average 1100 TK per year for medicine and physician cost.
It was found that, the survey participant's level of education, age and economic condition significantly influences the selection of water sources than other factors.
The deep aquifers should be the primary water source to be used by KWASA in the peripheral areas of Khulna city. In addition, uncontaminated surface water sources may also be a source of supply after applying proper treatment.
During rainy season, the rain water should be harvested as much as possible. Use of rain water ensures rapid fulfillment of ground water aquifers and slow depletion rate of Ground Water Table when the dry season comes. As salinity is the governance problem in the study area, solar desalination of water may be useful.

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A STUDY ON EFFECT OF INDUSTRIAL EFFLUENTS ON SURROUNDING ENVIRONMENT (WATER BODY)

Fahmida Parvin[1*], M.A. Yusuf[1], O. A. Chowdhury[1], M.A. Rahman[1] & Indrajit Barua[1]

[1] *Department of Civil Engineering, University of Information Technology and Sciences (UITS), Chittagong, Chittagong-4225, Bangladesh,*
**Corresponding Author, <lipi.parvin@gmail.com>*

ABSTRACT

Water is the most vital element among the natural resources. The environment, economic growth and development of Bangladesh are all highly influenced by water, its regional & seasonal availability, and the quality of surface & ground water. It is needless to say that without enough quality water our survival will be threatened. Quality water is a great challenge for 21st century and is more essential than its quantity. Water quality is deteriorated day by day due to numerous of biological, physical and chemical variables causing water toxicity. In terms of quality, the surface water of the country is highly affected due to untreated industrial effluents.

To study about the above, waste water sample have been collected from two major types of industries; Jute Mill & Ready Mix Concrete (RMC) Factory and tested them for physical, chemical, biological parameters; pH, TDS, TSS, BOD, COD, Temperature. Also the samples are tested for heavy metal; Cr, Fe, Ni, Cu, Pb& then the result has been compared with National Environmental Quality Standards (NEQS) for river water. In most of the cases, tested result value found much higher (e.g.TSS, BOD) than standard value. So, it can presume that, treatment is compulsory at source.

Keywords: Environment, Industrial Effluent, Water Quality Parameters, Heavy metals, NEQS.

INTRODUCTION

Water is essential to all forms of life and makes up 50-97% of the weight of all plants and animals and about 70% of human body. Water is also a vital resource for agriculture, manufacturing, transportation and many other human activities. Despite its importance, water is the most poorly managed resource in the world (Anon,1992).

The availability and quality of water always have played an important role in determining the quality of life. Water quality is closely linked to water use and to the state of economic development (Kulkarni, G. J. 1997). Ground and surface waters can be polluted by several sources. In urban areas, the careless disposal of industrial effluents and other wastes may contribute greatly to the poor quality of water (Mathuthu et al., 1997). Most of the water bodies in the areas of the developing world are the end points of effluents discharged from industries.

Wastewater is the combination of liquid raw water-transported waste which has originated from residential dwelling, commercial or industrial facilities and institutions. It also comprises ground water, surface water and rain or storm water that may be present at the time. Industrial wastes are those wastes arising from industrial activities and typically include rabbis, ashes, demolition and construction wastes, special wastes and hazardous wastes. Raw waste water usually comprises of high level concentration of organic materials numerous pathogenic microorganisms and also numerous nutrients toxic components.

Industrialization plays a vital role in growth and development of any country. This rapid industrialization is also having a direct and indirect adverse effect on our environment. Industrial development manifested due to setting up of new industries or expansion of existing industrial establishments resulted in the generation of industrial effluents, spatially small scale cottage industries which discharge untreated effluents which cause air, water, soil and soil solid waste pollution. This is considered as a global problem because of its adverse effects on human health, plants and animals.

Wastewater treatment provides an essential community service that is vital for the protecting of public health and the environment. Without affordable water and wastewater services, economic growth and the quality of life are diminished. Most cities, towns communities in our country provide drinking water and wastewater treatment industry faces a number of challenges, including urban population growth, the need to treat wet weather flows, more stringent discharge regulations, and demand for water conservation through wastewater reuse.

MATERIALS AND METHODS

Industry visit & Sampling

For the assessment of the effluent, total ten samples were collected from ten different industries i.e one sample from each. All samples were collected from the outlet of allied industry and with a clean dry plastic jar of 1000 ml & tightly closed. The sample was stored and labelled with sample ID, date and location on the sticker tag and kept protected from direct sunlight and then taken to the laboratory for analysis. Necessary precautions have been taken during carrying of sample. In **Table 1** is shown that name of the sample according to the industries and location of the industries in Chittagong.

Table 1: Sampling of waste water

Name of the Sample	Name of Industry	Location
Sitakunda Area-Jute Mill		
SJ1	Kashem Jue Mill	Shitolpur
SJ2	Hafiz Jute Mill	Bar-Awlia
SJ3	Gul Ahmed Jute	Baro Kumira
SJ4	M M Jute Mill	Banshbaria
SJ5	R R Jute Mill	Banshbaria
Nasirabad Industrial Area-Ready Mix Concrete(RMC)		
SR1	Concord RMC	Rahman Nagar
SR2	Eternal RMC	Chandra Nagar Bazar
SR3	SABL RMC	Rahman Nagar
SR4	CEM RMC	Polytechnic
SR5	EPIC RMC	Foy's Lake

Sample Analysis

Waste water collected from the study area was analyzed in the laboratories of Analytical Research Division, Bangladesh Council of Scientific and Industrial Research (BCSIR) Chittagong. The following physicochemical parameters were analyzed :

Temperature, pH, Total suspended solids (TSS), Total suspended solids (TSS), Biological oxygen demand (BOD), Chemical Oxygen Demand(COD), Heavy metals analysis: Copper (Cu), Iron (Fe), Nickel (Ni), Led (Pb) and Chromium (Cr).

RESULTS

The tested result found from laboratory is presented here in tabulated form and compared with National Environmental Quality Standards (NEQS) for river water. In tabulated form, physicochemical parameters & Heavy metals analysis results are shown in Table 2. For some constrain only graphical presentations of Total suspended solid, Biochemical oxygen demand & Chromium are shown in Fig.1, Fig.2 & Fig.3 respectively.

Table 2: Results of the physicochemical parameters & Heavy metals of the collected sample with NEQS value

SL. No.	Sample Description	Temp °C	pH	TSS (mg1/l)	TDS (mg1/l)	BOD (mg1/l)	COD (mg1/l)	Cr (mg/l)	Ni (mg/l)	Cu (mg/l)	Fe (mg/l)	Pb (mg/l)
1	Kashem Jute Mill(SJ1)	30	7.8	160	1500	50	60	0.85	1.20	0.80	0.95	0.53
2	Hafiz Jute Mill(SJ2)	35	5.5	350	2100	300	450	1.25	2.05	1.05	1.25	1.10
3	Gul Ahmed Jute Mill (SJ3)	26	7.5	425	5000	385	510	3.0	3.0	2.9	2.28	5.6
4	M M Jute Mill (SJ4)	33	6.8 5	650	5500	420	585	1.65	1.45	1.40	1.15	1.50
5	R R Jute Mill (SJ5)	29	6.0 2	275	3825	550	650	1.85	1.50	1.10	1.50	1.75
6	Concord RMC (SR1)	26	6	185	5230	125	175	4.45	2.85	2.0	1.55	2.10
7	Eternal RMC (SR2)	32	7.5	350	4125	130	185	3.0	2.6	1.85	2.00	3.1
8	SABL RMC (SR3)	25	8.0	650	5870	145	240	3.15	4.15	2.90	2.24	4.8
9	CEM RMC (SR4)	30	6.1 5	445	4250	135	225	1.98	3.75	1.95	2.25	4.5
10	EPIC RMC (SR5)	20	7.1	530	3615	120	165	3.02	4.55	1.50	2.50	4.2
11	NEQS	35	8.4 8	150	3500	80	150	1.0	1.0	1.0	1.0	0.5

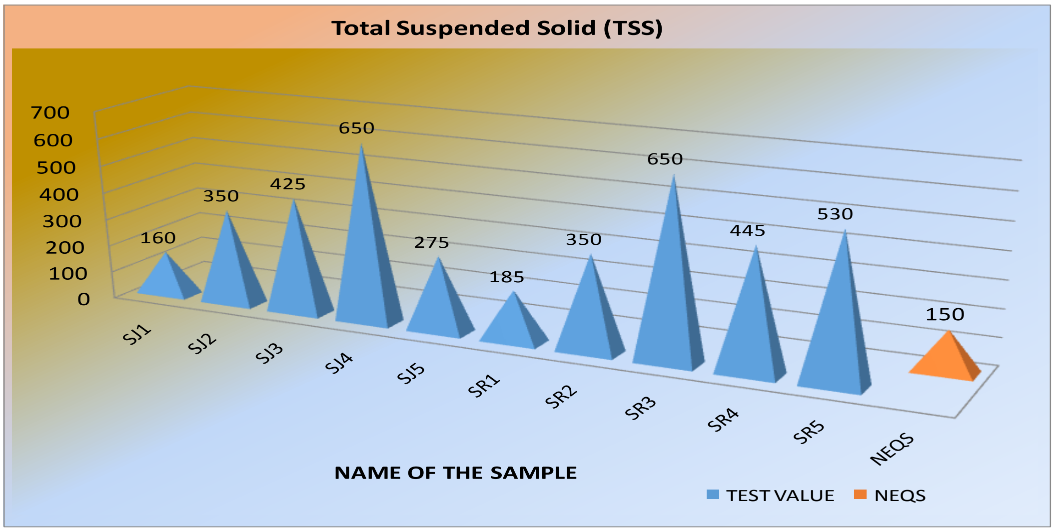


Fig.1:Comparison between laboratory test results & NEQS values for TSS

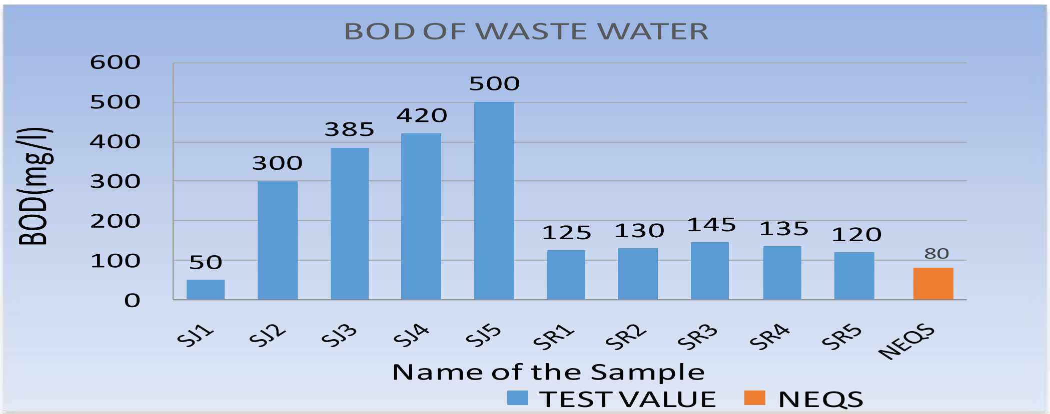
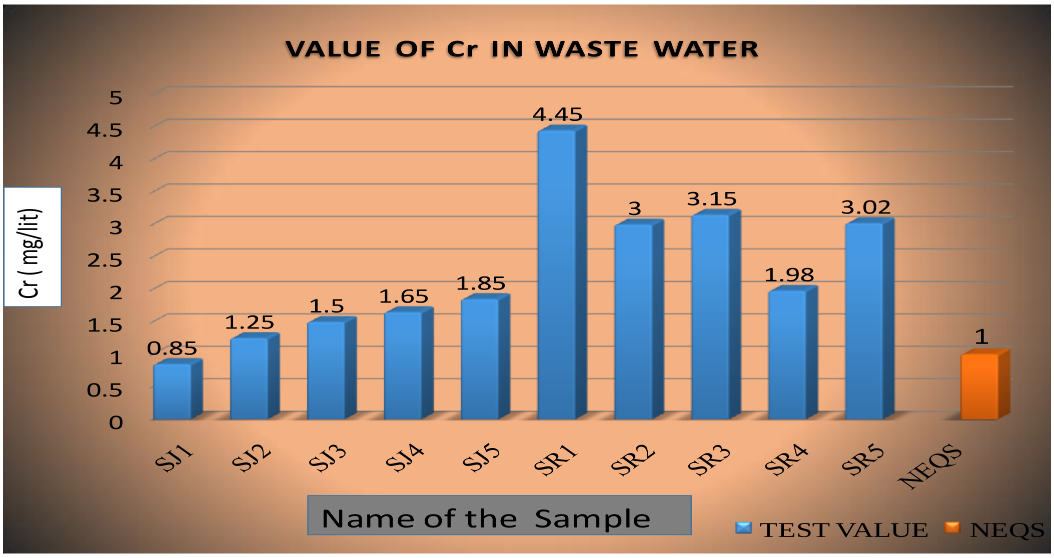


Fig.2:Comparison between laboratory test results & NEQS value for BOD



DISCUSSIONS

From the analysis of wastewater sample collected from different industry, it has been observed that almost all parameters for most industries exceed National Environmental Quality Standards (NEQS) for river water. Only temperature and pH value of all wastewater samples are found within permissible limit. The TSS values of the samples ranged from160-650 mg/l which exceeds the standard value. Effluents of such high TSS may cause handling problem, if directly applied to agricultural field or if this effluent is discharged to river or stream, it will make unsuitable for aquatic life. The BOD values ranged from 50 to 500 mg/l as presented which also exceeded. These effluents on entering freshwater (rivers, stream etc) make the O_2 depleted, causing suffocation of fish and other aquatic fauna and flora resulting in the death of aquatic life. It is noted that, all of the samples result of chromium except the value of Kashem Jute Mills are above than NEQS value (1.0 mg/lit). All samples have Nickel value above the permissible limits. This is really alarming for our environment. Cu concentration of the samples ranged from 0.8-2.90 mg/l. Only the sample of SJ1 has value with in permissible limit. The Iron (Fe) concentration of the samples ranged from 0.95- 2.50 mg/l. Except sample SJ1, all other samples have values above the permissible limits of NEQS value (1.0mg/l). Lead concentration in the waste water sample found value higher than NEQS value.

CONCLUSION & RECOMMENDATION

The amount of waste released into environment is increasing, continuously with significant increase in human activities. Industrial effluent has been and continues to be a major factor causing the degradation to the environment around us, affecting the water we use, the air we breathe and the soil we live on. Lack of awareness, treatment facilities & financial resource and inefficient environmental laws are aggravating the crisis. Industrial effluents contain chemicals and biological matter that impose high demands on the oxygen present in water. Thus contains low levels to dissolved oxygen as a result of the heavy biological Oxygen Demand (BOD) and Chemical Oxygen Demand (COD) placed by industries. It may conclude here that, Treatment of industrial effluent is the ultimate solution. These can be possible by adopting low cost treatment like Dissolved Air Flotation (DAF) for Physic-chemical parameter and Chemical Precipitation for heavy metal etc. In addition, it is necessary to have educated operations to run the treatment facilities. Local and other authorities should provide more strict regulations and directives. The authority should take much interest to imply the ETP as early as possible. It is necessary to locate defined storage area. The ultimate disposal point of liquid effluents should be improved. If, any filter media is used then suspended solid as well as dissolved solid will be removed in higher portion. This will make the effluent with low turbidity. As for example if the effluent in run through a large sedimentation tank then major portion of suspended solid will be settled down. The authority of the industry should take many steps to reduced bad quality of waste water. In future study, for more effective industrial effluent treatment system, the use of chemicals in several unit of production may be controlled to reduce the load on ETP.

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ASSESSMENT OF CAPCITY RELIABILITY OF NEWLY CONSTRUCTED SPECIAL SEWAGE DIVERSION SYSTEM AT HATIRJHEEL, DHAKA

R. Biswas[1*], M. A. Ali[1]

1 Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh,
**Corresponding Author, <rudrakshi.buet@gmail.com>*

ABSTRACT

Hatirjheel, which is now the largest surface water body within Dhaka, serves very important hydrologic functions of draining and detaining storm water from a large area of Dhaka city. As a part of a restoration project to save Hatirjheel from pollution and degradation, eleven especially designed sewage diversion structures (SSDSs) have been constructed at eleven major storm sewer outfall locations around Hatirjheel. The SSDSs are connected to a peripheral "diversion sewer" network. Although designed to carry storm water, the storm sewers discharging into Hatirjheel carry both storm water and domestic sewage. During the dry season, the entire flow (consisting of sewage) carried through the storm sewers up to Hatirjheel are diverted downstream along the peripheral "diversion sewer". The SSDSs are designed to allow overflow of storm water into Hatirjheel during wet season. However, since the storm sewers carry both storm water and sewage, the overflows discharging into Hatirjheel during wet season are in fact mixtures of storm water and sewage; this is the main reason for the deterioration of water quality of Hatirjheel. Until the storm sewers are freed from illegal domestic sewer connections, this situation will continue. In this study, the capacity of the peripheral main diversion sewer network of Hatirjheel to carry the estimated/projected wastewater flows in the future has been estimated. It has been found that the main diversion sewer system may not be able to accommodate the increased sewage flow in the near future (2015 and beyond).The water quality situation is likely to deteriorate further in the future as sewage flows increase with increasing population density, unless immediate steps are taken to separate storm sewer and domestic sewer networks.

Keywords: Special Sewage Diversion Structures, storm water, waste water.

INTRODUCTION

The low-lying areas behind Sonargaon Hotel and those of Hatirjheel, stretching from the eastern side of Tongi-Diversion Road up to the Rampura Bridge on the Progati Shwarani, serve very important hydrological functions of draining and detaining storm water from a large area of Dhaka city. Gradual encroachment and illegal filling of the lowlands of Hatirjheel over the years was a major threat to the already vulnerable drainage situation of Dhaka City (BRTC, 2005a; 2005b). The Hatirjheel lowlands receive storm water primarily through a number of major storm sewer outfalls. However, illegal connections of both domestic and industrial wastewaters to the storm sewer network are rampant in Dhaka. During dry season, the storm sewers carry domestic mainly sewage as well as industrial wastewater (BRTC, 2006). As a result, Hatirjheel was turned into a virtual wasteland and contributed to deterioration of water quality in Begunbari Khal-Balu-Sitalakhya river system (Alam, 2011; Ali and Rahman, 2004).

As a part of a restoration project, eleven especially designed sewage diversion structures (SSDSs) have been constructed at eleven major storm sewer outfall locations around Hatirjheel. During the dry season, the entire flow (sewage) carried through the storm sewers up to Hatirjheel are diverted downstream along the peripheral "main diversion sewer". The SSDSs allow overflow of storm water into Hatirjheel during wet season. Since the storm sewers carry both storm water and sewage, the SSDS overflows deteriorate the water quality of Hatirjheel. Water quality modelling suggested reasonable water quality of Hatirjheel, except for a brief period at the beginning of each wet season (Samad, 2009). However, it should be noted that the SSDSs and the main diversion sewers of were designed based on surveys carried out in 2007. However, continuously increasing population density and expansion of storm sewer network are putting extra pressure (both hydraulic and waste-load) on Hatirjheel water management system. Water quality of Hatirjheel deteriorated significantly during wet seasons of 2012 and 2013 due to huge overflows of mixed rainwater-sewage; there was significant spatial variation in water quality, depending on locations of SSDSs. It is important to assess whether the SSDS and main diversion sewer system of Hatirjheel would be able to handle the hydraulic load, and the extent of water quality deterioration of Hatirjheel in future years, for development of appropriate mitigation strategy.

MATERIALS AND METHODS

As a part of this study, sewage flows through the storm sewer network discharging into Hatirjheel have been estimated. For this purpose, GIS based catchment map of Hatirjheel, delineating catchment area under each SSDS, has been prepared based on storm sewer network maps collected from Dhaka Water Supply and Sewerage Authority (DWASA). Sewage flows within Hatirjheel catchment areas have been estimated from water use. Water use within Hatirjheel catchment has been estimated by analysing (using GIS) locations and production of water (for the year 2007) of DWASA deep tube wells (DTW) within Hatirjheel catchment; it was assumed that 90% of supplied water turns into waste water. Estimates of future (up to 2025) sewage flows have been made based on population projections. The capacity of the peripheral main diversion sewer network of Hatirjheel to carry the estimated/ projected wastewater flows in the future has been estimated.

RESULTS AND DISCUSSIONS

Catchment Area of Hatirjheel

Table 1 and Figure 1 show catchment areas of different SSDSs of Hatirjheel. The combined total catchment area of all 11 SSDS of Hatirjheel is about 23.20 km^2. Storm water from Kawran Bazar, Shangshad road, eastern part Dhanmondi Lake, Panthopath,Nilkhet Road flows to Hatirjheel through SSDS-1. SSDS-1 has the largest catchment area of 6.1 km^2, followed by SSDS-10 (4.87 km^2). SSDS-10 carries water from Mohakhali, Niketon, Nakhalpara and Montripara areas. Flow from Gulshan and Banani enters into Hatirjheel directly though Gulshan and Banani Lake (see Fig. 1).

Estimation and Prediction of Sewage Flow

Table 2 shows estimated flow of sewage reaching different SSDSs through the storm sewer network up to the year 2025. As noted earlier, the sewage flow was estimated from water use, based on water production of DWASA deep tube well (for the year 2007) located within the catchment areas of Hatirjheel. It was assumed that all sewage generated within the catchment areas flow through the storm sewer network and reach different SSDSs of Hatirjheel. The estimated sewage flows matched reasonably well the measured sewage flows at some of the SSDS locations in 2007.

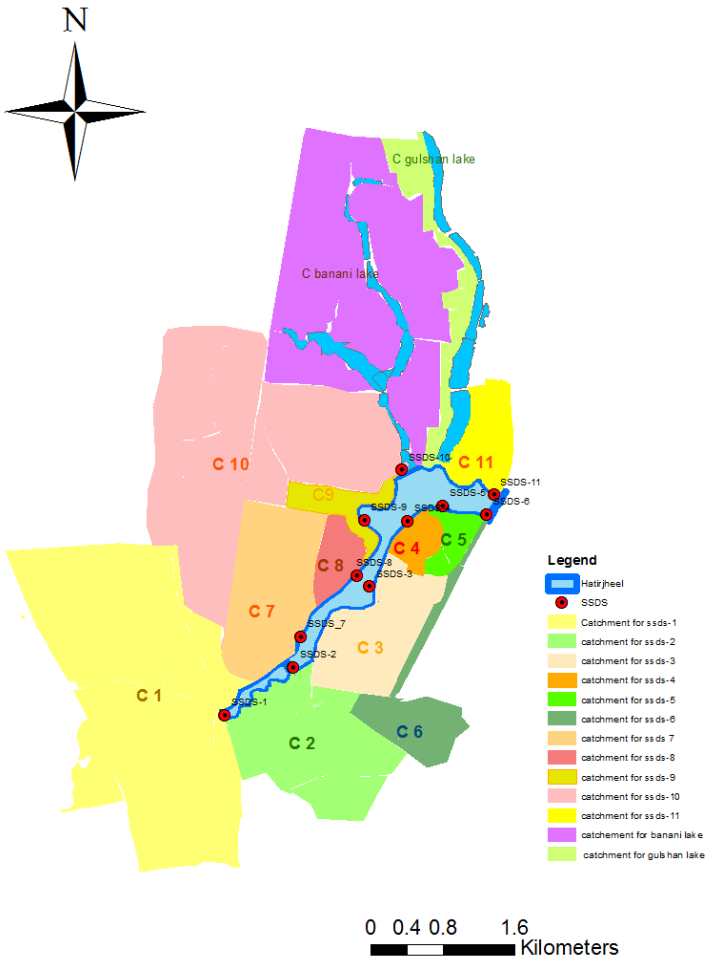


Fig. 1: Catchment areas of different SSDSs discharging into Hatirjheel

Table 1: Catchment areas of different SSDSs of Hatirjheel

SSDS/Area	Catchment areas of SSDS (km^2)
SSDS - 1	6.1
SSDS - 2	1.9
SSDS - 3	1.3
SSDS - 4	0.2
SSDS - 5	0.3
SSDS - 6	0.26
SSDS - 7	1.6
SSDS - 8	0.4
SSDS - 9	0.4
SSDS - 10	4.87
Gulshan lake and Banani Lake	5.23
SSDS - 11	0.64
Total catchment area	23.20 km^2

Capacity of Hatirjheel Sewer Network to Accommodate Sewage Flow

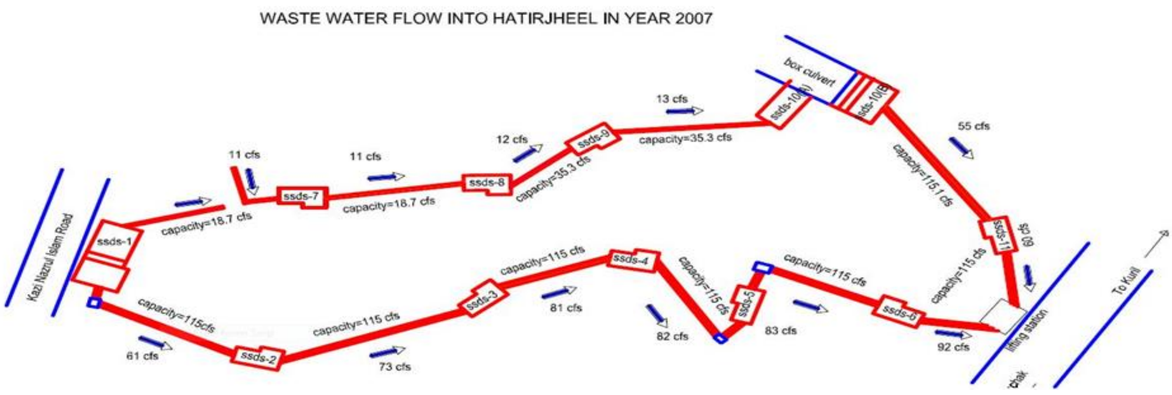
The capacity of main diversion sewer of Hatirjheel has been estimated based on diameter and slope of the sewers, considering 70 percent of the full flow capacity (to account for accumulation of sludge over time). The capacity of the main diversion sewer running along the entire southern periphery of

Hatirjheel (i.e., from SSDS-1 to SSDS-6 consisting of 1800 mm twin sewers) is 115 cfs. Along the northern periphery, the capacity of peripheral main diversion sewer varies; from SSDS-7 to SSDS-8 (1200 mm single sewer), the capacity is 18.7 cfs; from SSDS-8 to SSDS-10 (1524 mm single sewer), the capacity is 35.3 cfs; from SSDS-10 to SSDS-11 (1830 mm twin diversion sewer), the capacity is 115 cfs.

Table 2: Estimated sewage flow reaching SSDSs through strom sewer network

SSDS	Estimated wastewater flow (cfs)				
	2007	2011	2015	2021	2025
SSDS -1	62	66	93.5	124	153
SSDS -2	12.2	13	18	24	30
SSDS -3	7.8	8	12	16	19
SSDS -4	1.1	1.2	1.5	2	3
SSDS -5	1.1	1.2	1.5	2	3
SSDS -7	11	12	17	22	27
SSDS -8	1.1	1.17	2	2.5	3
SSDS -9	1.1	1.17	2	2.5	3
SSDS-10	42.1	45	64	84	104

Figures 2-5 show estimated sewage flow reaching Hatirjheel in the years 2007, 2011, 2021 and 2025; the capacity of the sewers also shown in the figures. Figures 2 and 3 show that the main diversion sewer system is able to accommodate the sewage flows in 2007 and 2011. However, the situaiton changes from the year 2015 onwards. In the year 2021, the total sewage flow reaching Hatirjheel has been estimated to be 308 cfs. The sewage flow reaching SSDS-1 has been estimated to be 124 cfs, which alone exceeds the capacity of twin main sewer (115 cfs) along the southern periphery of Hatirjheel (see Fig. 4). Along the southern periphery, the sewage flow exceeds the capacity of main diversion sewever from SSDS-7 to SSDS-8. The situation deteriorates further in the year 2025, as shown in Fig. 5; the sewage flow exceeds the capacity of main diversion sewer throughout the system, except for the portion of main diversion sewer from SSDS-8 to SSDS-10.



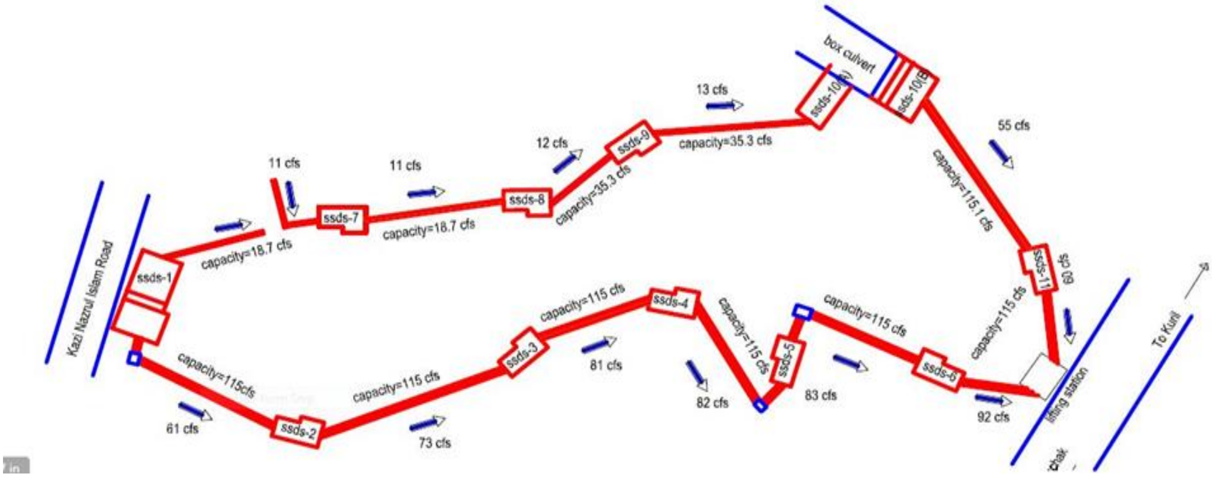


Fig. 2: Sewage flow reaching Hatirjheel in 2007 and capacity of main diversion sewer system

Fig. 3: Sewage flow reaching Hatirjheel in 20011 and capacity of main diversion sewer system

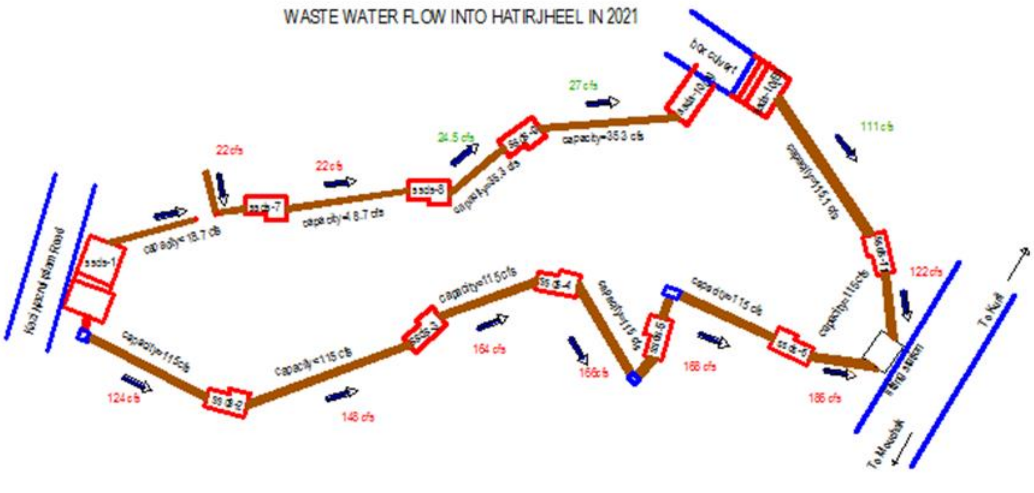


Fig. 4: Sewage flow reaching Hatirjheel in 2021 and capacity of main diversion sewer system

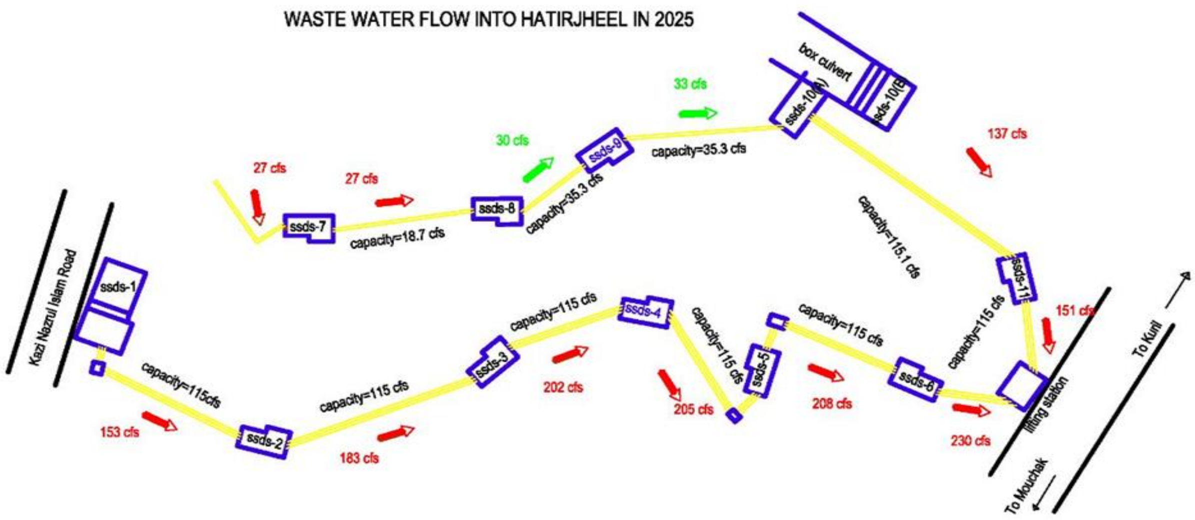


Fig. 5: Sewage flow reaching Hatirjheel in 2021 and capacity of main diversion sewer system

CONCLUSION

The capacity of Hatirjheel main diversion sewer system has been evaluated by estimating dry weather flow (i.e., domestic sewage) generated from the catchment areas of individual SSDSs for present and future times (2015, 2021, 2025). It shows that the main diversion sewer system may not be able to accommodate the increased sewage flow in the near future (2015 and beyond). Due to increase in sewage/ wastewater flow, the overflow of sewage-storm water mixture into Hatirjheel will increase significantly during the wet season; this is likely to cause significant pollution of Hatirjheel during the wet season. In order to improve water quality of Hatirjheel, the domestic sewer connections to the storm sewer system must be disconnected gradually by DWASA. At the same time sewage flows from certain sections of the Hatirjheel catchment should be diverted (e.g., toward Pagla sewage treatment plant from the southern part of Hatirjheel) so that the conveyance capacity of the main diversion sewer is not exceeded.

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CHARACTERIZATION OF GREY WATER GENERATED FROM A RESIDENTIAL HALL AND ITS POTENTIALITY FOR NON-POTABLE USES

M. S. Islam[1]*, M. S. Mia[2], M. Sumun[2], G. Biswas[2] & M. S. Osman[2]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
[2] *Department of Civil Engineering, Dhaka University of Engineering and Technology, Gazipur-1700, Bangladesh.*
**Corresponding Author, <shohidul99@gmail.com>*

ABSTRACT

The critical importance of water to sustainable development is clearly recognized in the Millennium Development Goals (MDGs). With the increasing of urbanization as well as population growth, not only the demand of fresh water but also the volume of wastewater is being increased. In general, increase in water demand and changes in the climate are responsible for water supply problems. So new water use models and wastewater reuse patterns are of utmost importance in water research today. Reducing the demand of fresh water can be achieved by reusing grey water (water discharges from laundry, shower, bath, ablution, sinks and kitchen). Grey water has great potential for reuse due to its availability (around 70% of domestic wastewater) and its low concentration of pollutants compared with combined household waste water. So there is close possibility to gain an effective and sustainable solution in the field of waste water reuse system. In our study, grey water generated from a residential hall was quantitatively and qualitatively characterized, and then grey water reuse requirements with treatment or without treatment were analyzed. Samples were characterized in accordance with pH, color, turbidity, TDS, Conductivity, COD, BOD5. The outcomes were compared with different water quality standards for non-potable uses. Our aim is to determine the grey water reuse potentiality for non-potable uses. This will reduce the volume and cost of wastewater treatment as well as the demand of groundwater. It has been found from this study that the water from bathing and ablution have the lowest concentration if compared to combined grey water.

Keywords: MDG, insufficient water supply, Grey water, BOD5, COD, non- potable uses.

INTRODUCTION

Water is a valuable resource and must be conserved. Water is an essential element for life. This resource however, has been badly mismanaged by over consumption and pollution. The dominance of water shortage is increasing, the amount of people currently living under water stress is 700 million and it is projected to be 3 billion people in 2035 (World Bank, 2005). The need to ensure balance between water supply and water demand cannot be over emphasized. In both developing and developed countries, it has been observed in recent past that many of the cities and municipalities are running out of such sources that have good quality water, mainly during the dry period. Dhaka, the capital of Bangladesh has been suffering water crisis for a few years. About 87% of its supplied water comes from ground water sources and 13% from peripheral rivers (Khan, 2012). But due to over extraction of ground water, the water table has gone below the pumping level which has resulted in abandoning deep tube wells at many locations. Also due to poor quality of peripheral river water, Dhaka Water Supply and Sewerage Authority (DWASA) cannot think of using these rivers as a potential source for future.

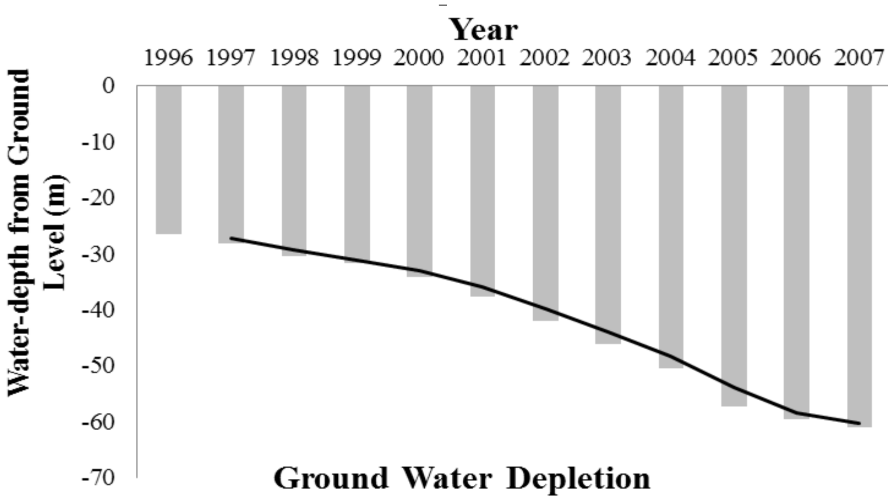

Fig. 1. Ground water depletion of Dhaka City (Source: Tamanna 2005)

Gazipur is an industrial area located beside Dhaka city where residents are growing day by day at a faster rate. Now ground water is the only source of fresh water for people of Gazipur.This source is very limited as per present and future need and scarce. This limited water is being used for potable and non-potable purpose such as Gardening, Car washing, Flushing etc. Once Gazipur district will be the dense populated city like Dhaka. It is much known to all that the ground water table in Gazipur district is lowering considerably day by day making some previously installed pump inert. Research on gray water use is critical right now and is likely to become a topic of major interest in the future. In many parts of the world, both in the industrial and developing countries, there has been an increased interest in the reuse of grey water. The grey water can be reused in irrigation include toilet flushing if it is collected separately before going to the sewage system. Grey water is all wastewater that is discharged from a house; excluding black water (toilet water).This includes water from showers, ablution, bathtubs, sinks, kitchen, dishwashers, laundry tubs, and washing machines. It commonly contains soap, shampoo, toothpaste, food scraps, cooking oils, detergents and hair. Grey water reuse can be the most effective sustainable solution for near future. Dhaka university of Engineering and technology (DUET) is situated in Gazipur city. Dr. F. R. Khan Hall is one of the Boys residential hall of DUET where there is a large amount of fresh water is used for gardening and toilet flushing. If a considerable portion of this water can be resolved by reuse of grey water that would be a sustainable solution.

Standards and guidelines required for reusing grey water

The available criteria for water reuse for toilet flushing (EPA, 1999) and for domestic water reuse (Surendran & Wheatley, 1998) are presented in Table 1

Table 1 : Standards and criteria/ guidelines for water reuse for toilet flushing and domestic water recycling.

Parameter	**Toilet flushing**		**Domestic water recycling**					
	US	**Japan**	**WHO**	**USEPA**	**USA NSF**	**Australia**	**UK**	**Germany**
pH	6-9	5.8-8.6		6-9				6-9
Turbidity(NTU)	≤2			5		2		1-2

Odor	odorless	NU						
BOD_5(mg/L)	≤10			10		20		20
TC(no./100ml)			1000(m) 200(g)	<10		<1	ND	100
FC(no./100ml)	ND	≤10 (E.Coli)		<10	<240	<4		10
Residual Cl2(mg/L)	1	Retained*						
Appearance		NU						

ND=not detectable; NU =not unpleasant

(m) =mandatory; (g) =guideline

* at last holding tank in distribution line

Table 2: Parameter to be required for using grey water on the Agricultural Sector

Parameters	Maximum Permitted Values
pH	6.5-8.5
Conductivity (Ds/cm)	2000
BOD (mg/L)	120
COD (mg/L)	200
Total Suspended solid (mg/L)	120
Fecal Coliform (MPN/100ml)	1000

Source: M.Platzer, V.Caceres, dan N. Fong, 2004

MATERIALS AND METHODS

To determine the water use in residential hall and find out the amount of grey water generated first we fixed the location where we could get the favourable environment for successfully completing our job, considering many logical things we selected Dr. F.R Khan Hall of DUET campus. Here we calculated the amount of water used for cloth wash, bathing, utensils and others (ablution, wash basin) by questionaries survey. For qualitative analysis of grey water we collected sample from the basin, bathroom and ablution point by stoping those outlets temporarily. Four types of sample were collected in different jar. They were: (i) Cloth washing water (ii) Bathing water (iii) Basin water & (iv) Ablution water. Then collected samples were tested in DUET Environmental Laboratory to determine seven water quality parameters which were (i) pH (ii) Color (iii) Turbidity (iv) TSS (v) Conductivity (vi) COD and (vii) BOD_5.

RESULT

According to survey and laboratory analysis following data has been found which includes grey water generation, water demand for gardening and toilet flushing and characteristics of grey water.

Table 3: Grey water generation at Dr. F .R khan Hall.

FIELD OF USING	QUANTITY AVERAGE (LPCD)
Cloth wash	30
Bathing	33
Wash basin	8
Ablution	10
Total	**81**

Grey water Generation:

Number of student and workers = 146

Total amount of grey water generate in a year = 81 x 146 x365 liters

= 4316490 liters\year

Water demand for gardening and Toilet Flushing:

The whole year has been divided into three seasons depending on water demand for gardening of proposed hall. The demands are mentioned bellow.

Discharge rate for sprinkling = 25/80 = 0.3125 liter/sec

(i) July to October = 3 hours per 6 day per month

=3x60x60x4x6x0.3125

= 81000 Liters

(ii) November to February = 4 hours per day

= 4x60x60x4x30x0.3125

= 540000 Liters

(iii) March to June = 2 hours per day

= 2x60x60x4x30x0.3125

= 270000 Liters

Total demand for gardening in a year, Q = (i)+ (ii)+(iii)

= **891000 Liters**

Water demand for toilet flushing in a year = 9 x146x365 liters

= **479610 Liters**

Water required for gardening & toilet flushing = 891000 liters +479610 Liters

= **1370610 Liters**

Table 4: Laboratory analysis of grey water.

Sample Name	**Water Quality Parameter**						
	pH	**Color (Pt-Co)**	**Turbidity (NTU)**	**Conductivity (µs/cm)**	**TDS (mg/L)**	**COD (mg/L)**	**BOD_5 (mg/L)**
Wash basin-1	6.62	106	76.5	370	183.3	27	20
Wash basin-2	6.85	103	75	375	187.5	ND	ND
Cloth Washing-1	6.95	417	315	731	371	246	194
Cloth Washing-2	6.92	415	314	592	295	528	425
Bathing-1	7.06	71	129	401	201	63	48
Bathing-2	7.13	73	131	400	193.4	ND	ND
Ablution-1	6.93	69	95	387	193.4	33	22
Ablution-2	7.03	70	92	386	193.6	ND	ND

ND: Not Detectable.

DISCUSSION

This study shows that the amount of generated grey water is much higher than required amount of water for toilet flushing and gardening in a year. So the grey water will meet the water demand for non potable use (toilet flushing and gardening). The pH values obtained from laboratory analysis (Table 4) all most within safe limit as per guidelines of toilet flushing and gardening. The turbidity and color of grey water particularly from wash basin and ablution is lower than that of bathing and cloth washing. Using soap is mostly liable for the higher value of turbidity and color in bathing and cloth washing water. The BOD5 values of wash basin, bathing and ablution are allowable for gardening but not for toilet flushing and that of cloth washing water is not plausible neither for toilet flushing nor gardening.

CONCLUSION

The main problems of grey water are color, turbidity and BOD_5. For this reason the grey water is dirty in color and of bad odour. This water can be used for toilet flushing and gardening after easy treatment using Rapid/Fine Sand Filter. Our study shows that the all types of grey water are not required treatment for gardening which can be used directly. That will also reduce the treatment cost. The portion of grey water that do not have required quality for toilet flushing and gardening can be treated and then used. The study concludes that all types of grey water do not require all level of treatment. The separate treatment may reduce treatment cost. But in real practise the grey water is discharged in mixture. The separate treatment process can be adopted only if the grey water discharge line is separated.

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RENOVATION OF SEPARATE SEWER NETWORK AND ITS ENGINEERING SIGNIFICANCES AT A RESIDENTIAL AREA IN BANGLADESH: A CASE STUDY

Quazi Shammas Sayeed[1*], S. M. Moniruzzaman[2] & M. H. Mishuk[2]

[1] *Department of Civil Engineering, Stamford University Bangladesh, Dhaka 1217, Bangladesh.*
[2] *Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna- 9203, Bangladesh.*
**Corresponding Author, <quazi.shammas@yahoo.com>*

ABSTRACT

Trustworthy wastewater collection and disposal system is essential for human health and economic development. A case study of such work is very important for its application to other areas of the cities and towns of Bangladesh. Construction and renovation of sewer network at a selected developed area is very tough and needs high supervision to complete this type of project in time with a good quality. residential area in Dhaka are a very complicated job as there are no by-pass lanes in the area for proper transport. Dhaka WASA has done a renovation work during 2013 through the Sudhi Samaj Abashik Area to make a separate waste-water collection system. The aim of this study is to find a better way of working procedure and how to cope with the limitations, enrich the data of this sanitation engineering sector and finding how to make a proper use of the resources. This paper contains an extensive study on sewer renovation work and describes selected important features as well as the limitations and the effectiveness of such renovation work. The result shows that, (i) the renovation work took comparatively shortest possible time; (ii) Contract signing with two separate contractors for a same project increase the efficiency and accuracy of the work; and, (iii) A higher quality of work was achieved by full co-operation of the community. (iv) Storm sewer failure, water entrapping, overflowing from manholes and waste water get contaminated with drinking water haven't overcome.

Keywords: Construction, Renovation, Conventional Sewer System, Waste-water collection system.

INTRODUCTION

Adequate water supply, effective waste-water collection and proper treatment are essential for human health and economic development. Installing and maintaining a well planned waste-water collection system is three times more expensive than the cost of constructing fresh water supply system. Developed and industrialized countries have fairly high standard water and waste-water control system. On the other hand, severs problems in water and waste-water collection systems are apparent in developing countries (Wilderer and Schreff, 2000). Lack of attentions and active organisations, the waste water collection and disposal sectors are being undeveloped in developing countries. The concern of this paper is a storm water collection system. Storm water collection and disposal system failure causes a heavy pollution. Moreover, contamination of storm water with sewage water cause an increase in the treatment cost of waste-water. Geographically Bangladesh is not experiencing uniform rainfall throughout the year. During monsoon a heavy rainfall occurs and the amount of net rainfall during winter is very little. Bangladesh is a developing country. It is the ninth most populous country and twelfth most densely populated country in the world. Moreover, the population growth in urban area is increasing rapidly. The more population is growing the more severe problem in waste management as well as the waste-water collection system produced. Dhaka is the most polluted city in the world (Wikipedia). As it is a developing country and due to insufficient budget, it is now our concern to construct an adequate waste-water collection system for Bangladesh.

Dhaka City Corporation is coming to the point of implementing such effective strategies to the urban area waste-water collection systems which are less costly as well as trustworthy. Waste-water collection system in Dhaka is a centralized waste-water collection system. In order to increase the capacity of sewers and to decrease subsequent failures due to age and third party damage, Dhaka City Corporation is planning and arranging renovation works among the communities in Dhaka with the help of Dhaka WASA. Sudhi Samaj Residential Area (SSRA) in Ramna Thana is one of those communities. A renovation project has done through this area during the year 2013. Previously this area was under a combined conventional system. Dhaka City Corporation and Dhaka WASA have constructed a separate conventional sewer system through this area. This paper contains the detail description of separate sewer construction work by Dhaka City Corporation. On the basis of collected data, a discussion on selected important features such as the engineering significances of a separate conventional sewer system according to the geological and economical basis of Bangladesh, benefits of inhabitant co-operation, limitations of this renovation work provided in this paper. This paper will be helpful for planning and projecting further renovation work in similar densely populated communities in Bangladesh. On the basis of collected data, selected important findings are concluded. To find the limitation and mistakes is not the principle aim of this case study, rather to identify the effectiveness and potentiality of such kind of renovation work according to the circumstance of Bangladesh such as scarcity, weak management system on rapid increase of population and a tremendous change in our climate as well. These circumstances are challenging the efficiency of urban drainage crisis management also (Macaitis, 1994).

METHODOLOGY

Based on the construction process, we have to classify the area in two parts. On the basis of primary data, we made a description about the renovation procedure. Using the secondary data, we discussed the limitations and effectiveness of this renovation work. This renovation project was managed under Dhaka City Corporation. All primary data were collected under two separate procedures.

We have made six times field survey to collect data. At first stage when the renovation was started, we collected numbers of roads, the lengths of the roads, average widths of the roads. At the second time, we collected the data about excavation, bedding and laying of pipes. In the third phase, data about Junctions were collected. Fourth phase was for backfilling. On fifth phase, we surveyed the situation after the renovation work and we made a last observation after one year during the month May and June to investigate the condition of the sewer system when it is subjected to heavy torrential rain. Secondary data were collected from some important waste-water management related books, papers, journals as well as internet. Those entire sources are investigated in this phase of the study.

DESCRIPTION ON SUDHI SHAMAJ RESIDENTIAL AREA (SSRA):

SSRA is located near the Ramna Thana of Dhaka City. It is an imperative residential zone of Dhaka. School, college, university, shopping mall are available around the area. SSRA is one of the well planned residential areas in Siddheswari under Ramna Thana. The tendency of living here is very high. There are total 56 multi-storeyed buildings; most of them are above six storied. Among them there are two fourteen storied residential buildings also. There are four stationary shops, one hospital, one mosque, two laundries, two academic organizations and 54 residential buildings. Around 13000 people are living here at present. In 2003, the population of this area was near 4000. The growth rate of the population of this area is 23.3% per year now. Calculation says that after 10 years the population will be 20000 (Shammas et. al. 2014). The total area is covering 0.25 km^2 only. So the population density is very high in this area. There are four openings with highway and have two subways through SSRA. This area is restricted from high weighted vehicle (Figure: 1). SSRA is a part of Dhaka city. So the geographical condition of this area is similar to Dhaka city. Dhaka is located in central Bangladesh at 23°42′0″N 90°22′30″E, on the eastern banks of the Buriganga River. Climate of Dhaka is hot, wet and humid. The average annual temperature is 27^0C. Rainfall over the city is not uniform.

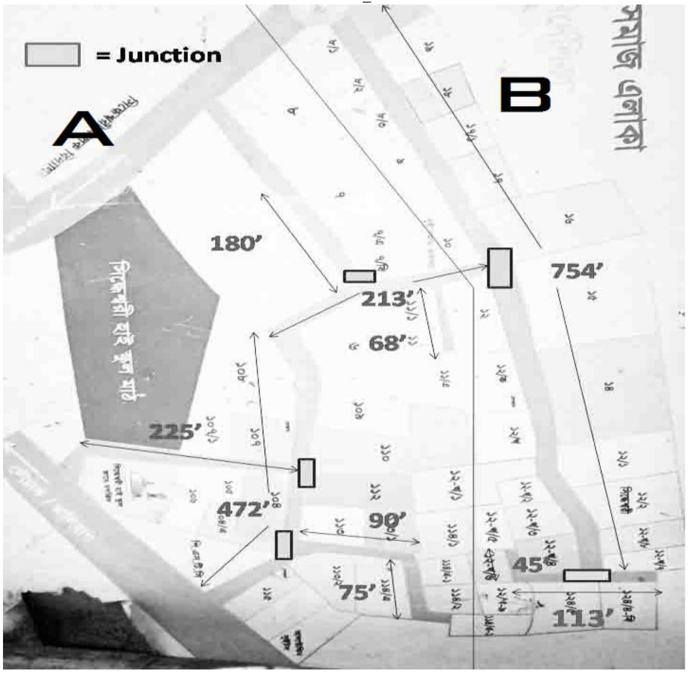

Figure 1: Map of Sudhi Samaj Abashik Area (SSRA)

DESCRIPTION OF RENOVATION PROCESS:

There are some factors, whether a separate conventional system or a combined conventional system will follow. Basically, it depends on: the population density, quantity of water fall over a year, the ratio between sewage and night soil. When there is a huge population discharges enough sewage for accelerating the night soil to obtain the minimum self velocity, and there is non-uniform rainfall through a year over the area, a separate conventional system is preferred. Moreover, to minimize the pumping and treatment cost, separate conventional system is helpful (Ahmed and Rahman, 2000). Depending upon these factors, installing a separate conventional sewer system was suitable for SSRA. This renovation process basically covered an installation of sewage sewer where the previously installed system was left for storm sewer. Though SSRA is a small residential area covering only 0.25 km^2, only one contractor could be completed the work. But two different service provider were assigned to this work. SSRA was divided in two parts A and B (Figure 1: Map) for contract making. Any General Material for sewer will be given considerations, but the material should be adapted to the local conditions. Such as: character of wastes, possibility of necessary space, soil characteristics, exceptional heavy external loadings etc. (NPELF40-2005). There are no Industries here. So, only domestic waste will pass through this sewer system. The roads through SSRA have a width at least 10 ft. Soil condition of this area is silty clay. As heavy vehicles are restricted from this area and there is no highway or railway through this area, so the sewer will not be subjected to heavy external loadings. In this circumstance PVC pipe was used to install the sewage sewer. The work started on 10th April, 2013 and finished on 28th May, 2013. It needed 48 days to complete the work. Total 4600 feet long sewer system was installed.

Maintenance crews used the plan view map for line and manhole location. Before beginning excavation, they also identified the nearby utility lines and to avoid any damage. Workers sent a notice of this excavation to all dwellers by their building's notice board before one week. As SSRA is a flat surface area, so the depth of the trench will not vary. Traditional method of excavation was followed. 24 inch wide trench was made for sewage sewer and 50 inch wide for storm sewer. The average depth of the trench was 6 ft. Trench sides were kept as nearly vertical as possible. Pipes were joined by concrete rings having an inner-radius as same as the outer radius of the concrete pipe. 1/400 slope is essential for the pipes having a diameter of 36 inches (3 feet) to obtain minimum self-cleaning velocity (2 ft/s- it is the minimum self cleaning velocity). Here the rise of the pipe is 2 ft from North to south (754 ft) in part A and 1.5 ft from East to the Middle West (685 ft) in part B. So the slope is 1 in 385 (0.26%) for part A, 1 in 476 (0.21%) for B. This slop is very close to the recommended slop at 1 in 400 for the plane topographical area (Ahmed and Rahman, 2000). There are four major cleanouts were made having a length of 4ft and a width of 3 ft. 10 more small cleanouts (2 ft wide and 2 ft long) were made along the main sewer line and lateral sewer line also. Inner side of Cleanouts was plastered, outer-side were not plastered.

During the year 2000 around 3000 people lived SSRA. Till 2008, the population growth rate was similar. After the year 2008, there is a revolutionary development done in this area. Small buildings are replaced with multi-storeyed buildings by various developer companies. As much the capacity increased, the population growth increased rapidly to 23.3% per year. The revolution has not completed yet. Still there are some high raised buildings are under construction. So, there is a possibility of a further population increase. 20000 will be the total population in 2018 predicted based on not current but do for average increment rate (assumption is done by simple algebraic method). This sewer construction consists three types of sewer lines. They are: Building sewer, Lateral sewer and Main sewer. Main sewer covering around 845 ft, the building sewer covering around 113 ft and the rest are lateral sewer. In total it was a 4600 ft long sewer renovation work. 464 pipes were needed to complete the project.

DESCRIPTION OF PREVIOUS SEWER SYSTEM:

Sudhi Shamaj Residential Area (SSRA) is a community established in 2002. Till 10th April, 2013, a combined sewer system was installed to collect the sewer from part A which was connected to the Trunk sewer along the "Siddheswari Road" and "Selina Parvin Road". Pre-cast Concrete pipe was used in previous sewer system. The diameter of the pipes was 18 inches. The length of the pipes was 10 ft. Thickness of the pipe was 1.5 inches. Service ability of small diameter pipes is more critical than the larger core. These sewers are increasing in age and no form of pre-emptive measures are taken it is inevitable that they will be subjected to a higher risk of failure in the future (Oli-Phant, 1993). After 2003, a huge number of new multi-storeyed buildings were constructed and a rapid growth in population has seen. This huge population started to produce a large quantity of waste-water. It was not able to convey out this huge amount of waste-water for the previous sewer system and the situation became uncontrolled.

ENGINEEIRNG ASPECTS:

During the renovation process following phenomenon were observed: Two service provider were assigned to do this job. The employment of more than one contractor means that a larger pool of skilled labour resources will be available for the works. Another positive point is, when two contractors are assigned for similar kind of work at the same time and same place, there could be competition in reputation and effectiveness of construction methods. Such sense of "Professional pride" leads the contractors to do their best effort (Tai R et. al, 2010). Bangladesh is a developing country. There is a shortage of machineries and heavy mechanical tools. Almost all development works are done by traditional methods. It implies that almost all development work is a lengthy process in our country. Comparatively renovation work at SSRA was done in a short time in a stress 48 days only. There was a total six days brake due to natural calamities. Shortage of tools for this work had seen. SSRA is a densely populated area. There was no open field such as a playground in SSRA. There was no enough space on the road side. So they had to use the road to keep construction materials. There was no railing stuffed beside the trance. Due to rainfall the work had interrupted. Due to lack of proper supervision, workers failed to lay these pipes according to the proper slope. Somewhere, they made wrong slop also. There was a plan of laying pipes minimum 5 ft below from the formation level (FL) of the roads. But they placed the pipes average 3 ft depth. To gain the minimum depth, the FL of the road will by higher. It will result in a minimization of the difference between the plinth level (PL) and the formation level (FL). In some cases, the FL becomes higher than the PL. Both inside and outside plastering is needed for acquiring the water tightness of cleanouts. Cleanouts were not plastered properly. So, proper water tightness of those cleanouts was not ensured. Sand is a good material for backfill. After laying the pipes, trenches were filled with native materials with a small amount of sand. Moreover, those materials contained sharp edge large rock and brick-chips. These rock and brick-chips will cause damage to those pipes. After all, it was done to reduce the cost. Workers simply filled the trench, but compaction was not done properly. Due to lack of well supervision, proper compactness wasn't done. No risk management plan was applied for both workers and properties of this area. No records are kept and provided about this sewer renovation. So it becomes hard to indentify the location of sewer below the ground. So there is a chance of interfering with the sewer while further construction; and often the absence of linkage between fixed asset financial database and maintenance management records (Grigg 1994).

After exactly one year, during sixth time observation, we have found that the road goes under water after being subjected to a 15-20 minutes heavy rain-fall. Moreover the trapped water gets diluted with overflowing sewage water from two manholes. It implies that, storm sewer fails as well as the sewage sewer. The causes of these failures are following: The storm sewer system is blocked with sediment during dry season as there is no water passing through it. A sewage sewer is blocked with night soil due to wrong slop done during construction. As a result, sewage is overflowing from those two manholes due to back-flow. Another important cause is, the system is subjected a large volume more than its design capacity. Generally, these types of blockade are formed due to lack of supervision during construction work and maintenance after the project is completed. Except this, the sewer system is adequate for SSRA. This project can be a model for the rest of the community, where there is no separate sewer system is installed yet. More research should be done in this area to enrich the data of this sanitation engineering sector.

CONCLUSIONS:

From the statistical analysis, it can be found that: according to the study based on the geological conditions of Bangladesh, this kind of renovation work discussed above, in sewerage network are essential for all cities and towns of this country if given. Dhaka is experiencing a non-uniform rainfall. To avoid overflow from sewage sewer separate conventional sewer system is helpful. Moreover, when storm water is not contaminated by sewage, the storm water can be disposed directly to receiving water body without any treatment. As a result, the treatment cost can be reduced. Capacity of sewer system should be maximized to avoid future failure. Separate conventional waste-water collection system should be implemented through all communities of Dhaka city. Like most other countries, there is a lacking of record keeping about the sewers beneath the ground. There is no proper record about the location of pipe lines which may interfere further developments along the line. Therefore, it is necessary to have a location map after completion of the work. The total renovation work of sewer network with length 2300 ft took duration of 23 days and the whole renovation work including construction of separate sewage sewer and road pavement took 75 days. It took comparatively shortest possible time. Contract signing with two separate contractors for a same project increase the efficiency and accuracy of the work. To avoid any interruptions by unexpected rainfall, this type of renovation works can be done in spring. A proper risk minimizing plan should be implemented for both the properties of the area and the workers. Necessary supervision is needed so that the contractors or the workers cannot do any fraud. Another important acknowledgement of this study is that, the community people were fully co-operate with the project work from its beginning to end and which make the project successful in a shortest possible time. Due to their awareness regarding the project, comparatively a higher quality of work was achieved. Dhaka WASA should give emphasis on assigning organizations. They should assign such companies that are rich in their instrument and experience manpower indeed. Proper maintenance and survey should be brought with them after a considerable period especially before the monsoon season all over the Dhaka city to provide a better, reliable, healthy and aesthetic waste water collection system to the citizen of Bangladesh.

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FEASIBILITY STUDY OF PHARMACEUTICAL WASTEWATER FROM CEPHALOSPORIN PROCESSING UNIT

Masrura, U. Sayeda[1*], Haider, K. Shafayet[1] & Farooque Ahmed[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka- 1000, Bangladesh.*
**Corresponding Author, <tamrin_06@yahoo.com>*

ABSTRACT

Pharmaceutical industries are now producing different pharmaceutical products including second and third generation Cephalosporin antibiotic with high bio-toxicity. Wastewater generated from Cephalosporin processing unit is very hard to degrade directly through biological process, and could cause great harm to the environment and human being if disposed untreated. This study has been carried out to develop an efficient treatment process of Cephalosporin processing unit wastewater through chemical, biological and physio-chemical processes. It has been attempted to breakdown the β-lactam ring of Cephalosporin through pH adjustment followed by biological treatment of waste mixing with different percent of sanitary sewage. It has been observed that the antibacterial activity of antibiotics affected by pH adjustment (an increase around 10.5) was effective for conducting biological treatment through breaking down β-lactam ring and 62% COD reduction could be achieved through simple aeration process without any further adjustment of pH and addition of nutrients. Finally chlorination has been practiced to ensure further reduction of COD value followed by Activated Carbon Adsorption process to reduce residual Chlorine concentration and to achieve the desired ECR, 1997 limits. Optimum aeration period and optimum chlorine dose have been determined through extensive laboratory test. Optimum rate of flow and contact time for Activated Carbon Filtration has also been identified.

Keywords: Wastewater from cephalosporin production, β-lactam ring breakdown, Activated Carbon Filter

INTRODUCTION

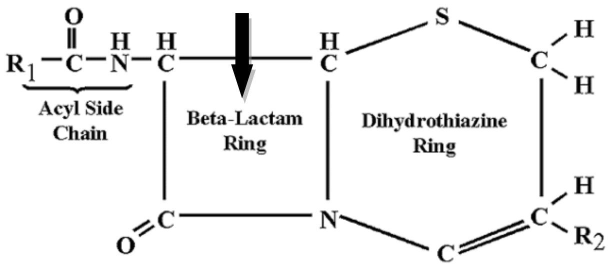
During the last one decade remarkable development has taken place in the field of Textile and Pharmaceutical industries and significant numbers of these industries have been established mostly around Dhaka city. This pharmaceutical waste contains β-Lactam antibiotics. β-Lactam antibiotics (beta-lactam antibiotics) are a broad class of antibiotics, consisting of all antibiotic agents that contains a β-lactam ring in their molecular structures. Thus it threatens the ecosystem and livelihood of tens of thousands of people who use these resources. Therefore, minimization of pollution from the wastewater is necessary for the protection of the environment. Selection of appropriate treatment option is one of the major steps towards reducing pollutants from pharmaceutical industries. While doing that information regarding treatment efficiencies of treatment option is very important. The sample was collected from a well-known pharmaceutical industry of Bangladesh. In this study attempt has been made to provide a guide line for designing and operating treatment plant of pharmaceutical liquid waste. Different treatment options have been discussed to develop efficient & cost effective treatment option which will encourage the industries to install and run effluent treatment plant. The Department of Environment (DOE) has completely prohibited the disposal of such wastewater in the environment (Zero Flow Discharge).

LABORATORY SCALE MODEL STUDIES

Laboratory scale model studies have been conducted to determine the design criteria and for the selection of appropriate treatment unit processes for an Effluent Treatment Plant under the present wastewater characteristics of a Pharmaceutical industry particularly from cephalosporin processing unit. In order to ascertain the treatability of pharmaceutical liquid wastewater, the wastewater quality parameters like pH, color, and turbidity values, BOD, COD, and suspended solids concentrations have been measured time to time.

INITIALIZATION AND PREPARATION FOR BIOLOGICAL PROCESS

pH adjustment and Nutrient addition – Initially three weeks continuous aeration showed no changes in the quality of raw wastewater even in the presence of nutrients. Therefore, it was understood that biological process is not feasible without inactivation of antibiotic property of Cephalosporin (breaking down of β-lactam ring). Both PH increase and decrease shows good result in the breakdown of β-lactam ring, however, increase of pH is much easier. Prior to diffused aeration process pH of raw wastewater samples were increased around 10.5 to breakdown the β-lactam ring of Cephalosporin through alkaline hydrolysis of β-lactam ring (nucleophilic attack on the β-lactam carbonyl group) causing inactivation of un-biodegradable antibiotics. An alkaline solution (caustic soda) was mixed carefully and was detained for 90 minutes period to complete the chemical reaction. pH value gradually reduced down between 8 to 8.5 due to the formation of CO_2 through bio-oxidation of organic matters during aeration process. Prior to aeration process for biological oxidation of organic matter Di-ammonium phosphate (DAP) was also mixed as a source of nutrients (N & P) to assure seamless bacterial activity.



General Structure of Cephalosporins

Direct Aeration without Seeding - Initially pre-aeration for 24 hours was done to obtain a homogenous mixture of wastewater sample without any seeding. Nearly 22% removal of raw COD has been determined probably due to the removal of Volatile Organic matters.

Addition of Sanitary Sewage - In the laboratory, no growths of organisms were observed even after one week continuous aeration period due to absence of any micro-organisms and high strength of raw sewage. Then it was decided to mix the industrial effluent with 50 percent domestic sewage containing small percentage of sanitary sludge from BUET sewerage system for seeding with micro-organisms. After three and half weeks of continuous aeration process healthy bio-flocs were observed indicating that seeded organisms have been acclimated and bio-oxidation of organic matter are taking place. Then the addition of sanitary sewage gradually reduced and finally stopped completely with the passage of time. However, addition of at-least 20% domestic sewage/sludge should be continued in the field for dilution of raw sewage and to ensure continuous seeding of micro-organisms.

ACCLIMATION FOR BIOLOGICAL PROCESS AND REDUCTION OF AERATION PERIOD WITH EXP. BATCHES

Variation of Aeration Period with Increased Experimental Batch Number – At the beginning of the biological aeration process it took around 48 hours for the acclimation of biological process. It was observed that with the passage of time (increased experimental batch) acclimation of biological sludge with the pharmaceutical samples became more effective and as a result the oxidation rate enhanced significantly and COD value reduced at a faster rate with aeration period. Every time seeding was done through settled sludge obtained from previously aerated wastewater samples. In the 7th and 8th batches of experiment residual COD value reduced down below ECR, 1997 permissible limit of 200mg/l within 8 hours aeration period as shown in figure 1. However, during the initial stage (1st and 2nd batches of experiment) that aeration period was over 48 hours. Therefore 8-9 hours aeration period in the aeration tank could be considered optimum to achieve the desired limit for COD as per ECR, 1997 provided that all above mentioned environmental conditions are satisfied. The desired BOD_5 as per ECR, 1997 of 50 mg/l, however, could not be achieved during this 8-9 hours aeration period indicating that tertiary treatment (chemical treatment) would be required.

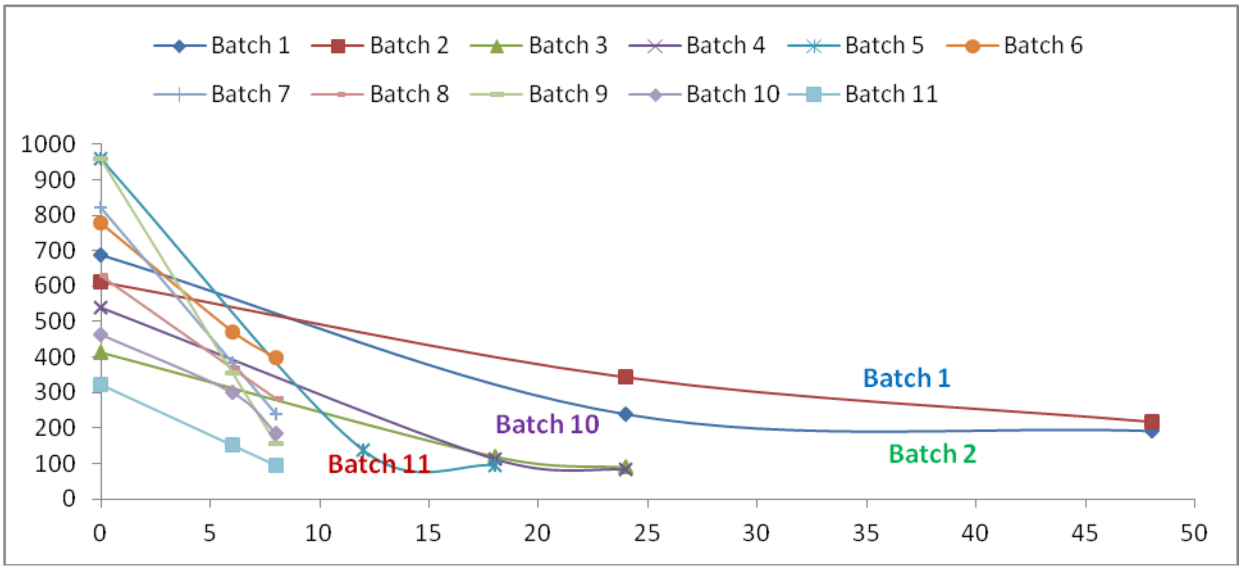


Figure 1: Reduction of COD with different batch of sample

Disruption of Biological Process with Change of Wastewater Quality – When mixed pharmaceutical wastewater with high COD values were supplied and added with the laboratory on going aeration process with Cephalosporin unit wastewater, caused complete disruption of biological aeration process and COD reduction process. To recover the previous experimental conditions mixed pharmaceutical samples were discarded, sanitary sewage and sludge were again added with Cephalosporin unit wastewater. After aeration for couple of weeks again for the acclimation process, the growths of fresh healthy bio-flocs were observed. Therefore, wastewater from other Pharmaceutical units should be treated separately or should be diluted to reduce the organic strength..

EFFECT OF TERTIARY TREATMENT PROCESS ON COD REDUCTION

Chlorination of Aerated Effluent – Chlorination of biologically treated effluent from Cephalosporin unit is a must for complete disinfection of micro-organisms before disposal in the environment. The effect of chlorination on COD reduction have also been determined and a chlorine dose around 20 mg/l and mixing time for an hour have been found adequate to achieve that level. During Laboratory Model Investigation it has been observed both COD/BOD_5 concentrations further reduced down below 30 mg/l .

Activated Carbon Filtration after Chlorination – To bring down the residual chlorine concentration within the reasonable limit of 0.2 to 0.3 mg/l before disposal in the environment, Granular Sand Filtration process followed by Activated Carbon Filtration process would be necessary. In the laboratory model test residual chlorine concentration reduced down to almost zero and complete disinfection of micro-organisms occurred after Activated Carbon Filtration process.

CHEMICAL TREATMENT OF PHARMACEUTICAL WASTEWATER

Exclusive chemical treatments with Alum coagulation process followed by chlorination process have been attempted in the laboratory to determine their effects. Significant COD reduction could be achieved with increased chlorine dose. Removal of color is not very significant; however, pH value reduced due to formation of acid. Exclusive chemical process is comparatively more expensive than biological process followed by chemical process.

SELECTION OF UNIT OPERATION/PROCESSES AND DETERMINATION OF DESIGN CRITERIA

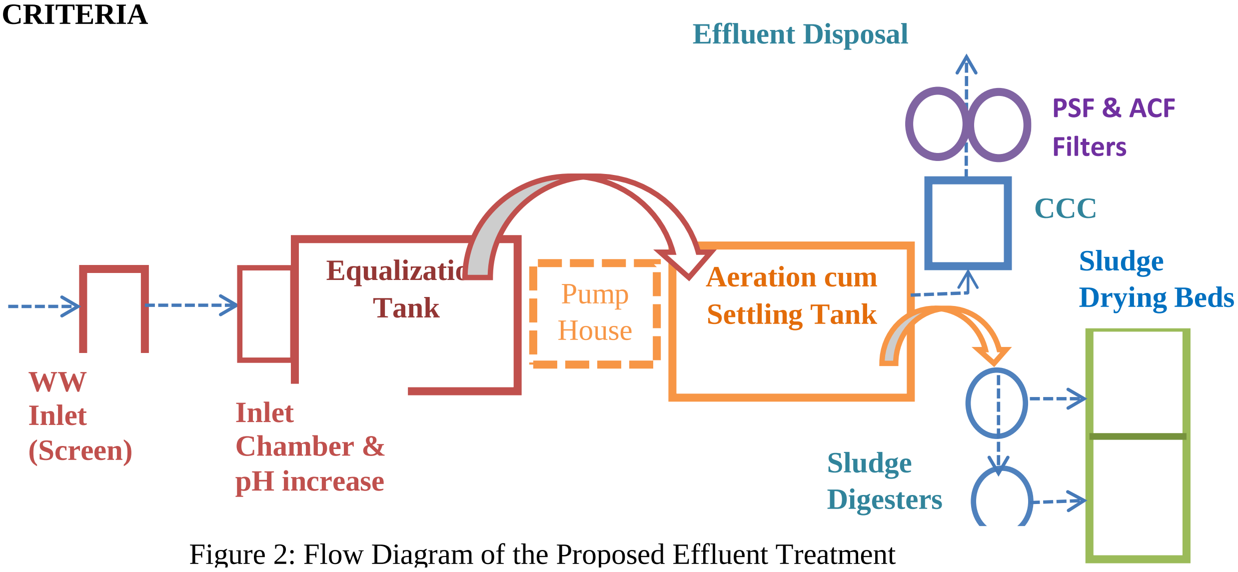


Figure 2: Flow Diagram of the Proposed Effluent Treatment

At the beginning of the treatment processes 3 stages *Screen* should be provided as a safeguard to machineries, pumps, diffusers etc. from any possible large suspended and floating materials. Since present of fine suspended solids are insignificant *Primary clarifier* would not be required for settling of sludge. Alkaline solution (lime/caustic soda) should be added and mixed in the *Inlet chamber* to increase pH value above 10.5 with one hour detention period. Air should be supplied for 24 hour in the *Equalization Tank* for homogenous mixing of incoming wastewater and to keep wastewater with all solids in suspension. It has already been observed that aeration period around 8 hour in the *Aeration tank* would be sufficient enough to achieve the desired ECR, 1997 maximum permissible limits of 200 mg/l for COD. For complete settling of flocculated bio-mass wastewater should be detained for another one hour in the *Aeration cum Settling Tank*.

Settled effluent should be chlorinated with 20mg/l Cl_2 dose and mixing for an hour in a *Chlorine Contact Chamber* (CCC) to oxidize residual organic matter and for complete disinfection process. Finally *Granular Sand Filters* followed by *Activated Carbon Filters* would serve as a polisher of the treated effluent and also for de-chlorination purpose. The average reduction of COD value in different treatment processes has been shown in figure-3.

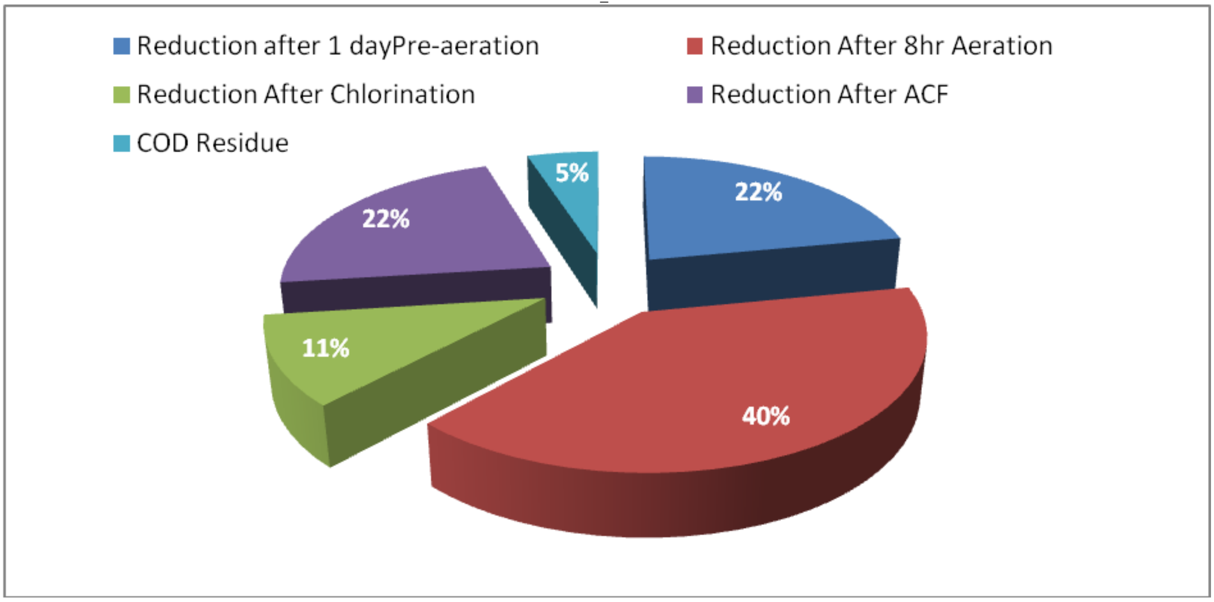


Figure 3: Average COD Reduction of Wastewater Sample

CONCLUSION

Because of the potential health risks associated with cross-reactivity (cross-sensitivity) of beta-lactams, manufacturers should assess and establish stringent controls (including appropriate facility design provisions assuring separation) to prevent cross-contamination. It is expected that if wastewater from Cephalosporin processing Pharmaceuticals units is treated properly in an ETP which (a) has been designed carefully on the basis of laboratory scale model studies result and (b) is operated properly and (c) is maintained carefully, any possible pollution of surrounding environment could be avoided and desired ECR, 1997 limits could be attained. Regular monitoring of effluent quality should be performed establishing an onsite Environmental Monitoring Laboratory with required apparatus and equipment. Presently one such ETP of an reputed Pharmaceutical Industry is in operation and functioning efficiently for the last 5 years. Another ETP of another Pharmaceutical Industry is under construction.

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First of all the authors acknowledge the blessing of Almighty Allah for giving his the opportunity and for enabling him to complete the thesis successfully and peacefully.

The authors concede there profound gratitude and indebtedness to his supervisor Dr. Farooque Ahmed, Professor, Department of Civil Engineering, BUET for his constant supervision, hearty support, continuous guidance, judicious suggestion and encouragement at every stage of the work.

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AIR QUALITY STATUS OF DHAKA CITY AND EFFECTS OF SEASONS, WEEKENDS, EID AND HARTAL DAYS

M.M. Islam[1] & S. Afrin[1*]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <sadiaafrinbuet@gmail.com >*

ABSTRACT

This paper aims at presenting overall air quality scenario of Dhaka, capital of Bangladesh,by assessing yearly, monthly, seasonal as well as diurnal variation of all the criteria pollutant (PM,NOX, SO2, CO and O3), except Pb. Effects of seasons, eid, weekend and hartal days on air quality of Dhaka are also discussed. Trace metals in PM10 are measured in three different locations in Dhaka in both dry and wet seasons in order to find out their seasonal sources. Both yearly 24-hr average PM10 and PM2.5 exceed Bangladesh National Air Quality Standards and WHO Guideline values. The yearly average concentration of NOx also nearly exceeds the standards, while SO2, CO, O3 are far below the standards. Not only PM, but also NOx, SO2, CO show seasonal variation. Only O3 shows distinctive diurnal variation throughout the year. It appears from daily plots that concentration of only PM10 appears to be reduced in weekends while PM10, PM2.5 and NOx are observed to drop immediately on hartal days.Concentration of NOx seems to be abated on eid day and remain lower for the next few days. PM2.5 exhibits strong positive correlation with NOx and CO in both dry and wet periods suggesting same pollution source in all periods. Higher concentration of trace metals in rainy season compared to dry season indicates possible transmission of metals by wind blowing from south-east direction during rainy season, suggesting that the foundries, small industries, Pb-battery recycling plants etc. in old town of Dhaka situated in the south-eastern periphery of the city are contributing to the degraded air quality.

Keywords: Air Quality, Seasonal Variation, Diurnal Variation, Eid, Hartal, Trace Metals

INTRODUCTION

Dhaka, with more than 15 million inhabitants, is one of the 20 megacities in the world (UN HABITAT, 2008) and facing severe urban air pollution problems. Based on publicly available air quality data from 1,100 cities including cities with populations of more than 100,000 people, WHO assessed that Dhaka is among the top 20 cities with the worst air pollution problem (WHO, 2011). Among the major air pollutants, six are identified as causing harmful health effects. As they are causing adverse impact on human health, ambient air quality guidelines have been established for these pollutants. These six pollutants are CO, Pb, NO_2, SO_2, O_3 and Particulate Matter (PM). They are known as “Criteria Pollutants”. Among the criteria pollutants, PM is of great concern due to its strong correlation with health outcomes and more recently due to its high concentrations in Dhaka city exceeding national ambient air quality standards during most parts of the year. PM is typically divided into two “modes”: fine particles with diameters smaller than $_{2.5}$ µm ($PM_{2.5}$), and coarse particles with diameters between $_{2.5}$ and $_{10}$ µm ($PM_{10-2.5}$). For regulatory purposes, PM_{10} is another widely used term, referring to all particles with diameters smaller than $_{10}$ µm. In general motor vehicles, fuel combustion in power plant, heating plant, industries, petroleum refineries, household and commercial wastes, industrial and hazardous waste incineration, re-suspension from roads, brick kilns etc are common sources of these air pollutants. A number of studies have been carried out on source apportionment (Begum et al., 2004, 2005; Biswas et al., 2001; Guttikunda, 2009), elemental characterization (Boman et al., 2005), and spatio-temporal distribution (Begum et al., 2006) of PM in Dhaka city. In this paper, temporal variation of PM and other criteria pollutants has also been investigated.. PM concentrations are also correlated with NO_x and CO to determine the degree of association of these pollutants with PM. At the same time sources of trace metals in PM are also investigated by measuring trace metals in different parts of Dhaka city in both dry and wet season.

Hartals and strikes are very common incidents in our country. Some sources [e.g. vehicle, industries etc.] of pollutants remain inoperative during these days and also in Eid and weekends. Moreover, some sources are seasonal [e.g. Brick-kilns, which are primarily confined to the dry periods (October to March) as current technologies do not allow production during wet periods (April to September)]. Study showing impacts of seasons and these special events on air quality of Dhaka is yet to be done. This study attempts to explain the influence of seasons, hartal day, Eid day, and weekends on the ambient air quality of Dhaka.

MATERIALS AND METHODS

Air quality in Dhaka is monitored systematically at Sangshad Bhaban Continuous Air Monitoring Station (CAMS) since April 2002, which is located in an open, flat area approximately 150 meters away from the heavily trafficked Rokeya Sharani and 300 meters from Manik Mia Avenue. This study is based on this CAMS data. Concentrations of CO, SO_2, NO_x, O_3, CH_4 and non-methane hydrocarbon are measured in CAMS by using multiple gas analyzers. 24 hr average concentrations of $PM_{2.5}$ and PM_{10} are also measured at the CAMS using high volume PM_{10} and $PM_{2.5}$ samplers. Hourly data of NO_x, SO_2, CO, O_3 are available from April 2002 to September 2005 but after that very few monthly data are available. Daily data of $PM_{2.5}$ and PM_{10} are available from April 2002 to May 2004 and monthly data of $PM_{2.5}$ and PM_{10} are available from April 2002 to April, 2014 [monthly data in all months are not available from 2010 to 2014].

Samples are collected in 2 days in different seasons [in dry season (27.02.2011) & in Rainy season (26.09.2011)] from Airport (N 23° 50' 46.8" E 090° 24' 43.5"), Kuril (N 23° 49' 10.3" E 090° 25' 13.3") & Mohakhali (N 23° 46' 46.4" E 090° 23' 55.0") for trace metal concentration determination.

RESULTS AND DISCUSSIONS

Yearly Variation

The time series data of annual average $PM_{2.5}$ & PM_{10} are plotted for the year 2002 to 2014 (Fig. 1). Though traffic volume is increasing with time, there is no significant increasing trend is found for yearly average PM concentrations from 2004 to 2014. But it is alarming that, concentration of both PM fractions exceed Bangladesh National Air Quality Standards (50 $\mu g/m^3$ and 150 $\mu g/m^3$ for PM_{10} and 15 $\mu g/m^3$and 65$\mu g/m^3$ for $PM_{2.5}$ for annual and 24-hour averaging period, respectively) and WHO Guideline values.

Yearly average concentrations of $NO_{X,}$ CO, O_3 and SO_2 are plotted for the available data periods (2002 tov2005) respectively. Fig. 2 depicts that NO_x concentration nearly exceeds the annual ambient standard of 53 ppb. On the other hand, concentrations of SO_2, CO, O_3 are far below the National Air Quality Standards and WHO Guideline values. To know their present status, it is highly recommended to measure concentrations of these pollutants specially NOx in CAMS.

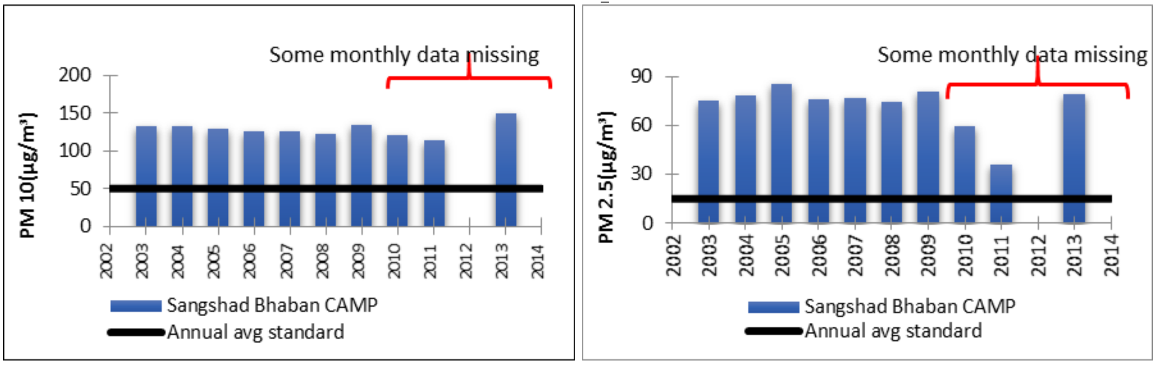

Fig. 1. Yearly average variation of $PM_{2.5}$ & PM_{10} in Dhaka City

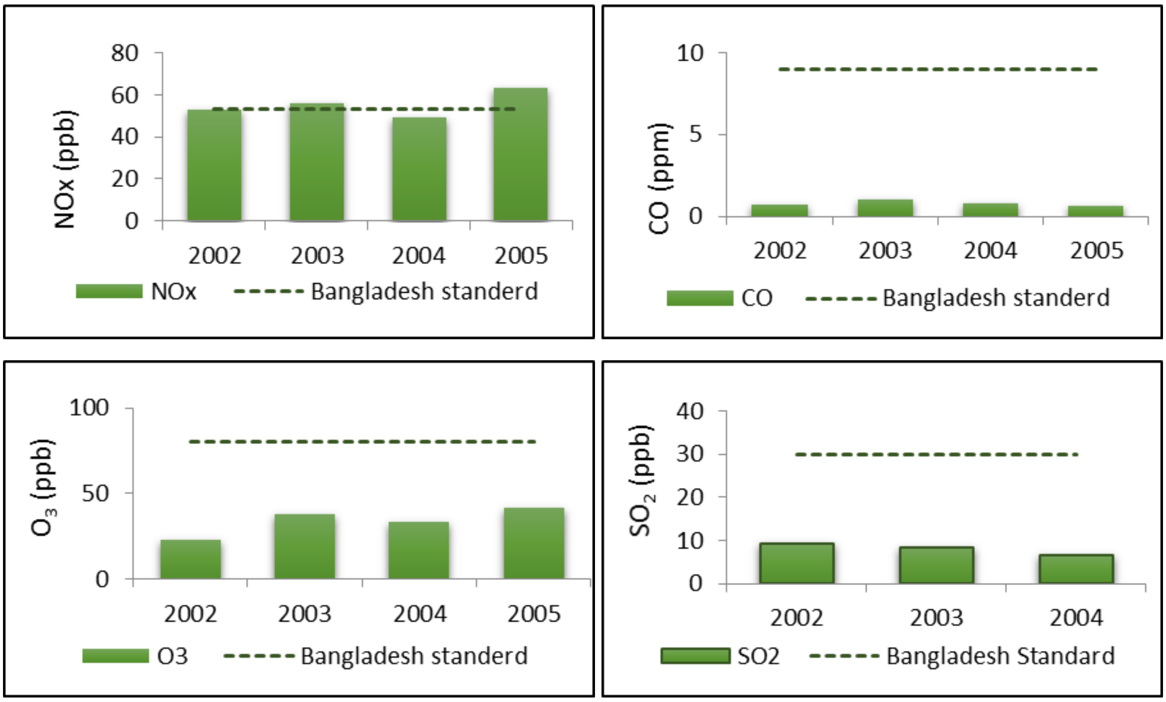

Fig. 2. Yearly average plot of NO_X, CO, O_3 and SO_2

Monthly Variation

Time series plot of monthly $PM_{2.5}$ and $PM_{2.5\text{-}10}$ concentrations during the period 2002-2014 show strong seasonal variation for both PM fractions (Fig. 3 & 4). Concentrations are higher in dry period (Oct-March) compared to wet period (April-September).

Similarly monthly trends of NO_x, SO_2, CO & O_3 dictate that like PM, these are higher during October to March and comparatively lower during April to September. This seasonal variation indicates the possible effects of meteorology and also some seasonal pollution sources. Usually wet deposition due to precipitation and brick kilns are considered responsible for this as brick kilns are also not in operation during rainy season.

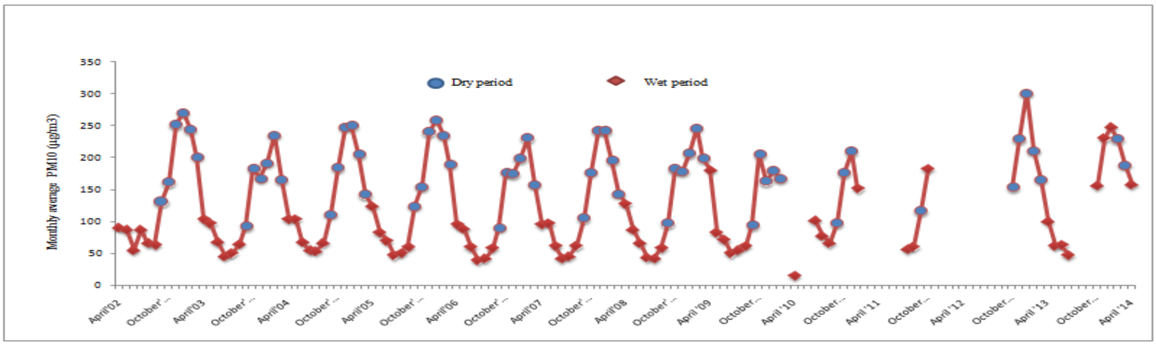

Fig. 3. Monthly average $PM_{2.5}$ trend in Dhaka

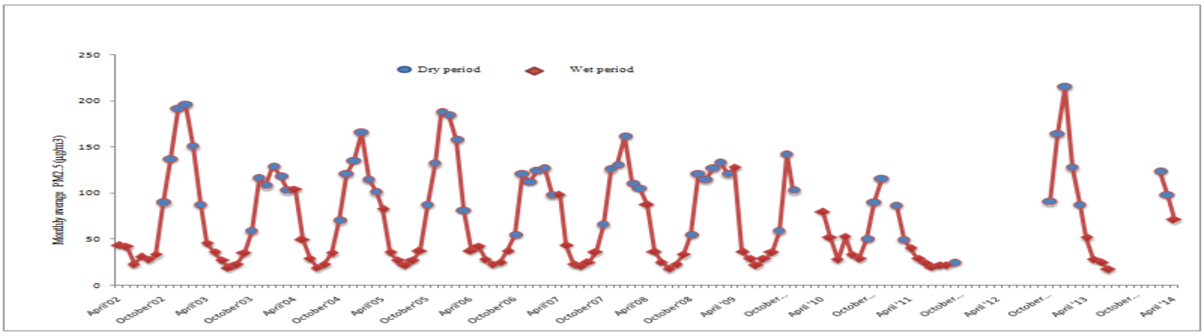

Fig. 4. Monthly average $PM_{2.5}$ trend in Dhaka

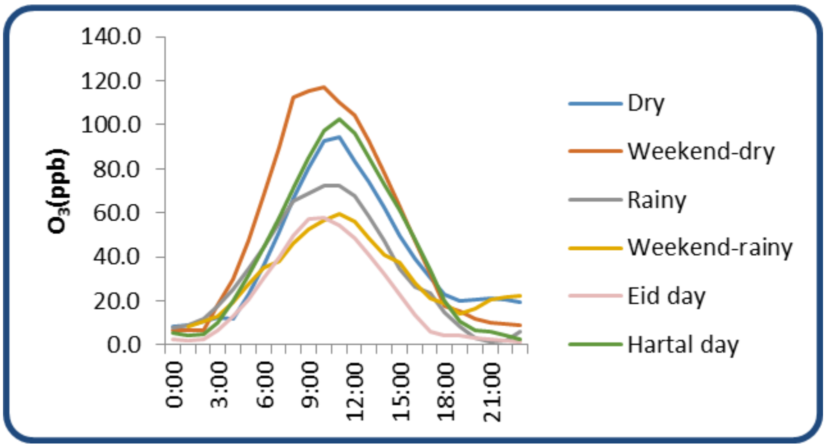

Fig. 5. Diurnal variation in the concentration of O_3

Diurnal variation

Among the pollutants, O_3 exhibits very consistent diurnal variation throughout the year even in eid, hartal and weekends (Fig. 5). At mid night O_3 concentration is very low followed by an increase which attains peak at 11am to 1pm and then decreases gradually. Other pollutants exhibit no such consistent diurnal trend.

Effects of Hartal, Eid & Weekends

The impact of hartal day on pollutant concentration is assessed by plotting daily concentrations in February, 2004. This is mainly because of available daily data at that period and also frequent occurring of hartals in that month (Table 3). It appears from Fig. 6 that $PM_{2.5}$ concentration is remarkably reduced during the hartal days and its effect also continues to the following day. PM_{10} and NOx are reduced in hartals days only. Hartal days are found to have no impact on CO concentrations. Behavior of pollutants for the other hartal days in that year (Table 3) exhibits nearly same trend. Although hartal & strike occurred more frequently in last year of our country, due to unavailability of daily data it could not be possible to assess the recent impact.

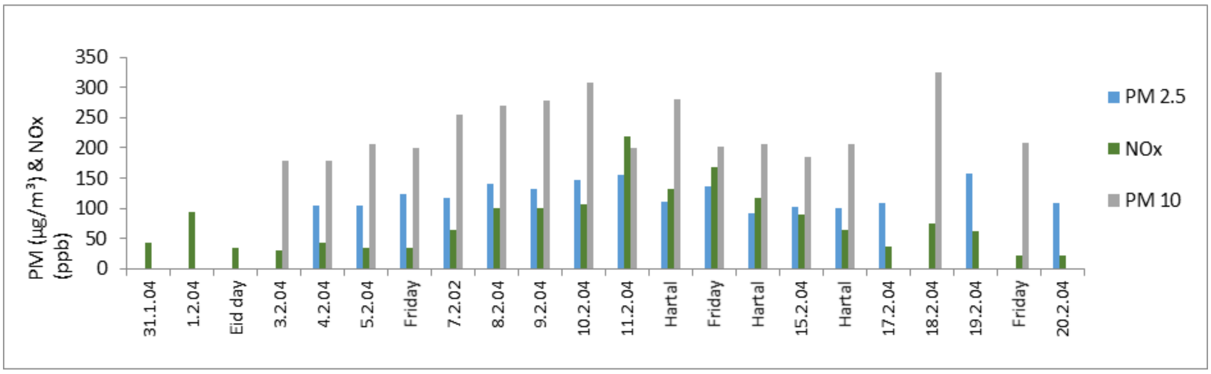

Fig. 6: Effects of Hartal and weekends on PM and NOx

PM_{10} concentration appears to be lower during weekends while $PM_{2.5}$ and NOx show no certain trend. Vehicular movements are not significantly reduced in weekends in Dhaka and this may be the reason of not decreasing $PM_{2.5}$ and NOx concentrations. Only daily concentration data of NOx are available on the period of eid days. Concentration of NOx appears to drop on eid day and remain lower for the next few days also (Fig. 7).

Table 3: Hartal days in the year 2004

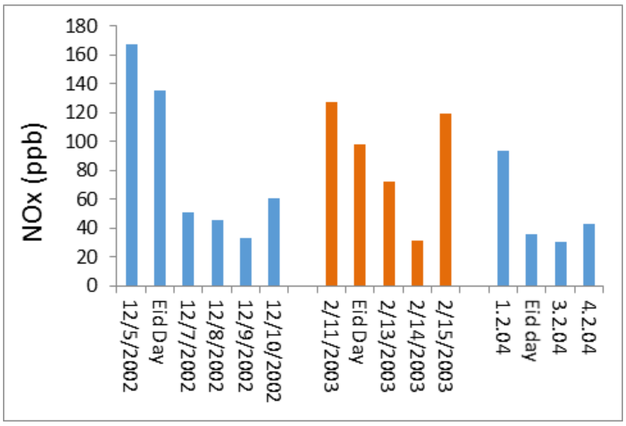

Fig. 7: Effects of Eid days on NOx

Hartal Days	
1/3/2004	4/8/2004
2/12/2004	4/28/2004
2/14/2004	4/29/2004
2/16/2004	5/9/2004
2/28/2004	6/5/2004
3/6/2004	7/3/2004
4/7/2004	7/11/2004

Correlation of PM with NOx and CO

Although $PM_{2.5\text{-}10}$ shows weak correlation, $PM_{2.5}$ exhibits strong correlation with NOx (r =0.64, 0.62) and also with CO (r = 0.69 and, 0.57 for dry and rainy season respectively) in both dry and rainy periods. (Fig. 8) Major anthropogenic sources of $PM_{2.5}$ are brick-kilns, vehicles (Begum et al., 2004, 2005; Biswas et al., 2001; Guttikunda, 2009), whereas vehicles are usually considered as the main source of NO_x in Dhaka city. In dry season all sources of $PM_{2.5}$ and NO_x are contributing and strong correlation between them is expected in this season. In rainy season, one of the major sources of $PM_{2.5}$ (brick-kilns) is not contributing and that's why the correlation of $PM_{2.5}$ with NO_x should not be as strong as dry season. But it is found that correlations remain strong in the rainy season also. That implies that, besides vehicles, brick kilns appear to be a source of NOx and CO also.

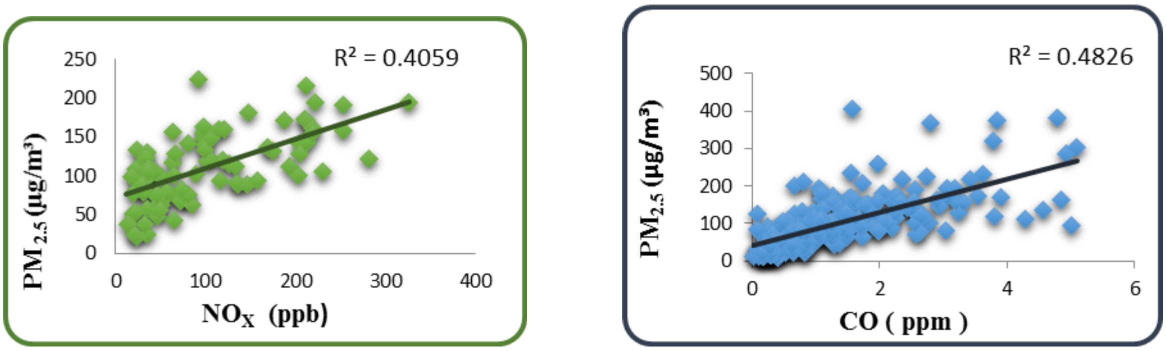

Fig. 8: Correlation of $PM_{2.5}$ with NOx and CO in dry season.

Trace metal in PM

Particulate matter are collected from the three sampling sites named Civil Aviation, Kuril and Mohakhali which are located in the northern side of Dhaka city and tested in the laboratory. The test results (Table 2) show higher concentrations of the trace metals in rainy season compared to dry season for each sites. This is may be the effect of wind as wind blow from south-east direction in rainy season passing through old town of Dhaka (Fig. 9). There are large numbers of foundries, small industries, Pb-battery recycling plants etc. in old town of Dhaka. So there is a possibility that in rainy season winds carry these metals from here to the north direction of Dhaka and consequently metal content increases.

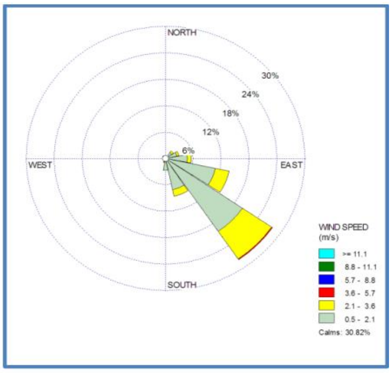

Fig. 9: Wind rose plot for Dhaka in Rainy season

Table 2: Trace Metal content in PM_{10}

Metal (gm/kg)	Air port		Kuril		Mohakhali	
	Dry	Wet	Dry	Wet	Dry	Wet
Fe	8.33	22.63	14.91	26.12	18.34	26.65
Zn	0.00	395.75	16.88	390.42	267.54	308.01
Cd	0.01	0.03	0.02	0.02	0.02	0.00
Pb	0.00	0.73	0.10	2.04	0.06	1.40
Mn	0.00	0.96	0.00	1.88	0.00	0.98
Cu	0.00	0.56	0.00	0.39	0.00	0.47

CONCLUSION:

In this study overall air quality scenario of Dhaka city is presented in yearly, seasonal and diurnal basis. Although the trend of yearly average PM is not rising, PM is found to be alarming as the yearly average PM_{10} and $PM_{2.5}$ exceed both Bangladesh National Air Quality Standards and WHO Guideline values. Among the other pollutants, NOx nearly exceed the standards. Although all pollutants show seasonal variation, only O_3 show constant diurnal variation throughout the year. Except CO, all the pollutants appear to drop in hartal days. NOx seems to be reduced on eid and next few days. Strong positive correlation of $PM_{2.5}$ with NO_x and CO in both dry and wet periods suggesting same pollution source in all periods. It can be inferred from the study that, lack of recent daily or shorter duration data of pollutants is a main constraint for the detail air quality analysis of Dhaka. With availability of these data in future, a more detailed and comprehensive study should be carried out.

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MANGANESE REMOVAL FROM WATER BY FILTRATION THROUGH MANGANESE OXIDE COATED MEDIA

B. Bose[1*], M. R. Islam[1] & M. A. Ali[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <bipsbose@yahoo.com>*

ABSTRACT

Manganese (Mn) is a common natural groundwater contaminant and high intake of Mn has adverse health impacts. Groundwater in many areas of Bangladesh exceeds the Mn standard for drinking water by a large margin. Thus, a cost-effective and efficient technology to remove Mn from groundwater is imperative for Bangladesh. Adsorption onto Mn-oxide coated media is an effective method for Mn removal from water. This research work focuses on understanding the mechanism of Mn removal from water during filtration through Mn-oxide coated media, and simulating Mn removal. Batch experiments were conducted, under controlled environment, to estimate the isotherm constants for describing adsorption of Mn on different filter media (Greensand, Sylhet sand, Synthetic sand).The results show that Mn adsorption increases significantly with increasing Mn content of the filter media; the Freundlich isotherm constant, k increases with Mn content of the media. A model, developed by Zuravnsky (2006) and modified by Ahmed (2013), was used to assess Mn removal from water using different filter media. The model estimated the depth of filter media required to achieve desired level of Mn in the effluent for different initial Mn concentrations and flow rates. It appears that Mn-oxide coated media could be effectively used for Mn removal from groundwater.

Keywords: Freundlich isotherm constants, Mn removal model, Greensand, Synthetic sand, Batch experiments

INTRODUCTION

Manganese is a mineral found abundantly in nature and a cause of growing concern with respect to contamination of groundwater in Bangladesh. WHO (2004) recommended a health-based drinking water guideline value of 0.4 mg/l for Mn; however, in 2011 eliminated it citing that this value (i.e. 0.4 mg/l) is well above concentrations of Mn normally found in drinking water (WHO, 2011). However, the National Hydro-geochemical Survey (BGS and DPHE, 2001) showed that three quarters of the 3,534 wells surveyed in 61 districts in Bangladesh exceeded the national drinking water standard of 0.1 mg/l for Mn and about 41% exceeded 0.4 mg/l limit. Thus, a low cost and efficient treatment technology to remove Mn from groundwater is imperative for Bangladesh.

The main focus of this research is to evaluate removal of Mn from water through adsorption of soluble manganese onto different types of Mn-oxide coated sand. The Mn-oxide coating adsorbs soluble Mn (II) from water and, in the presence of an oxidant, this adsorbed Mn (II) is then oxidized to solid Mn (IV) to create more sites Mn (II) adsorption (Merkle et al., 1997). Ahmed (2013) developed a model, based on the one developed by Zuravnsky (2006), to predict the removal of soluble Mn via adsorption and surface oxidation onto Mn-oxide coated media for steady state conditions under continuous media regeneration by dissolved oxygen (DO). The model was able to predict Mn removal in greensand filter media reasonably well. However, the isotherm constants for Mn adsorption onto different filter media needs to be determined and incorporated in the model to check the validity of the model in predicting Mn removal using different Mn-oxide coated media.

The major objective of this study was to improve understanding of Mn removal mechanisms from water through filtration in different types of Mn-oxide coated media. The specific objectives were:

Determination of the isotherm constants for Mn adsorption onto filter media with different Mn contents.
Applications of a mathematical model to assess the removal of Mn from water by different types of Mn-coated filter media.

MATERIALS AND METHODS

In this study Sylhet sand, Synthetic sand and Green sand were used as filter media. Synthetic sand has been prepared from Sylhet sand following the method of Merkel et al. (1997a, b), using $MnNO_3$ and NaOH (ITN-BUET, 2011). The Sylhet sand consisted of two size fractions; fraction passing #20 sieve and retaining on #30 sieve, and fraction passing #30 sieve and retaining on #40 sieve in the ratio of 1:2. Initial Mn content of the selected size fractions of Sylhet sand, Synthetic sand and Greensand was determined after selective leaching with hydroxylamine hydrochloride (Eley and Nicholson, 1993), and was found to be 5 mg/kg, 25,250 mg/kg and 14,400 mg/kg sand respectively. Figure 1, 2, and 3 shows the different sand media used in this research.

Batch experiments were conducted to estimate isotherm constants for describing adsorption of Mn on the filter media. Groundwater used in experiments was collected from a deep tube well pump station with Mn concentration of 0.022 mg/L. This water was spiked with Mn (II) stock solution to prepare water with different initial Mn concentrations. To several airtight containers (capacity 50 ml) containing water with different initial Mn concentrations, a different mass of Mn-oxide coated sand was added. Initial pH was regulated to be 7±0.1 (close to pH of natural groundwater) and temperature of water was 23±1 ºC. The suspension was then rotated in an end-over-end rotator (Fig. 4) and allowed to equilibrate. The final Mn concentration of water was measured after filtering with filter paper (0.4 μm opening). Final pH (7.3±0.1) and temperature of the water was recorded.

All chemicals used in this research work were of reagent grade. Mn (II) stock solution was prepared by dissolving $MnCl_2.4H_2O$ in deionized water. Samples collected for determination of soluble Mn were acidified with 0.1% concentrated HNO_3. Mn concentrations were measured using an Atomic Adsorption Spectrophotometer (AA-6800, Shimadzu).

Estimation of Isotherm Constants

Freundlich isotherm constants (*K*, *n*) were calculated using data from batch experiments. The equation for a linear trend line fitted to this plot is analogous to the linearized form of the Freundlich isotherm equation (Eq. 1), which allows the values of *K* and *n* to be determined. Freundlich Isotherm constants were verified by plotting the isotherm relationship along with the experimental data.

$$\ln (q) = \ln (K) + (1/n) \ln (C_f) \quad (1)$$

where:

C_f= Final soluble Mn concentration (mg/L)

q = Mn uptake capacity of media (mg Mn/gm media)

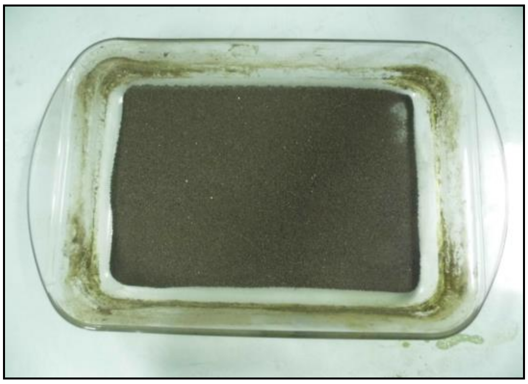

Fig.1: Synthetic sand in a glass tray

Fig. 2: Sylhet sand in a glass tray

Fig. 3: Greensand in a glass tray

Fig.4: Tilted rotator used for batch experiment

Estimation of model parameters

A model (Ahmed, 2013) was used to simulate Mn removal in a column containing Mn-oxide coated media. With the exception of the oxidation rate constant (k_r), all model parameters were calculated or determined from experimental measurements (Table 1). k_r was determined by fitting soluble Mn removal profiles derived from model output with experimental data from the multi-port column (Fig. 5) study. Best-fit values for k_r were determined using a Levenberg-Marquardt nonlinear regression algorithm. The important input parameters in the model are Mn concentration of influent and effluent, initial Mn and DO concentrations, hydraulic loading rate (HLR), Freundlich isotherm constants, and media particle diameter and porosity. Based on the HLR, media particle diameter and porosity, the model uses appropriate relevant values for other parameters such as pore-water velocity, mass transfer coefficient and axial dispersion coefficient (Table 2).

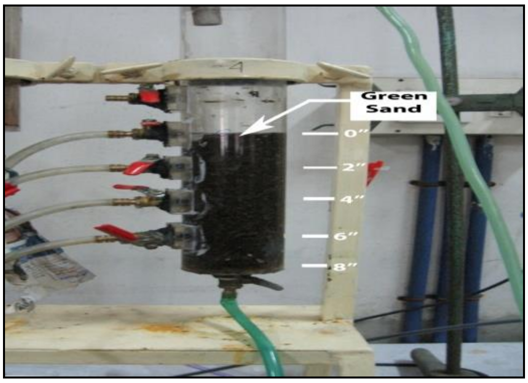

Fig. 5: Multi-port column (x-sectional area = 26.4 cm^2) for model calibration (Ahmed, 2013)

Fractional pore volume (ε_B) was measured as the volume of water that could be contained in a known volume of media bed. The density of the media (kg media/m^3 media) was measured by multiplying apparent specific gravity of the media with unit weight of water. This measurement was converted to bulk density of media (kg media/m^3 bed) by multiplying the media density by the fractional volume of media bed ($1-\varepsilon_B$). Particle diameter, d_p was determined as d_{50} from the sieve analysis results of sand.

Specific surface area of the media (A_v) was calculated using Eq. 2.

$$A_v = 6/d_p^{1.16} \qquad (2)$$

Table 1: Characteristic parameters for media used to determine k_r

Parameter name	Parameter value	Reference
Fractional pore volume, ε_B (m^3 water/m^3 bed)	0.44	Lab. experiment
Bulk density of greensand media, ρ_b (kg media/m^3 bed)	1305	Lab. experiment
Bulk density of synthetic sand, ρ_b (kg media/m^3 bed)	1514	Lab. experiment
Bulk density of Sylhet sand media, ρ_b (kg media/m^3 bed)	1556	Lab. experiment
Specific surface of media for green sand, A_v (m^2 media/m^3 media)	5489	Merkle *et al.*,1997b
Specific surface of media for Sylhet sand, A_v (m^2 media/m^3 media)	36252	Merkle *et al.*, 1997b
Particle diameter of green sand(m)	0.0028	Lab. experiment
Particle diameter of Sylhet sand and synthetic sand (m)	0.00055	Lab. experiment

Table 2: Input parameters in the model

Parameter name	Hydraulic Loading Rate HLR, ml/min/cm^2	Parameter value	Reference
Pore water velocity, U (m/s)	1	3.7891×10^{-4}	Lab. experiment
Axial dispersion coefficient, D_L (m^2/s)		7.0730×10^{-07}	Suzuki (1990)
Liquid to solid mass transfer coefficient, k_f (m/s)		1.5905×10^{-05}	Suzuki (1990)
Pore water velocity, U (m/s)	2	7.5782×10^{-4}	Lab. experiment
Axial dispersion coefficient, D_L (m^2/s)		1.4146×10^{-06}	Suzuki (1990)
Liquid to solid mass transfer coefficient, k_f (m/s)		2.0039×10^{-05}	Suzuki (1990)
Pore water velocity, U (m/s)	3	11.3673×10^{-4}	Lab. experiment
Axial dispersion coefficient, D_L (m^2/s)		2.1219×10^{-06}	Suzuki (1990)
Liquid to solid mass transfer coefficient, k_f (m/s)		2.2939×10^{-05}	Suzuki (1990)
Pore water velocity, U (m/s)	4	15.1565×10^{-4}	Lab. experiment
Axial dispersion coefficient, D_L (m^2/s)		2.8292×10^{-06}	Suzuki (1990)
Liquid to solid mass transfer coefficient, k_f (m/s)		2.5247×10^{-05}	Suzuki (1990)

Initial bulk aqueous-phase soluble Mn (mol/m^3) and initial DO (mol/m^3) were measured with each soluble Mn profile collected experimentally. Axial dispersion coefficients, D_L (m^2/s) were calculated using Eq. 3 as developed by Suzuki (1990), which uses Peclet number, P_e (dimensionless) to calculate D_L. Since the experimental study was under laminar flow regime (Reynolds number, $R_e < 100$), Peclet number was determined as 1~2 for $d_p \geq 2$ mm, and $1.2d_p$ (mm) for $d_p < 2$ mm (Suzuki, 1990).

$$D_L = (d_p.U)/P_e \quad (3)$$

Where d_p = particle diameter (m)

RESULTS AND DISCUSSION

The results of the batch experiments with different filter media were used to determine Freundlich isotherm constants for sand media from a log-log plot (as shown in Fig. 6) of the uptake capacity data. Figure 7 and Figure 8 show the relationship between isotherm constants and the filter media. It can be seen that as initial Mn content of the media increases, value of k increases.

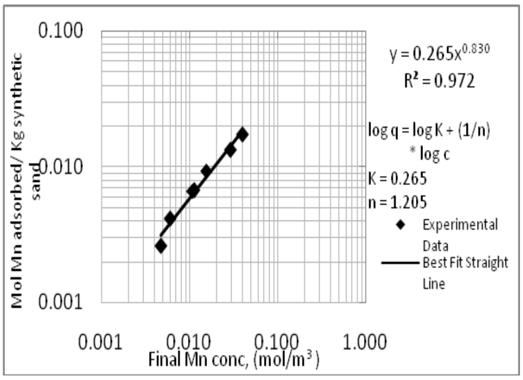

Fig. 6: Mn adsorption capacity per weight of Synthetic Mn-oxide coated media using groundwater at pH=7.0±0.1 over a range of soluble Mn concentrations.

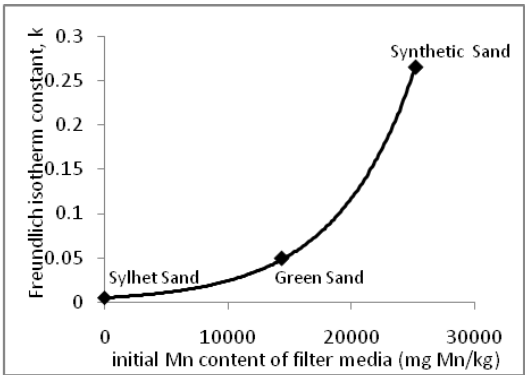

Fig. 7: Freundlich Isotherm Constant, k vs Initial Mn content of filter media (mg Mn/kg)

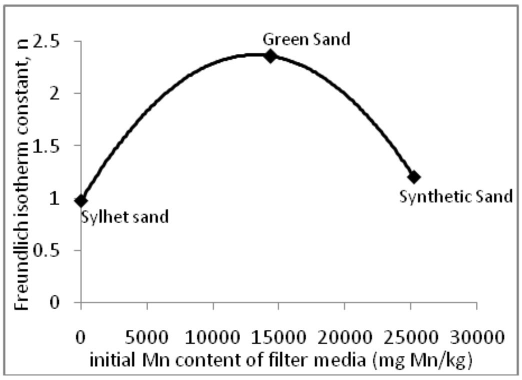

Fig. 8: Freundlich Isotherm Constant, n vs Initial Mn content of filter media (mg Mn/kg)

Figure 9 shows the effect of Mn-content of media on Mn adsorption. Sylhet sand with low Mn content (5 mg/kg) was found to have very low adsorption capacity, while Synthetic sand with high Mn content (25,250 mg/kg) demonstrated high adsorption capacity. These results are in agreement with those reported by Bouchard (2005) and Zuravnsky (2006). Figure 10 and 11 show the effect of flow rate and depth of media on removal of Mn from water for Greensand and Synthetic sand, respectively. k_r values for different flow rates were predicted using the Equation $k_r = 6x10^{-05}x^{0.444}$ for Greensand, and $k_r = 2.12x10^{-04}x^{0.063}$ for Synthetic sand; where x represents flow rate. Figure 10 shows that a 20.3 cm column of Greensand can effectively remove Mn to meet the Bangladesh standard of 0.1 mg/L at a flow rate of 1 ml/min/cm^2. At higher flow rates, higher depth of media is necessary to meet the

Bangladesh standard. Comparison of Fig. 10 and Fig. 11 shows that Synthetic sand is much more effective in removing Mn from water. Figure 12 shows the effect of influent Mn concentration on Mn removal at different flow rates. It shows that a 30.48 cm Greensand column can effectively reduce Mn concentrations to meet the Bangladesh standard for influent Mn concentrations of up to 6 mg/L.

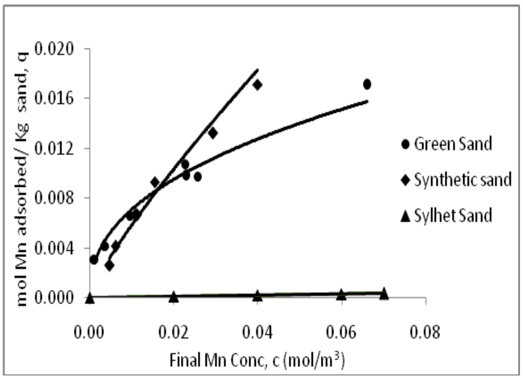

Fig. 9: Effect of initial Mn-content of filter media on Mn removal from water

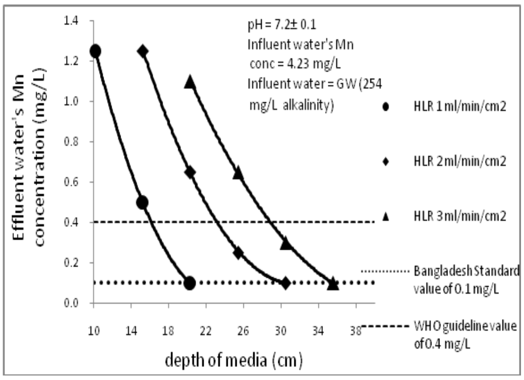

Fig. 10: Effect of flow rate and media (Greensand) depth on Mn removal from water

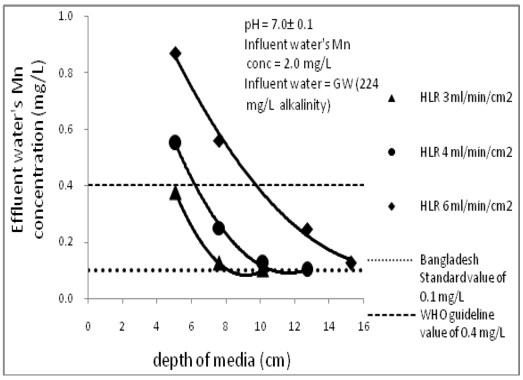

Fig. 11: Effect of flow rate and media (Synthetic sand) depth on Mn removal from water

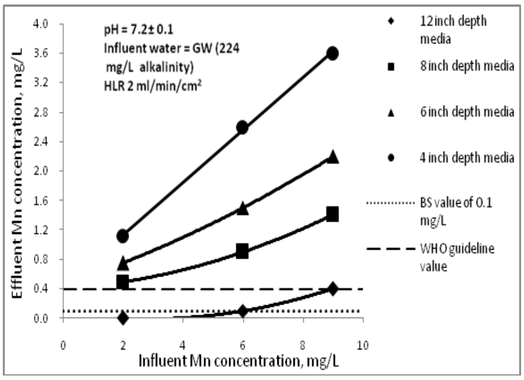

Fig. 12: Effect of influent Mn concentration and depth of media (Greensand) on removal of Mn from water

CONCLUSION

The following conclusions were made based on the results obtained in this research.

Higher Mn content of the media promotes higher adsorption of dissolved Mn onto the media.
As the Mn content of the filter media increases, the value of k, the Freundlich isotherm constant also increases, although the value of n did not show a similar trend.
Mn-oxide coated filter media can be effectively used for removal of Mn from water. Removal primarily depends on depth of media and flow rate through the media.

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PERFORMANCE EVALUATION OF EXISTING COMMUNITY LEVEL ARSENIC REMOVAL PLANTS FOR ARSENIC FREE DRINKING WATER SUPPLY IN JESSORE AND JHENIDAH DISTRICTS OF BANGLADESH

Prianka Ghosh[1*], Sayka Jahan[1], Tapos Kumar Chakraborty[1], Md. Abu Shamim Khan[2] & Gopal Chandra Ghosh[1]

[1] *Department of Environmental Science and Technology, Jessore University of Science and Technology, Jessore 7408, Bangladesh.*
[2] *Asia Arsenic Network, Jessore 7400, Bangladesh.*
**Corresponding Author, <taposchakraborty@rocketmail.com>*

ABSTRACT

The use of groundwater as drinking water in Bangladesh is favoured by its easy availability, microbial safety and absence of proper infrastructure for treatment and distribution of surface water. As a result, millions of people are affected by widespread arsenic poisoning through drinking water drawn from underground sources containing arsenic at concentrations well above the permissible limit of 50 mg/L. Since 2000, hundreds of community level arsenic removal plants have been installed in south-west region of Bangladesh. However, the performance of the plants over time is hindered by lack of information due to absence of long term water quality monitoring information. The objective of this study is to evaluate the performance of existing community level arsenic removal plants. In this study, we selected five arsenic removal plants (four plants were Arsenic Iron Removal Plant, namely AIRP; and one Granular Ferric Hydroxide Based Arsenic Removal Unit, namely SIDKO) located in Jessore and Jhenidah district. All AIRPs achieved the Bangladesh standard for arsenic in drinking water of 50 mg/L and the SIDKO achieved the World Health Organization (WHO) guideline of 10 mg/L. The AIRPs removed 64% of influent arsenic on average. However, the SIDKO removed 80 % of influent arsenic. Treated water quality parameter (such as pH, EC, TDS, Fe, NH3) of the plants were within the WHO standards, except NO3- (0.1 to 7.4 mg/L), and NH3 (0.01-1.89 mg/L) for long term uses.

Keywords: Arsenic, Ground water, Arsenic removal plants, Drinking water.

INTRODUCTION

Arsenic is a toxic, poisonous and carcinogenic metalloid, which is ubiquitous in rock, soil and water (Morton et al., 1994). High concentrations of arsenic in groundwater have been found in many environmental conditions originating from natural processes and from anthropogenic sources. Natural occurring arsenic in ground waters associated with geothermal activity is recognized to be significant (Webster et al., 2003). In Bangladesh alone, 57 million people are exposed to arsenic levels of up to 3200 g/L (BGS, 2001), well in excess of the maximum contaminant level (MCL) recommended by the World Health Organization of 10g/L (WHO, 1993).Recent measurements show that in many parts of the Ganges and Bhrahmmaputra basin more than 60% of the shallow and deep tubewell water contains arsenic above the WHO guideline value of 10 mg/L and more than 30% of the tubewells contains arsenic above the Bangladesh standard of 50 mg/L (Chatterjee, et al., 1995; Das et al., 1995). The prolonged drinking of this water has caused serious health hazard in the form of hyperkeratosis on the palms and feet (Choudhury et al., 1998). Long term exposure to low concentrations of arsenic has been reported to cause cancer of bladder, skin and other internal organs (International Agency for Research on Cancer, 1980). The health hazard caused by drinking arsenic affected water can be arrested by drinking arsenic free water because the biological half-life of arsenic appears to be between ten hours and four days (National Research Council, 2000). Drinking As-free water is no doubt the best option to alleviate its toxicity and several options been proposed in this regard, viz.

treatment of surface water by low-cost methods, rain water harvesting, drinking water from deep aquifers and treatment of As contaminated tube-well water etc. All these options require major technological innovation in water supply except the latter one, through which huge number of tube-wells likely abandoned can easily be revitalized. Methods for removal of As from water have been highlighted in a number of papers (Cheng et al., 1994; Joshi et al., 1996; Hering et al., 1996; Hug et al., 2001 & Zaw et al., 2002) .The most common arsenic removal technologies can be grouped into the following four categories: Oxidation and sedimentation, Coagulation and filtration, Sportive filtration, Membrane filtration (Cheng et al., 1994; Hering et al., 1996; and Joshi et al .,1996). In the process of coagulation and flocculation, arsenic is removed from solution through three mechanisms: Precipitation: The formation of insoluble compounds, Co-precipitation: The incorporation of soluble arsenic species into a growing metal, hydroxide phase, Adsorption: The electrostatic binding of soluble arsenic to external surfaces of the insoluble metal hydroxide (Edwards, 1994). During the last few years a number of low-cost household As removal technologies in context of Bangladesh have been developed (Safiullah et al., 2000; Khan et al., 2000a; 2000b; Crisp et al., 2001 & Meng et al., 2001) and some field based evaluation have also been done and some evaluation has already done (Tahura et al., 2001; Munir et al., 2001;Sutherland et al., 2001; 2002). In this study, evaluate the performance of two indigenous Arsenic removal filters namely Arsenic Iron Removal Plant (AIRP), and SIDKO arsenic removal plant. The evaluation method was conducted by measuring water chemistry parameters such as pH , Nitrate (NO_3^-), Iron(Fe++), Phosphate (PO_4^{3-}),Ammonia (NH_3), Total Dissolved Solid (TDS) , and Electrical Conductivity(EC), finally these parameter were compared with WHO drinking water standard.

MATERIALS AND METHODS

Overview of the Investigated Plants

Arsenic Iron Removal Plants (AIRP):

The conventional small-community type iron removal plants [Fig.1], which operate on the principles of aeration of ferrous iron to convert them to ferric iron to co-precipitate arsenic. Groundwater has drawn by hand tube -well drops into storage (aeration/ sedimentation) chamber for oxidation of iron and arsenic with air to co-precipitate. Water from storage chamber passes through filtration chamber due to the pressure head of aeration/ sedimentation chamber and subsequently collected into a storage tank for public uses. Filtration media comprises of brick chips, charcoal and sands. Filtration media is periodically (3 to 4 times a year) back washed, and sludge is collected in a holding pond (Ahmed, MF et al; 2000).

SIDKO Arsenic Removal Plant:

Granular ferric hydroxide (AdsorpAs®) is a highly effective adsorbent used for the adsorptive removal of arsenate, arsenite, and phosphate from natural water. It has an adsorption capacity of 45g/kg for arsenic and 16 g/ kg for phosphorus on a dry weight basis (Pal, 2001). M/S Pal Trockner (P) Ltd, India, and Sidko Limited, Bangladesh, have installed several granular ferric hydroxide-based arsenic removal units in India and Bangladesh. The proponents of the unit claim that AdsorpAs® has very high arsenic removal capacity, and produces relatively small amounts of residual spent media. The typical residual mass of spent AdsorpAs® is in the range of 5–25 g/m3 of treated water. The typical arrangement of the Sidko/Pal Trockner unit [fig. 2] requires aeration for oxidation of water and pre-filtration for removal of iron flocs before filtration through active media. Chemi-Con and Associates has developed and marketed an arsenic removal plant based on adsorption technology in which crystalline ferric oxide is used as an adsorbent. The unit has a pre-filtration unit containing manganese oxide for oxidation of As(III) to As(V) and retention of iron precipitates (Pal, 2001).

Fig. 1: Arsenic Iron Removal Plant

Fig. 2. SIDKO

Sample collection, preparation and analysis for As and other parameters estimation.

Sample collection: Water samples were collected from randomly selected Arsenic Iron removal Plant (AIRP), and SIDKO Arsenic removal filter installed at Jhenidah and Jessore District.(Detail Table 1)

Table 1: Sample collection system in the study area.

Sampling location	Sampling point	Selected plants name	Sampling frequency
Barabazar, Kaligonj, Jhenidah	Majdhia village	AIRP1,AIRP2,AIRP3	3×2=6
Churamonkati, jessore sadar, jessore	Shymnagor village	SIDKO	1×2=2
Phulsara, Chugacha, jessore	Phulsara village	AIRP4	1×2=2

Sample Preparation: For the arsenic and iron test 2ml conc.HNO3 acid was mixed with 100ml sample water and the rest samples water (400ml) were kept for testing other parameters.

Sample analysis: Arsenic was estimated by Atomic Absorption Spectrophotometer [(Shimadzu (Japan) Model: - AA-6200 Range: - 0.01 to 10 ppb (As)]. Nitrate (NO3) was estimated by Cadmium reduction Method from HACH DR/2010 spectrophotometer, USA, Range: 0 – 4.5 mg/L (NO3). Phosphate (PO43-) and Ammonia (NH3) was estimated by Powder pillows method no: 8048, and Powder pillows method no: 8038 from HACH DR/2700 spectrophotometer, USA. Range: 0.02-2.50 mg/L. Iron (Fe++) was estimated by Atomic Absorption Spectrophotometer, Shimadzu (Japan) Model:- AA-6200 Range:- 0.01 to 05.00 ppm (Fe++) . Total Dissolve Solids (TDS) and Electrical Conductivity (EC) were estimated by Electrode method from HACH Sension -156 multi parameter. USA Model: 156. Electrode Model: 51975. Electrode type: Conductivity probe combination with temp. pH (Hydrogen Ion Concentration) was estimated by MARTINI instruments, pH 56 pHWP.

RESULTS AND DISCUSSIONS

Arsenic and Iron: The Arsenic removal efficiency of SIDKO was better (80%) than AIRP1 and AIRP2 was (70.50%) and (78.62%) but in case of Iron removal AIRP1 (86.16%) and AIPR2 (85.25%) is comparatively good then SIDKO (17.14%) because in SIDKO raw water iron concentration is very low than AIRP1 and AIRP2. In case of arsenic removal AIPR3 (61.82%) and AIRP4 (44.44%) is lower than other plants and iron is negatively removed by AIRP3 (-21%) due to lower operation and maintenance.

TDS and Electric Conductivity:. The performance of SIDKO (28.73%) was higher than other AIRP plants (18.93%, 6.67%, 1.93%, and 14.39%) because both parameters concentration in raw water was higher than other plants raw water concentration.

pH: In case of pH all plants were removed negatively because the pH of the filtered water increased by one unit, possibly a result of decarbonation. This is also evident from the decrease in bicarbonate concentration.

Table 2: Removal efficiency of SIDKO and AIRP

Parameters	$AIRP_1$ %	$AIRP_2$%	$AIRP_3$%	$AIRP_4$%	SIDKO %
PH	-3.44	-1.84	-2.08	-6.63	-0.123
Nitrate (NO_3^-)	66.44	-66.11	99.56	0	89.47
Phosphate (PO_4^{3-})	-36.84	-36.36	35	-100	84.11
Electric Conductivity (EC)	18.91	6.67	2.09	14.17	28.57
Ammonia (NH_3)	76.38	13.30	97.87	95.23	93.75
Total Dissolved Solid (TDS)	18.93	6.67	1.93	47.39	28.73
Arsenic (As)	70.50	78.62	61.82	44.44	80.00
Iron (Fe++)	86.18	85.25	-21	0	17.14

Ammonia and Nitrate: Except AIRP2 (13.30%) plant, other plants AIPR1 (76.38%), AIRP3 (97.87%), AIPR4(95.23%) and SIDKO(93.75%) performance was good for Ammonia removal and in case of Nitrate removal AIRP1 (66.44%), AIRP3 (99.56%) and SIDKO (89.47%) was good but AIRP2 (-66.11%) was negatively removed because irregular cleaning of filter media increase ammonia aeration as a results nitrate concentration are increased in treated water.

Phosphate: For phosphate removal SIDKO (84.11%) was good but other AIRP plants was very bad. Lower operation and maintenances is responsible for this performance.

Treated water quality parameter (such as pH, EC, TDS, Fe, NH_3) of the plants were within the WHO standards, except NO_3 (0.1 to 7.4 mg/L) and NH_3 (0.01-1.89 mg/l) for long term uses (Table 3).

Table 3: Drinking water quality parameters: Comparison of SIDKO and AIRP water with World Health Organization (WHO).

Parameters	WHO standard	Raw water	Treated water	Filters Name	Parameters	WHO Standard	Raw water	Treated water	Filters Name
PH	6.5-9.2	7.84	8.11	AIRP1	Electric Conductivity (EC)	2000 us/cm	751	809	AIRP1
		8.14	8.29	AIRP2			720	672	AIRP2
		8.16	8.33	AIRP3			621	808	AIRP3
		7.84	8.36	AIRP4			847	727	AIRP4
		8.07	8.08	SIDKO			1085	775	SIDKO
Iron	0.3-1.0	3.91	0.54	AIRP1	Total	1000	375	304	AIRP1

(Fe^{++})	mg/L	3.12	0.46	AIRP2	Dissolved Solid (TDS)	mg/L	360	836	AIRP2
		1.0	1.21	AIRP3			311	305	AIRP3
		0.30	0.30	AIRP4			424	363	AIRP4
		0.35	0.29	SIDKO			543	387	SIDKO
Phosphate (PO_4^{3-})	5.6 mg/L	0.19	0.26	AIRP1	Ammonia (NH_3)	0.5 mg/L	1.27	0.30	AIRP1
		0.3	0.45	AIRP2			2.18	1.89	AIRP2
		0.71	0.46	AIRP3			0.47	0.01	AIRP3
		0.11	0.22	AIRP4			1.26	0.06	AIRP4
		1.07	0.17	SIDKO			0.32	0.02	SIDKO
Nitrate (NO_3^-)	45 mg/L	7.45	2.5	AIRP1	Arsenic (As)	50PPb	130.5	38.5	AIRP1
		4.25	7.06	AIRP2			203.5	43.5	AIRP2
		2.25	0.01	AIRP3			27.5	10.5	AIRP3
		0.01	0.3	AIRP4			9	5	AIRP4
		1.9	0.2	SIDKO			20	4	SIDKO

CONCLUSION:

Many people in the study area relies on the AIRP and SIDKO Arsenic removal plant because in studied area are Arsenic affected. Performance of the AIRP and SIDKO Arsenic removal plant is somewhat dependent on the operation and maintenance and continuous monitoring. Removal efficiency of different parameters was highest in SIDKO than AIRP because it is newly constructed and its operation and maintenance occurs regularly than AIRP plant. All of those performances are satisfactory because each plant fulfill the criteria of WHO drinking water quality standard. For ensuring safe drinking water in long future different types of government and non-government should come forward creating awareness or consciousness among local community about the proper operation and maintenance of existing Arsenic removal plants.

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PROSPECTS OF DIAPER DISPOSAL AND ITS ENVIRONMENTAL IMPACTS ON POPULATED URBAN AREA LIKE DHAKA CITY

S. H. Rahat[1*], A. T. Sarkar[2], S. A. A. Rafie[2], S. Hossain[3]

[1] *Dept. of WRE, BUET, Dhaka, Bangladesh.*
[2] *Dept. of Civil Engineering, BUET, Dhaka, Bangladesh.*
[3] *Institute of Water Modeling, Dhaka-1206, Bangladesh.*
[*] *Corresponding Author, < rahat.shr@gmail.com>*

ABSTRACT

Diapers are made of cloth or synthetic disposable materials which allows for defecation or urination in a discreet manner. Disposable diapers contain absorbent chemicals and are thrown away after use, remaining un-biodegraded in the disposal sites. An average human baby initiates the use of 8,000 such diapers during the first few years of life with each diaper taking half a millennium to completely decompose. Additionally, since improper disposal of nitrogenous waste is a major factor in the spread of infectious diseases, the dumping of one- use diapers is an issue of great concern with regards to solid municipal waste management. This article deals with the disposal of diapers as a solid waste in the context of Dhaka and is an attempt to identify locations where diaper pollution is most prevalent. Other goals incorporated include data collection with regards to the lack of awareness concerning the issue of diaper disposal and finding possible crosslinks between pollution effects associated with improper diaper disposal. This article contains a preliminary feasibility analysis and a design suggestion for cost effective, replaceable, environmentally friendly Diaper changing stations in locations requiring it the most.

Keywords: Diapers, Recycling process, Dhaka City

1. INTRODUCTION

Disposable diapers are absorbent products for personal hygiene designed to absorb and retain urine and feces from babies or from adults with incontinence problems. An unused disposable diaper generally consists of (EDANA,2008): (i) a liquid-permeable membrane lining the inside surface, made of non-woven polymer; (ii) a watertight membrane on the outer surface made from polymers, starch, woven cloth or rubber; (iii) an absorbent core (pulp fluff) made up of a fibrous material (cellulose, hemp or synthetic materials) enclosed in water-resistant paper; (iv) the absorbent part also contains a super-absorbent polymer material (sodium polyacrylates), which has a high capacity for bonding with water, making it possible to retain urine within the absorbent part. The efficiency of a diaper is highly dependent on its capacity to absorb and retain urine; (v) finally; a diaper also contains minor amounts of tapes, elastics and adhesive material. With the rise of industrialization and urbanization, solid waste generation and its management has a significant impact on the society. In Dhaka city more than 4,750 tons of solid wastes are generated every day. Of the total waste produced, nearly 20% is used for recovery and recycling and about 37% remains scattered laying around on roadsides, Residential and commercial sources produce most of the total municipal waste. The JICA study shows 63% (2120 tons/d) of the total waste (3,340) is from residential sources while another project of the Dhaka City Corporation (DCC) estimated that it is 49.08%. The use and dispose of diapers have increased due to the rise in household income and for its utility. An average human baby uses 4-5 diapers a day which weighs 210g. The percentage of diapers in solid waste may be very little but the environmental effect can hardly be denied. Disposable diapers mainly consist of organic compound. There is an absence of landfill directives in Dhaka city so, it is usually collected with household waste and disposed of in the landfill or incineration. The landfilling of such biodegradable contents is highly risky because of the emission of methane, possible percolation of leachate in the groundwater, land occupation. Some states of the US and some countries have determined effort to keep the disposable diapers away from the landfill for its perceived impact on the landfill space. Even so very little work has been done in the past due to lack of cost-effective and widely acceptable recyclable methods. It has been attempted to identify locations where diaper disposal is most prevalent by means of infant population and analysis of other related data. The data basis provides an understanding and future prediction of the infant density in wards. An online survey was conducted with regards to the lack of awareness concerning the issue of diaper disposal. Numerous assessment methods were approached in this study to evaluate the best possible method of disposal of diapers.

2. CURRENT PROSPECTS OF DIAPER DISPOSAL

The composition of baby diaper has evolved in past few decades. The technology used has greatly changed; the size has become smaller yet efficient. Economic growth spurred increased the use of diapers in Bangladesh. Bangladesh mostly imports diapers from Australia, China, Algeria, Iran, Egypt and Japan etc. Two local manufacturer, Incepta pharmaceuticals and Bashundhara contribute small but significant amount to the diaper market recently. Currently the annual market for baby diapers stands at nearly Tk 300 crore, registering more than 40% growth a year. Bangladesh is expected to be one of the biggest markets for baby diapers by 2020, according to the industry. Given the composition and how it is discarded makes it harder to dispose. Though the cellulosic materials are considered as biodegradable, their assimilation by the environment takes a few years to few hundred years.

Nevertheless disposal diapers have a large carbon footprint. According to the Environment Agency, a disposal diaper is responsible of 630 kg of greenhouse gas, which is equal to an average car driven 1800 miles. The most obvious impact of disposable diapers on environment is that they are thrown way piling up garbage every day. Another impact is that when these are discarded along with untreated feces and urine which is forbidden by the World Health Organization (WHO). Those human feces can leach causing contamination and spread communicable disease when disposed in the landfills. While most disposal diapers can decompose within five months as they are wood product or cotton, but the superabsorbent gel and the plastic need at least 500 years to decompose. An integrated system needs to include an optimized waste collection system and efficient sorting, which is then followed by one or more of the options below in order to recover value as materials, organics or energy prior to landfilling the residues:

Materials recycling, which will require access to reprocessing facilities?
Biological treatment of organic materials to produce marketable compost and to reduce volumes for disposal
Thermal treatment such as incineration which will reduce volume render residues inert and should include energy recovery
Landfill which will increase either amenity via land reclamation or will through well engineered sites, at least minimize pollution and loss of amenity.

In addition, the environmental benefits for separate recycling of a relatively small waste fraction such as baby diapers and incontinence products are questionable and would be disproportionately small in relation to the economic costs.

3. LITERATURE REVIEW

The following is a short description on studies related to diaper disposal and diaper changing stations in the past around the world. Unfortunately, very little work has been done in this regard in Bangladesh and there is no published report up until now. Disposable diapers, though not the largest, are the third largest individual constituent of municipal solid waste accounting for probably somewhere between 1.5% and 4% of the total (Taylor and Allen, 2004). Feasibility analysis of composting baby diapers has also shown plenty of promise as per the literature reviewed since currently, diapers cannot be recycled. At present, diapers are generally not collected separately and are disposed of as solid municipal waste for further treatment, the majority of them ends up in landfills or incinerators (Mirabella et al., 2013) and, to some extent, by composting (Colón et al., 2010, 2013) or anaerobic digestion.. The collection and treatment of compostable diapers jointly with the source-separated organic fraction of the municipal solid waste does not imply any change in the operation and process of composting of this municipal solid waste stream (Colon et al, 2013). Moreover the cultivation of P. ostreatus (biological decomposing agent) in used disposable diapers for the treatment of this urban waste leads to a reduction of more than 85% of the mass and volume of waste (Espinosa-Valdemar et al., 2011).

Life Cycle Analysis reports undertaken between comparing single-use disposable diapers with reusable cloth diapers and/or compostable diapers. Disposable diapers are shown to generate significantly more solid waste, to consume greater quantities of energy and raw materials, and to generate more potentially toxic pollutants on a per-diaper-change basis. Although using disposable diapers generate notably more carbon monoxide and particulate air emissions, both single-use and reusable diapers produce nitrogen oxide, sulfur oxide and hydrocarbon emissions in similar ranges (Lehrburger et al, 1991). Some of the other key aspects of the LCA report by Lehrburger, Mullen and Jones are:

Disposable diapers create more than 7 times post-consumer wastes and 3 times more manufacturing or possess solid wastes than cloth diapers,
More energy is consumed in the life-cycle of disposable diapers than the cloth ones,
Cloth diapers laundered at home uses 77% more water than disposable diapers.

As chart 3.1 indicates, most of the mass of a discarded used diaper (assuming proper flushing of solids) is urine (C. Richer, 2006). It is essential to point to out here that these values vary from brand to brand. Generally speaking, the greater the mass of super absorbent polymers a disposable diaper contains, the lower the mass of paper pulp (the overlap slice accounts for this variability). Acquisition distribution layers (ADL) are used to help urine more efficiently distribute throughout the polymers. The paper pulp, including a part of the overlap, tends to made from virgin fluffed wood pulp and would be recyclable a good number of times were it not part of a diaper (J. Hanson, 1991).

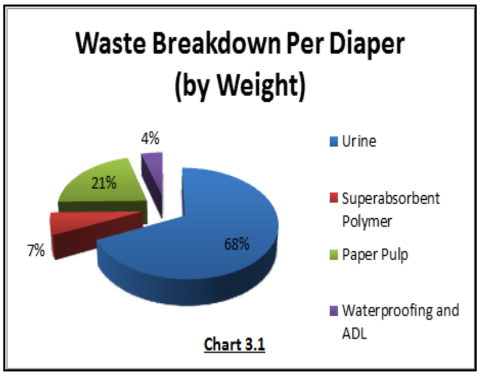

Figure 3: Waste Breakdown per Diaper (By Weight)

4. RESEARCH METHODOLOGY

The review of the literature provided an initial overview about the necessary data and factors that need to be focused on and use the data to see which methods were fit for consideration in the study with regards to the prospects of probable disposal methods. The population data for this study was collected from the Bangladesh Bureau of Statistics website and processed using the GIS application available on the site. Other sources both secondary and primary include the data collected through a brief and preliminary questionnaire based survey carried out by the authors on participants in Dhaka city, which included parents with children between 0-4 years old. The questionnaire was based on questions and parameters identified by the authors based on interactive sessions conducted with parents at different day care centers in the city.

5. ANALYSIS OF DATA

Data regarding Diaper demand: Collected from secondary sources and industry consultation firm resources (Table 1)

Country	Maximum Market Potential (Billions/year)	Current Market Today (Billions/year)	Market Penetration
Sri Lanka	1.439	0.259	18%

Bangladesh	15.058	0.301	2%
Pakistan	19.043	0.571	3%

Population Data: The data Collected from the Bureau of Statistics on Dhaka and Bangladesh was detailed as follows:

Figure 4: The Detailed data collection from the Bureau of Statistics on Dhaka and Bangladesh

The data obtained from the census data as mapped on through the application of GIS software:

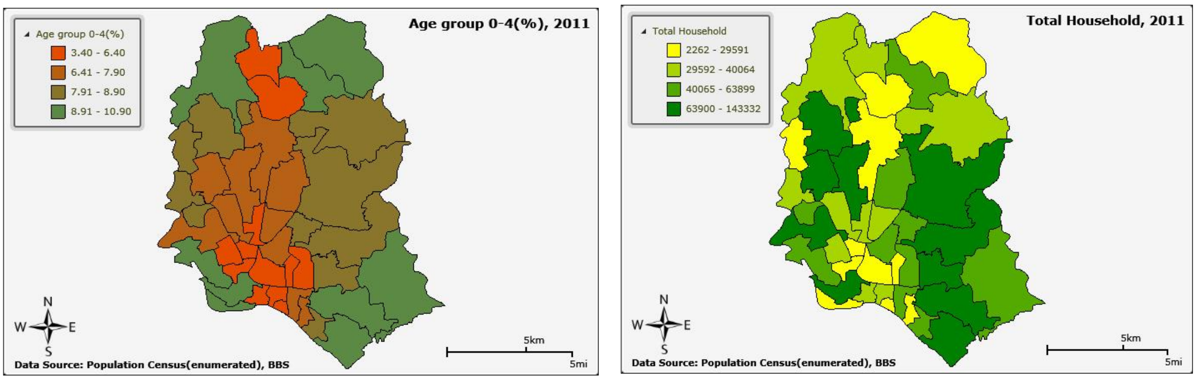

Figure 5 Mapped Census Data through GIS software

Key data from survey:

Some of the key data obtained through participants of the questionnaire survey carried out:

Table 2

Question	Answers
Use disposable diaper for child	Yes - 81%, No - 6%, Combination of Reusable and Disposable - 10%
Choice between disposable and cloth	Disposable diapers -87%, Cloth Reusable diapers - 13%

Diaper used is biodegradable	Yes 3%, Yes, to a certain extent -52 , No - 23%, What is biodegradable - 23%
More Negative Impact on Environment	Disposable diapers - 81%, Traditional Cloth Diapers - 19%
Adequate diaper changing facilities in Dhaka shopping centers	Yes- 10%, No - 90%
Use diaper changing station as a public facility	Yes- 58%, No - 42%
Diaper used is Biodegradable	Yes -3%, To a certain extent – 52%, No- 23%, Don't Know What Biodegradable Is – 23%

6. RESULTS AND DISCUSSION:

The population data obtained from the Census data from 1991 to 2011, points to a gradual increase in the percentage population of 0-4 year olds in Dhaka which along with an increase in the number of total households and the average household size indicates an imminently greater demand for diaper usage in the future. It can also be deduced from the population density maps of Dhaka that the locations with the higher percentage of 0-4 year olds are most likely to have a greater degree of diaper pollution. Looking back at Table 1 it is evident that there remains a great market potential for Bangladesh to fulfill which is evident from the recent incentives being offered to producers by the state. It seems that local production of diapers will increase in the future with a positive growth trend as has been observed from the literature reviewed. The survey data (as per Table 2) obtained showed that most parents prefer using disposable diapers over cloth/reusable diapers which can be attributed to the fact that it removes the task of laundering the soiled cloth diapers and allows for a simple use and dispose system as with the disposable diapers. While about 80% of parents agreed that disposable diapers are harmful to the environment, 87% of the respondents said they would prefer a disposable diaper to a reusable diaper. This conflicting statistic can be said to arise from the fact that while cloth diapers are cheaper to use in terms of per unit cost and that one such diaper can be used multiple times before being rendered unusable, disposable diapers allow the parents an ease of use. It appears from the answers provided by the respondents that almost one fourth of them had no idea about certain environmental terms like "Biodegradable" or different modes of waste removal. Awareness it appears perhaps is at a very low level regarding the issue of Diaper Disposal. The survey also points to almost a complaint by the respondents regarding the absence of adequate childcare facilities in shopping centers around Dhaka city, but they were almost equally divided regarding whether or not they would use diaper changing stations if it were provided as a public facility.

7. RECOMMENDATIONS:

From the literature reviewed and the data collected during the course of this research the following actions and ideas were found to be commendable:

Since most of Dhaka's municipal waste makes its way to the Landfills or else is just dumped onto other sites, a more "homegrown" initiative of consumers setting up their own small scale composting systems for the biodegradable fraction of the municipal waste in their neighborhood appears to be a solution. A communal approach to composting could reduce the amount of waste being passed off to the rather inadequate waste collection facilities offered in the city. It would also allow for propagation of more environmentally friendly initiatives like rooftop gardening etc.
An increase in the availability of diaper changing stations is perhaps the best course of action to cater to the demands of parents in Dhaka. It is suggested that the baby care rooms are provided to aid the meet the necessary nursing and hygiene requirements of the 0-4years old age group.
Better solid waste management practices must be adopted by the city corporation to make the citizens more aware about the waste dilemma at hand. The current situation of disposing just

40 to 50% of the total waste at the landfill sites while leaving the rest on the roadsides does not bode well for the future with a larger population

Waste Disposal awareness campaigns should be arranged in tandem with post natal and Child Care programs so that maximum number of households can be reached.

8. CONCLUSION:

This study implied that considering the solid waste management practices of Dhaka city, biological treatment processes with composting could help reduce the municipal waste from the generation stream. Diapers it appears is a considerable part of the municipal waste fraction in developed countries and Bangladesh's great potential for growth in the diaper market indicates a need in the future for better waste management practices to be available. Community awareness amongst the urban population appears to be at inadequate levels along with the conflicting nature of consumer decisions makes the need for more study regarding this issue. A great number of investigations are still under way or need to be carried out before confirming the final technical feasibility at industrial-scale of materials recycling and energy recovery from used diapers. Indeed, the characteristics of the plastics as raw material and their possible recycling patterns need to be studied, as do the characteristics of the effluent from the diaper treatment process and consequent water processing needs. The survey conducted was very small scale with a limited number of participants, so the data obtained could be said to be more of a representation of public opinion and rather inaccurate. The survey needs to be conducted on a larger scale for the data to be more accurate and reliable.

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STUDY ON COMPACTED CLAY LINER USED IN A PILOT SCALE SANITARY LANDFILL

Md. Assaduzzaman[1*], Md. Rajedul Islam[1] & Muhammed Alamgir[2]

[1] *Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka-1000, Bangladesh.*
[2] *Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh.*
* *Corresponding Author, <asadbuet14@gmail.com>*

ABSTRACT

This study illustrates the desirable characteristics of clay soil to minimize hydraulic conductivity and suitability of clay soil that frequently can be used as compacted clay liner materials are capable of achieving appropriate hydraulic conductivity values. Use of Compacted Clay Liner (CCL) is the most important element of a liner of a sanitary landfill. Physical properties test was conducted on clay samples are generally used to investigate soil minerals proposed as CCL that collected from an active ultimate disposal sites of MSW. This liner system consists of a compacted clay liner in 400 mm and a sand layer in 200 mm were set up in PVC pipe which diameter was 8 in inch and height of 3.5 in feet. The compacted clay liner was made at the bottom of the pipe and then sand layer was placed. The municipal solid waste was placed above the sand layer about 300 mm and about 8 liters water was poured on the top of municipal solid waste in order to justify the infiltration rate of leachate in mili-liter per day through the CCL layer. There was no infiltrated leachate from first 8 days. From 9 days the infiltrated leachate was collected by using volumetric cylinder and the infiltration rate in mili-liter per day of the leachate movement through CCL was measured by dividing the infiltrated leachate in mili-liter by elapsed time in days. From this study the conclusion can be drawn the clay soil possesses desirable characteristics to minimize hydraulic conductivity and frequently can be used as compacted clay liner materials are capable of achieving hydraulic conductivity values much lower than the Environmental Protection Agency. So it can be used as the hydraulic barrier for waste containment facilities. The study reveals that the characterization of clay is suitable as a CCL material for the construction of sanitary landfill in Bangladesh.

Keywords: Landfill, Hydraulic Conductivity, Standard Proctor Test, Clay Liner, Infiltration Rate and Municipal Solid Waste (MSW)

INTRODUCTION

Use of Compacted Clay Liner (CCL) is the most important element of a liner of a sanitary landfill. The main requirements of liners are the minimization of pollutant migration, high adsorption capacity and retardation of pollutants, resistance to chemicals and low swelling shrinkage potential. Low permeability compacted soil liners, also referred to as CCLs are the historic engineered component used in landfills. Clay rich soil is placed in layers and compacted with heavy equipment to form a barrier of movement of liquids and gases. The soil liner is typically designed to have a hydraulic conductivity = $1x\ 10^{-7}$cm/s. The origin of this design criterion is unclear; $1x10^{-7}$ cm/s was evidently selected on the assumption that this was an achievable value that would result in negligibly small seepage through the liner (Daniel & Keorner, 2007). The low hydraulic conductivity of clay minerals makes them potential materials to use as CCLs in sanitary landfill for environmental protection. Soils classified as inorganic clay with high plasticity is considered are the suitable material for landfill liner (Oweis & Khera 1998). If natural available clay or clayey soil is not suitable for liner, kaolinite or commercially available high swelling clay (Bentonite) can be mixed with local soils or sand. In Bangladesh this materials are not locally available and would have to be imported from elsewhere and could significantly increase the cost of construction (Alamgir et. al. 2005a). CCL are constructed primarily from natural soil materials that are rich in clay, although the liner may contain processed materials such as bentonite. Compacted clay liners are constructed in layers call lifts that typically have a thickness after compaction of 400mm. On side slopes, the lifts in final cover systems are almost always placed parallel to the slope. However, lift parallel are very difficult or impossible to construct on side slopes steeper than 2.5(H)-to-1(V) (Daniel et al.,1997).

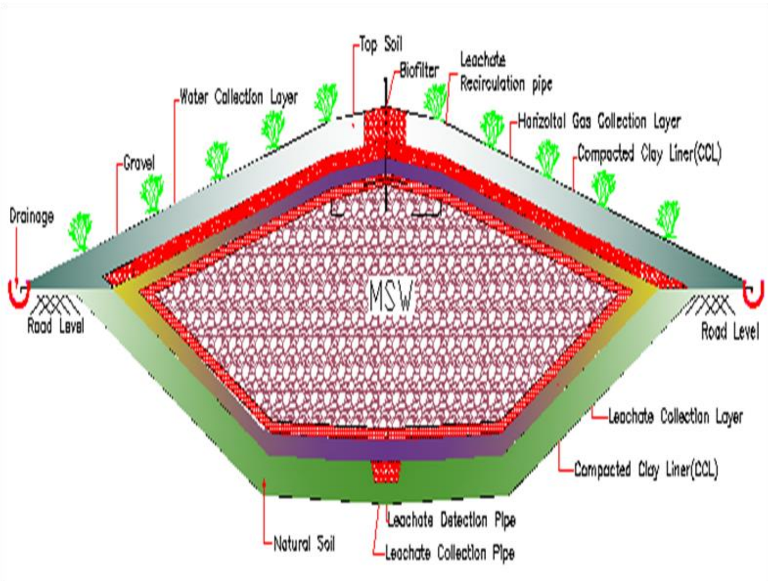

Fig. 1: A cross-section of pilot scale sanitary landfill

The design of CCL will depend on site-specific factors including the properties and engineering characteristics of the soil being compacted, the degree of compaction attainable, the total expected load, and the expected precipitation. The design criteria of CCL are shown in Fig.2.

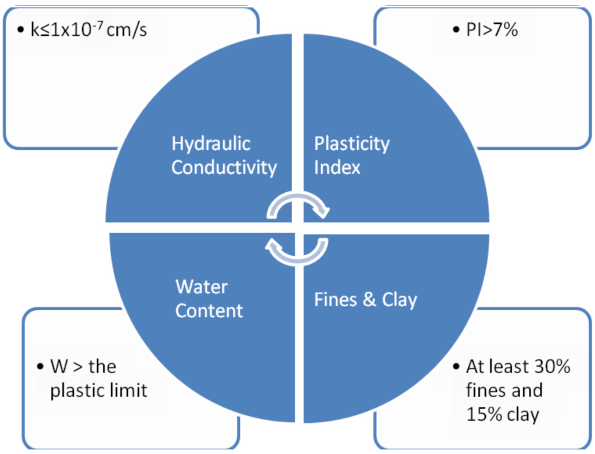

Fig. 2: Criteria to satisfy the clay as a CCL

Sanitary landfills also called modern, engineered or secure landfills; these usually have physical barriers such as liners and leachate collection systems and procedures to protect the public from the exposure to the disposed wastes. The term sanitary landfill may be defined as the operation in which wastes the operation in which wastes to be disposed of are compacted in layers and covered with a layer of earth at the end of the each day's operation. A typical landfill consists of subsystems such as the landfill liner, leachate collection and management system, landfill gas management system, landfill gas monitoring & leachate monitoring systems, road network, drainage system and final landfill cap. Their function is to secure the normal landfill operations and to control the anticipated emissions generated mainly by the decomposition of organic matter such as leachate and landfill gas. These criteria are quite complex to investigate because the performance of the liner materials is influenced by many variables such as clay contents, liquid limit, plasticity index, activity, amount of fines, properties of the fines, grading, compaction level, water contents and mineralogy.

Typical tests that are generally used to investigate soil minerals proposed as CCL such as Atterberg limits, hydraulic conductivity at different water content, standard proctor test were conducted on samples of Khulna clay collected from an active ultimate disposal sites of MSW, namely, Rajbandh at Khulna city. The test results reveal that the subsoil properties, which should be considered as a potentially suitable material for the compacted clay liner of sanitary landfill. The test set up is very important for the performance study of the base liner system of pilot scale sanitary landfill. Physical model of sanitary landfill liner was developed to demonstrate the students who does not see such kind of model before. The objectives of this study can be outlined as follows: 1) to determine the characteristics of the clay used as CCL. 2) To simulate the behavior of base liner used in sanitary landfill and to identify the basic requirements of clay used as CCL.

Materials and Methods

This chapter discusses about the physical and compaction properties of CCL and the performance of base liner system through test set up.

Characterization of Compacted Clay Liner

Clay used as CCL

The soil used as base liner in the landfill was collected from Rajbandh, Dumuria, Khulna in the north side of Khulna-Satkhira Highway and 8 km far from the city centre. The soil samples were transported to the geotechnical laboratory of KUET to conduct the necessary tests. The physical mechanical properties of CCL as shown in Table 1.

Table 1: Physical Properties of used sample in this study

Properties	**Value**
Specific gravity, Gs	2.71
D_{50} (mm)	0.007
D_{10} (mm)	0.0012
Coefficient of uniformity , C_u	7.08
Coefficient of gradation, C_c	1.57
Liquid limit (%)	52
Plastic limit (%)	30
Plasticity index (%)	22
Shrinkage limit (%)	16.
Maximum dry density(kN/m^3)	15.41
Optimum moisture content (%)	23
Silt (%)	83.6
Clay (%)	16.4
Hydraulic conductivity (cm/s)	0.57×10^{-7}

Test set up of the base liner system

The test set up is very important for the performance study of the base liner system of pilot scale sanitary landfill. This liner system consists of a compacted clay liner in 400 mm and a sand layer in 200 mm.

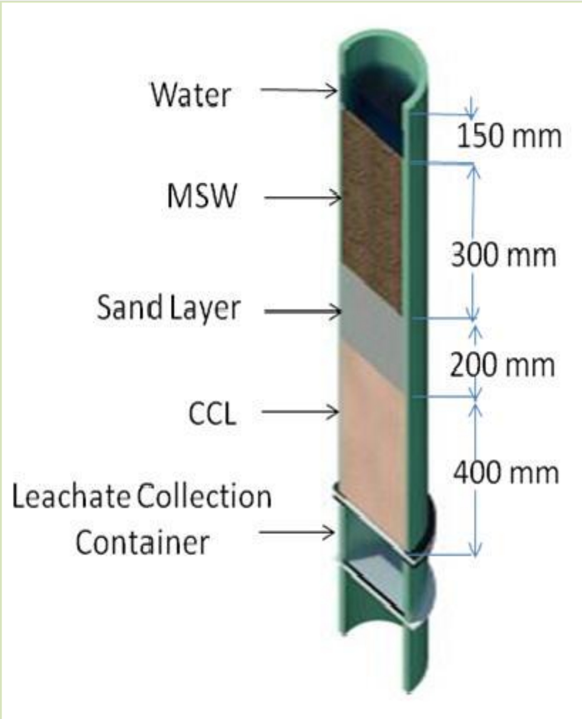

Fig. 3: Longitudinal cross section of test set up of base liner system

Fig. 4: Test set up for the performance study of base liner system

The compacted clay liner and sand layer were set up in PVC pipe which diameter was 8 in inch and height of 3.5 in feet. The leachate collection container was set up with the pipe. The compacted clay liner was made at the bottom of the pipe and then sand layer was placed. The municipal solid waste was placed above the sand layer about 300mm. And about 8 liters water was poured on the top of municipal solid waste in order to justify the infiltration rate of leachate in mili-liter per day through the CCL layer (i.e. the performance of the base liner system). The test set up was kept under observation about 15 days. There was no infiltrated leachate from first eight days. Because, it is required several days for filling up with water the pore spaces of clay. After saturation of clay the leachate infiltration through it was started. From 9 days the infiltrated leachate was collected by using volumetric cylinder and the infiltration rate in mili-liter per day of the leachate movement through CCL was measured by dividing the infiltrated leachate in mili-liter by elapsed time in days.

RESULT & DISCUSSIONS

The performance of landfill liner depends on the basic characteristics of the soils. The evaluated index properties of the clay are listed as liquid limit= 52%, plastic limit= 30%, plasticity index= 22%. The plasticity of a soil refers to its capability to behave as a plastic material. Plastic clay is a typical suitable material for sanitary landfill liner. Clays with high liquid limit generally have low hydraulic conductivity. The soil with plasticity index as low as approximately 10% can be compacted to achieve a hydraulic conductivity= 1x 10^{-7} cm/s (Daniel & Koerner, 2007) recommended that the liquid limit of the liner material be at least 20%. And the plasticity index must be more than 7% (Daniel, 1993).The behaviour of studied soils compaction was established in the laboratory by preparing several batches of soil at different molding water contents and then compacting the materials from each of the batches into the molds of known volume. Hence, Standard Proctor Test was used. The maximum dry density or dry unit weight occurs at optimum water content. The main reason for developing a compaction curve is to determine the optimum water content and maximum dry density. The optimum water content and maximum dry density were found 23% and 15.41 kN/m^3 from compaction curve. For natural soils, the water content of the clay liner material at the time of compaction is perhaps the singly most important variable that controls the engineering properties of the compacted material. Soil compacted at water contents less than optimum tend to have relatively high hydraulic conductivity;

soil compacted at water contents greater than optimum tend to have a low hydraulic conductivity and low strength (Daniel & Koerner, 2007).

Hydraulic conductivity is a main indicator of CCL for the construction of landfill. Liner soil should have at least 30% fines and 15% clay to achieve hydraulic conductivity in the range of 1 x 10^{-7} cm/s (Daniel 1993b; Benson et. al. 1994). Liner soil was found 83.6% fines and 16.4% clay. Hence the clay can be used for natural barrier to achieve a hydraulic conductivity in the range of 1 x10-^{7} cm/s, as it possesses suitable amount of clay and fine fractions. The hydraulic conductivity at optimum water content was found 0.58 x 10^{-7} cm/s. The result of hydraulic conductivity is satisfied the design criteria of CCL. The performance of the base liner system (i.e. the CCL) was found from the permeability test and leachate infiltration rate test comparatively better. Because the hydraulic conductivity of the compacted clay liner was below the allowable limit 1 x10^{-7} cm/s. And the leachate infiltration rate through the CCL layer was also found very low. So it (CCL) can be used as the hydraulic barrier for the waste containment facilities.

Table 2: Performance study of base liner system

Days	Infiltrated leachate (ml)	Infiltration rate (ml/day)
1	0	0
5	0	0
7	0	0
8	0	0
9	10.5	1.2
11	31	2.8
12	57.7	4.8
14	84	6.0
15	98.2	6.6

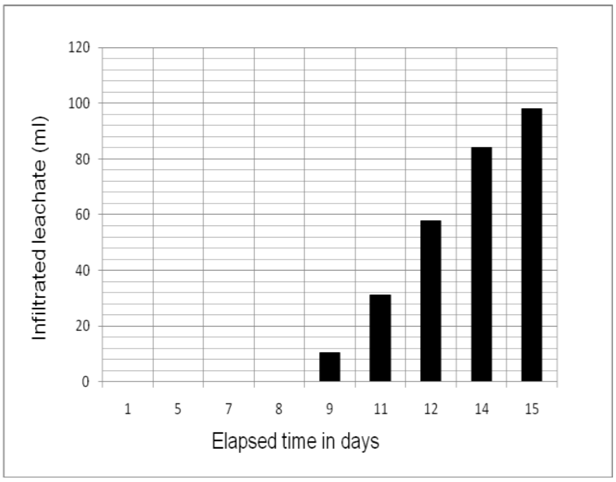

Fig. 5: Bar diagram of elapse time vs. infiltrated leachate

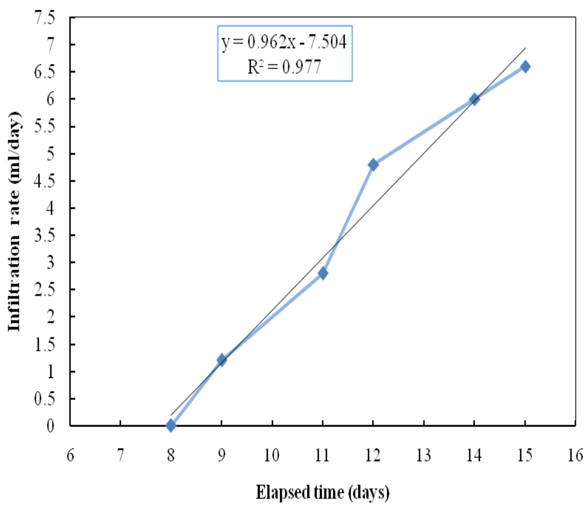

Fig. 6: Relationship of elapse time vs. infiltrated rate

CONCLUSION

Clay liner materials of a Sanitary Landfill are selected so that a low hydraulic conductivity will be produced after the soil is remolded and compacted.

From this study the conclusion can be drawn the clay is highly plastic. Generally this type of soil possesses desirable characteristics to minimize hydraulic conductivity and frequently can be used as compacted clay liner materials. The index properties of the soil satisfy the basic requirements as a CCL material.

Compacted clay liner systems are capable of achieving hydraulic conductivity values much lower than the EPA's (Environmental Protection Agency) minimum value of 1 x10^{-7} cm/s. Values in the range of 10^{-8} and even 10^{-9} can be achieved with careful construction and quality assurance. With such low levels of hydraulic conductivity, the compacted clay liner is a viable choice as a municipal landfill liner system. To achieve hydraulic conductivity values in this range, several aspects of the clay liner must be considered. The ideal placement conditions for the compacted clay are achieved with a high dry density and a moisture content wet of the line of optimums. At a molding water content of 23% or more, the hydraulic conductivity of soil satisfies the value of 1x 10^{-7} cm/s.

From the performance study of the base liner system was found that the leachate infiltration rate is very low. So it can be used as the hydraulic barrier for waste containment facilities.

Thus, the study reveals that the characterization of clay is suitable as a CCL material for the construction of sanitary landfill in Bangladesh.

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STUDY ON HAZARDOUS WASTE IN BARISAL CITY

Md. Hamidul Islam[1*], Shampa Reza Katha[2], Md. Sabbir Ahmed[2], Protiva Adhikary[2], Susmita Howlader Muna[2] & Ayesha Siddiqua[2]

[1] *Department of Building Engineering & Construction Management, KUET, Khulna-9203, Bangladesh.*
[2] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <hamidcekuet@gmail.com>*

ABSTRACT

Hazardous waste is the waste that poses substantial or potential threats to public health or the environment. A hazardous waste is special types of waste because it cannot be disposed of by common means like other products of our every life. Hazardous wastes exhibit four characteristics: ignitability, corrosivity, reactivity, toxicity. Barisal is one of the divisional cities among seven divisions of Bangladesh. It is situated on the bank of Kirtonkholariver. It is 142 km (373 km by road) from capital city Dhaka. It's Latitude: 22°38′ and 22°45′ north, longitude: 90.36′. The city consists of 30 wards and 50 mahallas. The area of the town is 20 km². Barisal municipality was established in 1957 and was turned into a city corporation in 2000. Its population is about 224389; male 123402, female 100987. The aim of the study is to calculate the amount of various types of hazardous waste, calculate the total amount and find out the recycling pattern of hazardous waste. Studying hazardous waste helps to know about the wastes in addition from where it originates and disposed. It also helps to make or arrange a proper waste management process. In Barisal city approximately 965-1000 syringes, 965 ml chemicals and 350-400 vacuum needle are used in all the medicals and labs per day. This study faced some hinders for collecting information and faced some unnecessary questions at the time of questionnaire. It should be recommended that the city corporation, NGOs and GOs help to promote waste management process. To enhance the waste management process public responsibility is a key role in this manner to keep environment safe, hygiene and sound.

Keywords: Hazardous waste, Barisal city, Recycling pattern, and Types of hazardous waste.

INTRODUCTION

In the present age of globalization, its well knows about the waste which can be hazardous as well as solid waste. Hazardous waste creates dangerous or potentially harmful effects to human health or the environment. Hazardous wastes are in the stage of liquid, solid, commercial generator or sludge. It can be by products of manufacturing process or simply discarded commercial products like cleaning, fluid or pesticide (Sejuti and Islam, 2011). Worldwide the United Nations Environment Program (UNEP) estimated that more than 400 million tons of hazardous waste is produced universally every year. The Environmental Protection Agency (EPA) has a list of more than 5000 specific hazardous waste. The news of hazardous waste (July 15, 2014) was "Pressure cell for reproducing deep-earth chemistry". Barisal is one of the divisional cities among seven divisions of Bangladesh. About 5 lakh people live here. In this area many hazardous wastes are produced every day. The hazardous waste or the clinical waste in Barisal city is going on without any scientific management and training. More than 800 kg waste produced per day in two public hospital and 50 private hospital, clinic, diagnostic centre (New Nation, Bangladesh published on 20.08.2008) and also more than 8,000 kilograms of clinical waste are produced every day in 25 clinics 45 diagnostic centre (New Age, Bangladesh). Hospital and clinic use syringes, vacuum needles, chemicals which creates an amount of wastes after use. Under the city corporation authority, the wastes are collected for dispose in the dumping site in a manual method. No other developed methods are used for the disposal of wastes. If they know about all the methods such as incineration, it could be helpful for them to keep the environment health and hygiene.

MATERIALS AND METHOD

Barisal city has been selected as the study area to calculate the amount of various types of hazardous waste, calculate the total amount and find out the recycling pattern of hazardous waste. Barisal city is an old part of Kiortankhola river on northern shore of the Bay of Bengal in southern Bangladesh. It is 142 km from the capital city of Bangladesh. The area of the town is 24.91km^2. It is now the headquarters of both the Barisal division and Barisal district. The city consists of 30 wards and 50 mahallas. Barisal municipality was established in 1957 and was turned into a City Corporation in 2000 (Banglapedia, 2014). The data collected for these studies are mainly two type's e.g. primary data and secondary data. Primary data collected from field survey through questionnaire and discussion. The hazardous waste related institutions such as hospital, clinic, laboratory, school, college, university, tanneries, and electrical materials production industry. Secondary data collected from different sources according to need. Secondary information's such as statistical data, report, maps have been collected from various GOs and NGOs organization, Banglapedia and survey reports. Different resource paper, recent related publication, journals, articles and books were collected from website. Some steps are followed for this study:

Interview

Interviews were conducted with officials and peoples responsible for waste production with management. Data are as follows such as the total amount of waste generation, component of generating waste, amount of waste disposed, awareness, current waste management pattern, availability of recycling facilities and disposal method.

Waste Audit

A waste audit is a formal structured process used to quantify the type and amount of waste being generated by an organization. Information from audit helps to identify current waste practices and how they can be improved. In this study, waste auditing was done to analyse the composition of hazardous waste in Barisal city. The waste audits were done over a period time to get an average value. By the audit know that generating waste stored or dump temporary in the selected place of the institution or industry in a week by separation; then the weight of the waste component was measured and after one week these are sent to the disposal sites.

Data Analysis

After getting the information we analysed the data. Basic descriptive statistical analysis tools were used to compute and present the collected data. Voluntary participation of the respondents as well as confidentiality of their information was strictly maintained.

RESULTS AND DISCUSSIONS

Hazardous waste is a waste that is dangerous or potentially harmful to health or the environment. Hazardous waste may be liquid, solids, gases or sludge's. They can be discarded medical or clinical products like syringe, vacuum needle, gouge, chemical and liquid or the by-products of manufacturing process.

Hazardous waste management is the collection, treatment and disposal of waste material. When this management is handling improperly then it can cause harm to health and environment. It can be hazardous during inadequate storage, transportation, and treatment or disposal operation.

In Barisal city many products are used which are hazardous mainly syringes, chemical, vacuum, gouge, sludge. The main effect of these hazardous wastes is on our human health and environment. In every clinic, hospital, diagnostic centre the approximate amount of chemical 5740 ml, syringes 5600 piece, vacuum 4550 piece, gouge 55 kg, and sludge 50 kg per week in the study area. According to Barisal city corporation the hazardous waste produced approximately in clinics 1-1.5ton, medicals 3 ton per day. All these wastes are disposed in the dumping site. The government doesn't maintain any law to reuse the wastes but some NGOs reuse these wastes in unlawful ways.

Fig. 1: Present scenario of medical waste in Barisal city.

In Barisal city different types of waste are produced. Hazardous waste is one of them. Hazardous wastes are produced in clinic, hospital, diagnostic centre, tannery and others. The wastes collected from the labs are kept in a specific place from where the odour doesn't spread in the hospital or clinic.

Fig. 2: Current scenario of tannery waste in the study area.

Tannery is one of the elements of hazardous waste. Bangladesh Agricultural Research Council (BARC) in 2012 and 2013 said about 48% poultry feed contains tannery waste. In Bangladesh tannery waste management process is insufficient because of people's unawareness. Waste collectors collect this waste after the end of week to dispose the waste in the dumping site.

If people and government can be concern about the hazardous waste management then it can be not only helpful but also less harmful for people and the environment.

CONCLUSION

Wastes are substances or objects, which are disposed of or are intended to be disposed of or are required to be disposed of by the provisions of national law. Waste is dangerous to human health and environment. In Barisal city there are different types of waste component found such as syringe, vacuum needle, gauge, sludge, hazardous pathology laboratory chemical. The hazardous waste collection at clinic, hospital, diagnostic centre are chemical 5740 ml, syringes 5600 piece, vacuum 4550 piece, gouge 55 kg, sludge 50 kg per week in Barisal city. Barisal city corporation authority informed that only one NGO Prodipon works for the management of hazardous waste. The city corporation has no burning unit, Prodipon has a burning unit that burns only clinical waste but it does not work properly. Due to lacks of good intention of the authority, technical knowledge, financial availability, public awareness and other inherent limitations the overall solid waste management scenario is not good. To enhance the overall waste management the people and city corporation expend vital role in this scenario.

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UTILIZING ELECTRONIC WASTE: AN EFFICIENT WAY TO TECHNOLOGICALLY DEVELOPED BANGLADESH

Md. Abdus Samad[1*], Md. Jobayer Hossain[1], A.S.M. Riyad[2] & Md. Sarwar Jahan[1]

[1] *Department of Electronics and Communication Engineering, Khulna University of Engineering and Technology, Khulna-9203, Bangladesh.*
[2] *Department of Civil Engineering, Khulna University of Engineering and Technology, Khulna-9203, Bangladesh.*
**Corresponding Author, <sami.samad79@gmail.com>*

ABSTRACT

Electronic wastes or in short, E-wastes are electrical and electronic equipments and devices that have been exhausted and generally find no use. Due to increasing concern about threats of these wastes to environment and human health, managing these wastes is getting much more important these days. This paper investigates the present scenario of E-waste production in Bangladesh and its possible reuse schemes. The result of these explorations is quite promising. The useless devices for example discarded computers, PCB board, UPS, televisions, VCRs, stereos, electric lamps, cell phones, audio equipment, batteries, printers etc. may be reused by connecting active parts from two or three of such e-waste products. The rest non-recycled parts may then be disposed following environmentally safe processes. Thus proper management of e-wastes may give a way to economic growth and environmental safety. This paper also proposes an outsourcing model of this sector as profitable business using huge workers of the country. It will also play important role in eradicating unemployment in a sound way. Besides, this will also help reaching electronic products to the low-earning people and hence building technologically developed modern Bangladesh.

Keywords: E-waste, Disassembly, Ghettoization, Humiliating, Pulverization, Reuse, Recycling

INTRODUCTION

The E-waste products in Bangladesh are increasing day by day with the increase of users. The country has enough resources but not properly utilized most of the cases. E-waste processing is a promising sector on which it can helps to build technological Bangladesh. E-waste is an informal name for electronic products nearing the end of their useful life. . E-waste such as discarded computers, televisions, VCRs, stereos, copiers, fax machines, electric lamps, cell phones, audio equipment and batteries[4] if improperly disposed can leach lead and other substances into soil and groundwater as they contain Antimony, Arsenic, Barium, Beryllium, Cadmium, Chlorofluorocarbon (CFCs), Cobalt, Copper, Gallium, Lead, Mercury, Nickel etc. (Chatterjee, 2012; Hossain et al., 2010; ESDO, 2005; Gupta, 2008; Nzioka et al.).When these elements mixed with air, water soil by the unhygienic disposal of e-waste products pollutes air, water, soil and these has a great impact on human health, environment and ecological system (Sabha, 2011). Brain disorders, kidney, renal and neurological damage, mental retardation, behavioral problems, hearing impairment, lung damage, fragility of bones, high blood pressure, nerve and brain damage, kidney and liver disease and even death may be happened (Gupta, 2008; Nzioka et al.). Where properly organized recycling technology is not available and the unorganized operators are extracting precious metals through crude means for easy money. Many of these e-waste products exported in various countries and after re-servicing, packaging these e-waste products again imported in Bangladesh. So these product's prices becomes high than the probable price if these e-waste products will re-servicing in Bangladesh. However the e-waste recycling can be made a profitable business if it is managed in a professional way. Even for a small improvement good price is found (Enayetullah et al.). E-waste contains various valuable materials including metal, plastics and glass etc. which are of 95% of the total e-waste and 3-5% of the total e-waste contain valuable metals like gold, silver, copper, and other precious metals like palladium, tantalum etc. (Chatterjee, 2012; Nokia). In the developed countries, all the well-established processes are available for processing these to extract the precious metals with highest yields (Kahhat et al.). CRT monitor is recycled with CRT C-002 (CRT recycling). Mobile phone recycling (Recycling Mobile Phones; The recycling process in action), Refrigerator recycling (The refrigerator recycling process), Battery recycling (Battery) as well as other e-waste recycling (E-Scrap Recycling, Engineering solution for waste from electrical and electronic equipment) processes are done in developed countries in a proper way using various machines. These processes are automated and minimal involvement of manpower is required. In contrast, the e-waste processing technologies in developing countries are not yet matured and the recycling is still being carried out in primitive ways. It is estimated that 2.7 million metric tons e-wastes are generated in Bangladesh (Chowdhury et al.). As a result, a substantial amount of valuable materials are being lost due for this unskilled operation. So it may be a very profitable, skillful, environmental, employment processing way for Bangladesh to advance its technology. Bangladesh may also be economically developed so these e-wastes need carefully handle to maintain the ecological system in balance and protect the environment. This paper highlights the development of an e-waste technological field in Bangladesh and environment, healthful way of proper management of these e-waste products.

PRESENT SCENARIO OF E-WASTE PRODUTION IN BANGLADESH

In Bangladesh many companies produce e-waste products at a high rate. It is producing e-waste much in town areas than rural areas. Here generates waste from discarded computers, televisions, VCRs, stereos, copiers, fax machines, electric lamps, cell phones, audio equipment, amplifier, audio and sound system, cassette player, CD player, Cathode Ray Oscilloscope, detector, equalizer, mixer, modem, telephone and batteries. In recent days as the usage of cell phones, computers, televisions are increases. Its waste also increases. According to BEMMA, 3.2 million tons of electronic products are used every year in Bangladesh and every year generates roughly 2.8 million metric tons of e-waste (Hossain et al., 2010, Chowdhury et al.). In recent years 2009-2010 various companies named Transcom, Energypac, Sony Ericson, Anik, Osaka etc. produces 94,55,964 products and their yearly generated e-waste(2009-2010, up to June) 93,75,850.973 products (Hossain et al., 2010).

Table 1: Yearly Estimation of E-waste Generation

Companies in Bangladesh	**Production(Yearly) (2009 -Jun 2010)**	**Generated e-waste(Yearly) (2009-2010, up to June)**
Transcom	5253,313	5200,254.538 (98.99%)
Energypac	2626,657	26 16,150.372 (99.6%)
Other companies (Osaka, Anik, sony Rangs, Ericson etc.)	1575,994	15 59,446.063 (98.95%)
Total	94,55,964	93,75,850.973

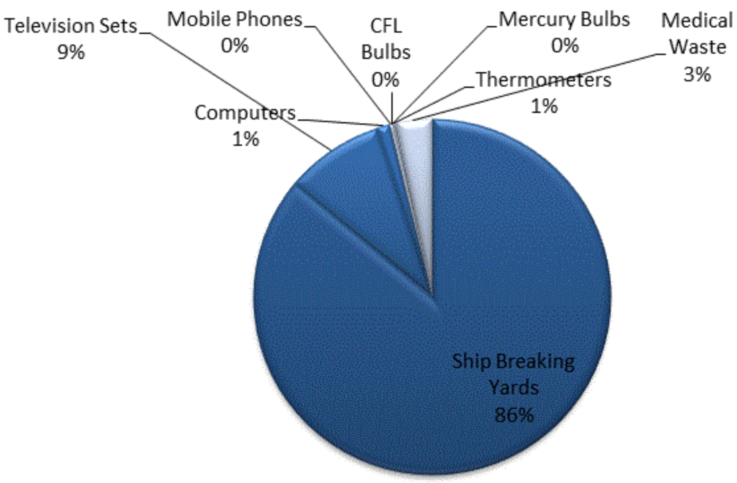

Fig.1: Graphical representation of different e-wastes in Bangladesh (per year)

It will cause a great risk for Bangladesh as there is no proper way of management system of these e-wastes. There have laws but have no application. As a result some private business factory set up for processing these. These factories have no effective idea of collecting and processing these. Consequently a huge amount of e-wastes remains out of processing. As a result their utilities are damaged and thus pollute environment & affect human health as well as affect ecosystem. The increasing rate of e-waste in Bangladesh is shown at Fig.1.

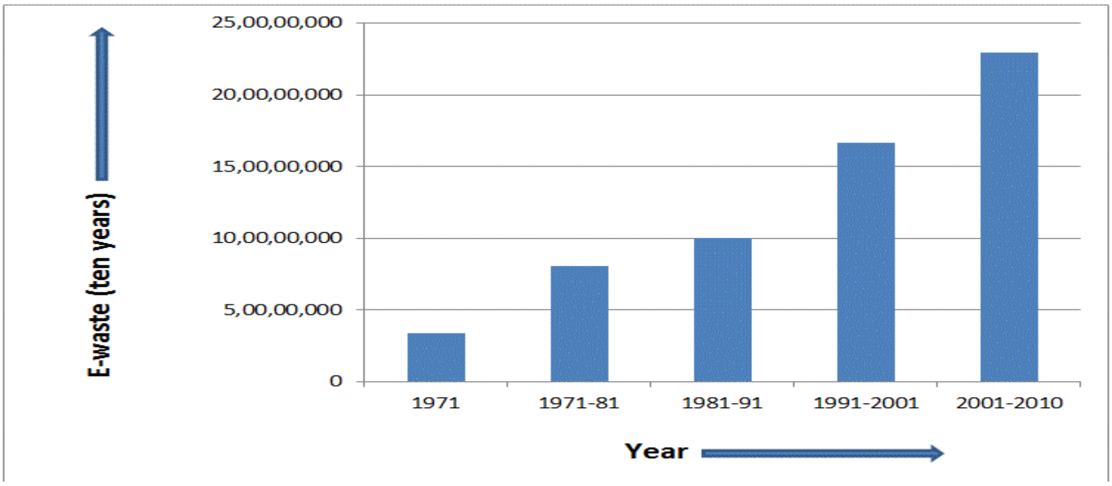

Fig.2: Increasing rate of E-waste in Bangladesh (*Source: ESDO 2010*)

PROPOSED E-WASTE MANAGEMENT POLICIES

The conventional systems running in Bangladesh are not developed. They damage a huge amount of valuable element while processing and pollutes environment while discarding these rest non-recyclable e-waste. E-wastes are produces at home, institution, medical centre, shop, industry, factory, ship building agency etc. Almost all the people in Bangladesh are needed to aware about its adverse effect. So they throw out these products roughly here and there. Such practice causes a threat on environment. Some policies can be taken to develop the management system of these e-wastes for their effective usage.

A. Collection

In Bangladesh collecting process is so primitive, unskilful, unwise and unhealthy. Here maximum E-waste products are collected with other waste and it is mostly done by the illiterate, unskilled, poor people, and many of those are teenagers and women. They collect it randomly with others waste and diminish its reusable utility. So the following systemic policies need to take to develop the current system

a. Creating a sound network

b. Old for new policy

c. Selecting category

d. Making laws

e. Rising awareness by communication and education.

f. Proper training.

B. Reusing

Reusing means using the devices or products after fixing it, without recycling or changing its shape and property. This process needs no high technology or instrument. By attaching one active parts of a device replacing with inactive part of another one, can reuse the device. It is very easy and effective operation to use E-waste. Even a person can fix it at home. It recovers a large amount (about 95% by weight) of E-waste. Rest others 5% needs to recycle (Chatterjee, 2012). In formal and informal sector reusing are being done in Bangladesh. The person who knows about the device can reuse the device and can contribute in reusing individually and it is an effective way.

Fig.3: Reusing process at home appliance E-waste

C. Recycling

120000 urban poor people directly or indirectly are involved with recycling process with the informal sector in Dhaka city. There process about 15% waste (approximately 475 tons /day) of the total waste are being recycled daily (Gupta, 2008). About 95-97% of the e-waste according to weight contains various metal, glass and plastics, those can be easily pretended and ghettoized manually without affecting environment. And the rest about 3-5% according to weight of e-waste contains PCBs, wires (Chatterjee, 2012). The conventional recycling system running in Bangladesh losses many e-waste products. So for highest production the system should be developed in the following ways:

a. Metals, plastic and glass can be managed through the conventional recycling process

b. The ghettoized PCB and wires will be milled by expert people to make uniform powder. Then the powder has to process with high technology and skillfull way.

E-WASTE RECYCLING EXAMPLES

E-waste management systems are running widely in various countries in the world. PCB recycling, Mobile phone recycling, CRT recycling, Battery recycling, Refrigerator recycling are mostly referable.

Recycling of Printed Circuit Board (PCB)

PCB contains about 3 to 5% according to weight of total e-waste, various precious metals named gold, silver, copper, palladium, platinum, and other metals (Chatterjee, 2012). Power transformer, oils and waste oils in the instruments or machine can be likely as 51,655.50 Kg and 4,453 Kg respectively (ESDO, 2005). For recovering of these metals needs high skill, complex equipment and various machines. The un-scientific methods running in Bangladesh for recovering of these valuable metals like gold, silver, copper are poor. So the process can be developed in the following ways:

For recycling the PCB some steps have to develop to get the metal. The PCBs are turned to powder of small size with various processes like physical structural impaction, crumbling granulation, etc. The process grinding breaks down the PCBs into pieces by excruciating or rubbing. Then it is processed with various processing technique according to the metal. PCBs are granulating into small pieces. Then by adopting separating technique the metals are separated from one another. Using magnetic attracting separation technique magnetic materials like iron, nickel and cobalt are separated from the powder. The metal aluminum is separated by Eaddy current separation process separates aluminum and electrostatic separation process separates the plastic. Various thermal processes like thermal burning are used for recovering the metal. Chemical process like chemical extraction separates waste liquid products. Electrolytic process is used for recovering copper. Gold is recovered by a chemical reaction using cyanide solution.

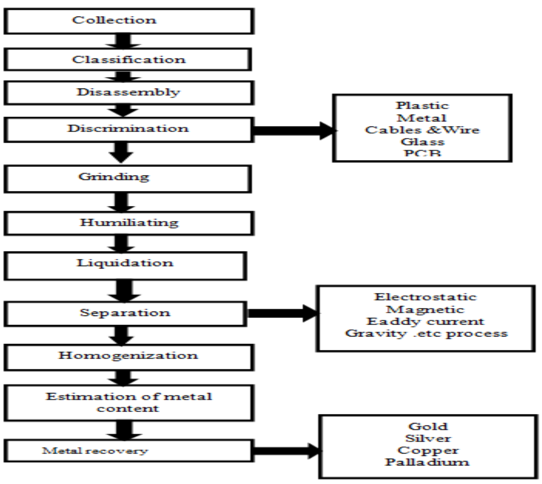

Fig.4: Recovery of Various Metals from E-waste (PCB)

Market values of the metal which can be recovered from 1000Kg of PCBs (E-Scrap Recycling)

Table 2: Market value of metals from1000 Kg PCBs

Recovered metal	Weight (gm)	Cost in US$ (approximate)
Gold	279.93 g	6115 (685.00$ per 31 g)
Precious metals (Pt, Pd, In)	93.31 Kg 3852	3852 (1284.00$ per 31 g)
Copper	190.512Kg	1470 (3.50$ per 453.59 g)

Aluminum	145.152 Kg	448.00 (1.28$ per 453.59 g)
Lead and Tin	30.844 Kg	144.16 (2.12 $per 453.59 g)
Silver	450 Kg	213.15 (14.70$ per 31 g)

MOBILE PHONE RECYCLING

Some processes can be adopted in Bangladesh to recycle mobile phone like Nokia Corporation (Nokia). From their customer care office who drop off their old phone. Old mobile phones are also collected by the users by raising awareness among them. After collecting phone, separate batteries from phone. The separated phones are then crushed into small pieces by hammer and rotor. Using grinding, pulverizing Gold, copper, platinum, aluminum, copper, palladium, plastic are separated like fig.5. Following these process Bangladesh can develop in mobile recycling.

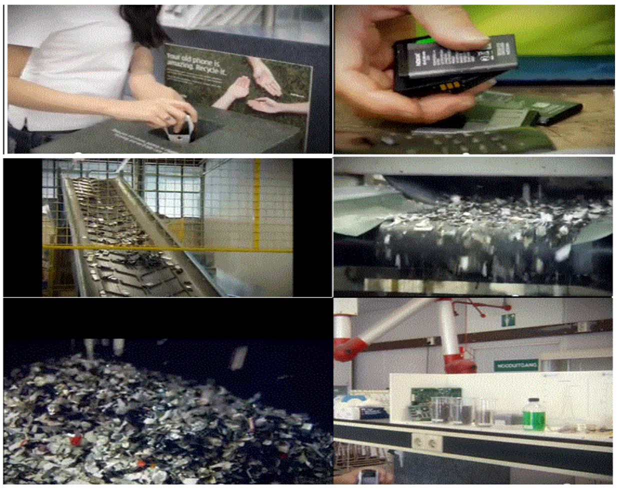

Fig.5: Mobile recycling by Nokia (*Source: Nokia*)

C. CRT recycling

CRT recycling requires some processes to develop the conventional process in Bangladesh. It can be done by a machine named CRT C-002. At first the electronic gun is cut with explosion-proof belt by CRT cutter machine CRTC-002. A heat band can be then covered around the frit at CRT. The band will heat to a high temperature and it will heat the CRT. For creating a temperature differential to break the glass at slightly above the frit causing the frit and the Funnel can be separated from the Panel for further processing, cold air is then applied. In this part of vacuum cleaner machine phosphor layer are removed using high power. There are also some filters inside it. It cuts the CPU into pieces named panel which is can be processed easily. Safety glass top makes it safety to operate. Its performance is high and can cut 20 CRTs per hour. It requires only two operators for this purpose (CRT recycling).

D. Battery Recycling

Battery recycling process is an easy and effective process. Although some private industries run their business, they can be more benefited if they adopt this process in an organized process using battery recycling plant (BRL-1000) which is run with highly mechanism process. It separates the LEAD from lead battery easily. After processing this product lead powder will be gotten without any plastic inside. Pb-Sb alloy, Ebonite also found here (Engineering solution for waste from electrical and electronic equipment). The battery contains lead metal and metal oxide 65%, polypropylene or a fibrous glass mat at PVC made cell separators is 7%, dilute sulphuric acid 16%, plastic catching 12%

(Nzioka et al.). Totally it consists of 4 modules (Engineering solution for waste from electrical and electronic equipment):

i. Breaking into parts

ii. Separate the parts into different categories

iii. Desulfurization

iv. Discharge treatment system.

The machine contains different parts like battery cutter, belt conveyor, intelligent shredder IC-1100, vibrating screen, stainless reaction stainless vessel, compartment filter press, effluent treatment tank, granulator, water float separator, Spiral screw conveyor and shaking table. The features of this battery recycling plant (BRL-1000) have given below.

a. It runs in an automatic process

b. Any types of lead battery can be recycled.

c. Recycling process can be shared.

d. It does not affect environment much.

E. Refrigerator Recycling

Refrigerator contains metal, glass, plastic, oil etc. It can be recycle 95% of its materials. It also contains some hazardous material such as CFC-12, poly-urethane that contains CFC-11 carried by foam in refrigerator.CFC-12 is an ozonized bleeding powerful chemical which contains 10950 times greater than cl2.One refrigerator contains 2.5 tons cl2 equivalent. Foam contains CFC-11 which is 4680 times greater than cl2.It contains 10 pounds foam which can produce 20Kwh electricity. It can save $150 a year in energy cost (The refrigerator recycling process). This recycling process needs to update to increase production in the following ways:

Collecting the waste refrigerator, it is classified at different sizes and materials it content iron body frame, foam, compressor, plastic etc. are separated. Then need to use drillers to accrued insulation. The refrigerator contains oil and this oil can be cleaned at gravity pressure with care. Then the exterior steel from interior foam is separated. And the foamed are packed. Then it is sorted into material streams having different subsequent processing demands. In mechanical process its metal scrap are made into fine particles through granulation. By taking various separation techniques precious metal particles are further concentrated. Then magnetic separation technique is used to separate magnetic materials like iron, nickel and cobalt from the material stream. The metal aluminum is separated by eddy current separation technique. By applying electrostatic separation technique plastic can be separated. The metal recovery involves various thermal and chemical processes depending on their properties. The Thermal process avoids the liquid waste disposal problems related with wet chemical extraction methods. Thermal burning process is applied to separate metal. Electro-refining process is generally used alter thermal processing for the purification of copper with the separation of precious metals. It has advantages to recover of pure metal products directly from waste streams.

Fig.6: Recycling Process of Refrigerator

It can be used as fiber insulation sheets for doors. A rubber gas kit that keeps door closed. It can save up to $150 a year in energy cost (Converse Energy Future). Repayment is assured. It keeps the equivalent of up to 10 tons of CO_2 out of atmosphere that is equivalent to two cars exits CO_2 in a year. It can also reduce regional power demand. It keeps material out of landfills and it is economically profitable.

F. Other E-Waste Recycling

Other E-waste like Fan, heater, washing machine, power supply , UPS, IPS, fax machine, Rice cooker, Microwave oven, medical E-waste like ECG,EEG,X-ray machine etc. can be recycled. These can be recycled fruitfully using the following process. The entire E-waste products are loaded into a banker which carries these into a vacuum cleaner for cleaning. Then with dishwasher these product are washed out and with rotor shear of primary shredding cuts these into pieces at sizes 150mm.These materials are then agitate with vibrating feeder. In picking station removes contaminants. Batteries and hazard materials are removed from this. Incline belt helps to throw it into secondary shredder with acoustic enclosure. The materials are cut into pieces at 25-30mm with cutting chamber of secondary rotor. Then with vibrating feeder copper wool are sorted. Then it passes through the zigzag shifter and discharging in big-bags. The sorting process sorts heavy and light fraction removing foils, dusts etc. With light fraction it is discharged into big bags. After this process it coveys heavy fraction with the rotor. By over band magnet ferrous fractions are separated and collect ferrous fraction into a container. The remaining ferrous fractions are extracted via rare earth neodymium drum magnet. On-ferrous fraction and remaining waste (mainly plastics) are separated via eddy current separator. The wastes are taken into hammer mill that grinds it into approximately below 8 mm. The screening station separate the materials into below 8mm and 4mm. Sorting material density shifter separates into aluminum, copper, nickel and brass (Engineering solution for waste from electrical and electronic equipment) shown in Fig. 9.

Fig. 7: Aluminum and Copper particles after Recycling (Source: Engineering solution for waste from electrical and electronic equipment)

G. Burying the rest at landfills

The small amount of rest of the e-waste products which can't reuse and recycle have to bury at landfills. The rest unwanted product after recycling process can be buried at land not digging much or less. Because if digging much it will pollute ground water and if it is digging less it will pollute environment. Any cultivation will not run at these landfill areas because the harvest product will also contain hazards product which may cause harm to human health. But farming such as animal farming, poultry farming, cottage etc. may continue at these areas. Human re-habitation may also be done at these areas. So proper use of areas are possible adopting these procedure waste can be managed in a sound manner where the present E-waste products are huge and need more landfill areas.

PROBABILITIES OF SETTING UP INDUSTRIES IN BANGLADESH

For setting up an industry firstly it needs its raw materials availability and a mandatory to set up the process and scope of gaining profit. Considering all these it has found in Bangladesh that it has huge amount of E-waste, but not have proper process. The e-waste collection in here is very poor and primitive and maximum people are unaware about it. Very often the waste collector misuses these e-wastes because of their ignorance and lack of training (Country Analysis Paper-Bangladesh). If these e-wastes product are collected properly industry can be set up. So it is possible to set up industries as this recycling process does not require high experience and critical technology. There is an extra benefit for recycling is that the labour in Bangladesh is available and their cost is low. In Bangladesh some private industries have run their business with it but due to not getting the raw materials (e-waste) available for unskillful collection process and for unawareness of the people, these industries can't run properly and recycle very small amount. If these private sectors along with the government will come forward to use these e-waste products, proper use of e-waste will be possible. Two sectors can be run in Bangladesh to recycle these e-wastes 100%. Formal sector recycles the e-waste and separates the plastics from metal stream. The other sector informal sector is to recover metals from metal stream. The second sector needs somewhat high technology but possible for Bangladesh. Engineering universities can play a vital role in this respect to analyze about these processes and about these materials. Khulna University of Engineering and Technology (KUET) has already run contribution on this. Many countries have run their mobile phone recycling process. This process is easy because the program can be expanded in cooperation with different partners. Different companies like Nokia aim to learn from each of the recycling program and wants to develop the programs to be more lucrative, suitable and efficient. Recycling process can be done in cooperation with various telecom operators, business firm, NGOs and with various technical schools and universities in many countries. They can be attracted by raising awareness about recycling business and profit and showing availability of huge amount of recyclable mobile phone. This partnership business can help to make awareness about this recycling program by education, advertisement in media. Nokia has already started a recycling program in these countries. Various Universities also come forward for this. Technical University of Monterrey in Mexico, United Nation University, Aalto University and Tongii University in Finland and in many other country's universities are working with recycling. Nokia has already run their business 85 countries (Nokia). Among these countries India is also included. So it is a favorable environment for Bangladesh to set up industry and raise its technology.

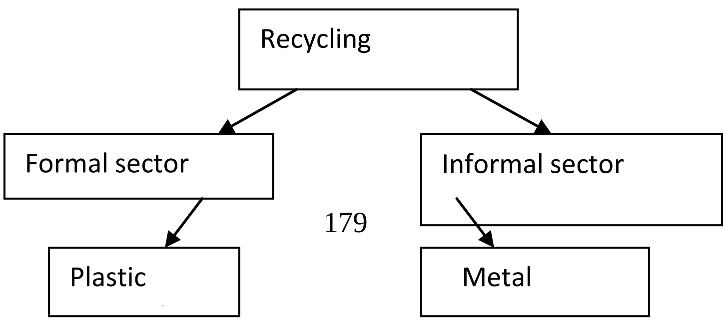

Fig. 8: Recycling Model of E-waste

CONCLUSION

A technological system using e-waste material can be developed in Bangladesh. Reusing, recycling system can be used for this purpose. Industries can be set up efficiently with good profit reusing and recycling the e-waste effectively. This also helps to eradicate unemployment and burying the rest e-waste pollution can be prevented. E-waste materials are the raw material for this. By raising co-operation with both private and public sectors, adopting the above directions and procedures and raising awareness among the people about e-waste, a good profit and healthy environment is possible to ensure.

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ANALYZING THE IMPACT OF CYCLONE MOHASEN ON THE COASTAL CORDON OF BANGLADESH

Md. Shafiqul Islam[1], Md. Hamidul Islam[2*], Ferdaush Ahmed[3], Khursid Alam[3], Asaduzzaman Nayeem[3]

[1] *Department of Geography and Environment, Dhaka University, Dhaka-1000, Bangladesh.*
[2] *Department of Building Engineering & Construction Management, KUET, Khulna-9203, Bangladesh.*
[3] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <hamidcekuet@gmail.com>*

ABSTRACT

The study aims to investigate the impacts of cyclone Mahasen on coastal community in aspect of social, economic and environment. The study also concentrates on options to reduce the impact of such kind of cyclonic hazard. This study is based on both primary and secondary data and information. The primary information for this study was collected through empirical field observation and interview. A questionnaire survey was conducted among 50 sample households. Data and information was also collected from different relevant organizations such as Disaster Management Bureau (DMB), Ministry of Disaster Management and relief division (MoDMRD). Cyclone Mahasen caused at least 17 people death but damaged a vast number of resources in the coastal belt. Government assessments revealed that 49,178 homes were totally destroyed while another 45,825 were damaged. Tens of thousands of trees were downed, causing travel disruptions all of whom were in the southern coastal areas. Thousands of people were also injured and many livestock were killed. The most severe damage and greatest loss of life took place in the Barguna District. Severe damage to agriculture also took place, with many standing crops flattened by gale-force winds. Roughly 128,000 hectares of crops in Patuakhali were damaged, roughly half of which was for sweet potato. A total of 1,285,508 people were affected by the storm throughout the coastal cordon of Bangladesh. To reduce the impact of cyclone some kind of important things should be done as sheltering by proper shelter management also increase the number of cyclone shelter in the most cyclone prone area, early warning dissemination system, build disaster resilient coastal housing, embankment and so on.

Keywords: Impact, Assessment, Cyclone, Coastal area and Bangladesh.

INTRODUCTION

Bangladesh is a densely populated country. She faces a couple of natural disasters in every year. Cyclone is one of them. After Cyclone Sidr, Aila, Nargis the Cyclone Mohasen moved Bangladesh coast through Bagerhat, Patuakhali, Bhola, Noakhali and Chittagong districts on May 16, 2013 at 15:00 hour local time and causing widespread damages the social, environment and economy of this country. According to SOS Form of Bangladesh Government for Cyclone Mahasen as on 16 May, 2013 a total of 13 people died, 1,285,508 people affected, 49,178 houses fully and 45,825 houses partially damaged in 312 unions of 39 upazila (sub-district) under 10 districts [1]. The Bangladesh government distributed 3,501 MT of rice and BDT 12.3 million (approx. CHF 145,000) in cash to those affected by this tropical cyclone. The government also supported around 2,000 families to re-build damaged shelters in Barguna, Patuakhali and Bhola districts through cash-based support [3]. Cyclone Mohasen affected in several sectors such as WASH, food security, agricultural, shelter, education, health and so on. Due to the geographical location of these districts (coastal islands) they are very vulnerable to cyclones and tidal surge. [2]. Gender discrimination at shelter, Public Superstition, Lack of warning and other knowledge about cyclone, lack of awareness, low number of cyclone shelter, fragile socio-economic condition, low material of houses are the responsible for increasing the impact of cyclone in the coastal area of Bangladesh. Government steps regarding cyclone disaster management is few. The objectives of this study as follows (a) to investigate the impacts of cyclone Mahasen on coastal community in aspect of social, economic and environment (b) to identify options to reduce the cyclonic effects.

METIARLS AND METHODS

Study population

This study based on both qualitative and quantitative methods which were carried out during 5th June 2013 to 5th December 2013 in Cyclone Mohasen affected coastal districts of Bangladesh. The study populations include male, female, local, national as well as international NGO$_{s.}$ (>18 years of respondents age).

Sampling

Male and female informants were identified from randomly selected 50 households in the Cyclone Mohasen affected communities.

Data collection

Data for the study were collected in mainly two ways include primary and secondary data collection. The primary information for this study was collected through empirical field observation and interview. For these 12 Focus Group Discussions (FGD), 5 Key Informants Interview (KII$_S$) and 10 observation session were carried out from different Mohasen affected areas. Data and information was also collected from different relevant organizations such as Disaster Management Bureau (DMB), Ministry of Disaster Management and relief division (MoDMRD), local, national as well as international NGOs working in climate change and disaster management in the coastal area of Bangladesh. Besides that data also have been collected from newspaper, published article, Internet.

Map Preparation

Essential Map for this study was prepared by using ArcGIS10.1 of GIS Software.

RESULTS AND DISCUSSIONS

Affected Areas

Firstly Cyclone 'Mahasen' made land fall between Bhola and Patuakhali districts of Barisal division and then moved over southern Bangladesh. This tropical storm mostly affected in Barguna, Patuakhali, Bhola, Chittagong. Moreover many districts of coastal area of Bangladesh mainly exposed coast were affected by cyclone Mohasen.

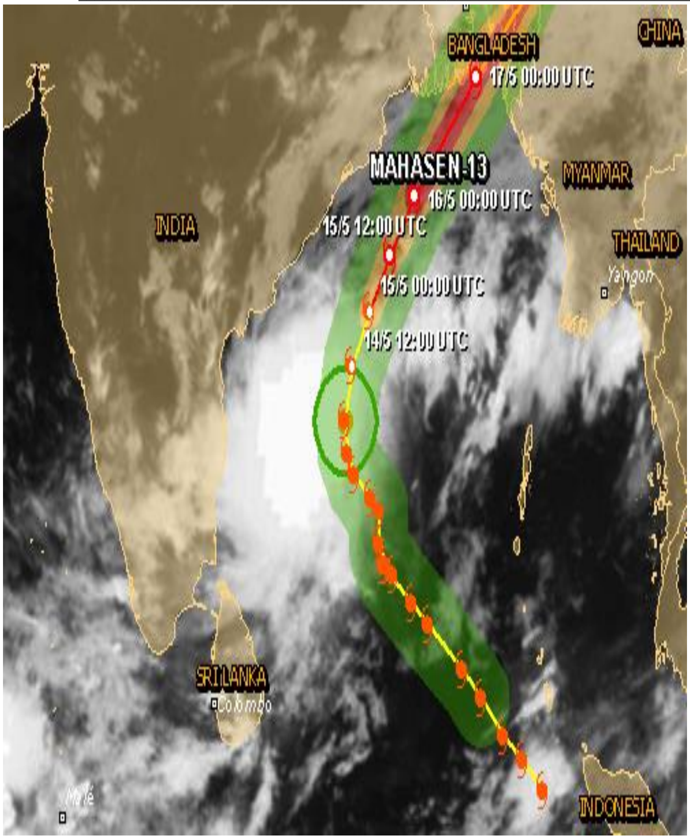

Fig 1: Mohasen Storm truck (Source: Internet)

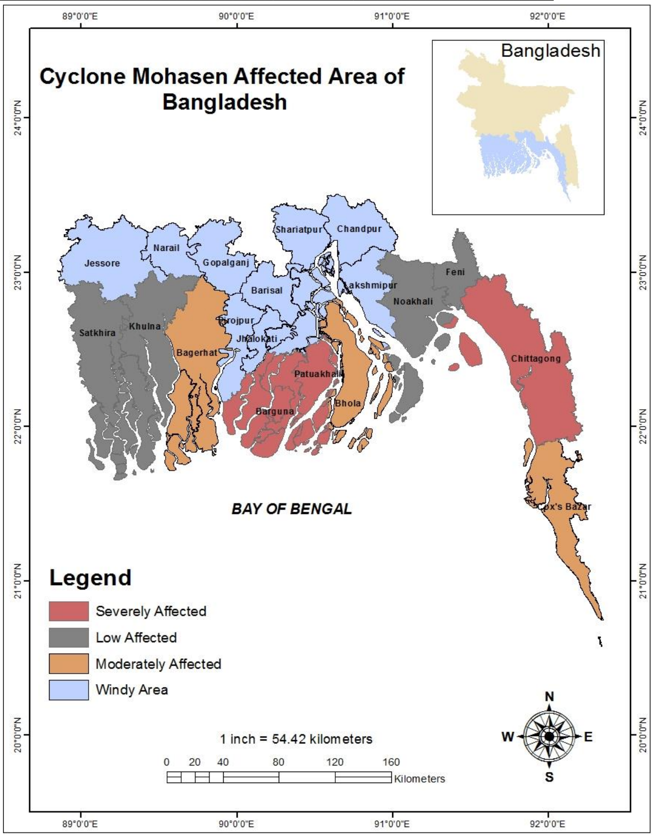

Fig 2: Affected area (Source: Authors)

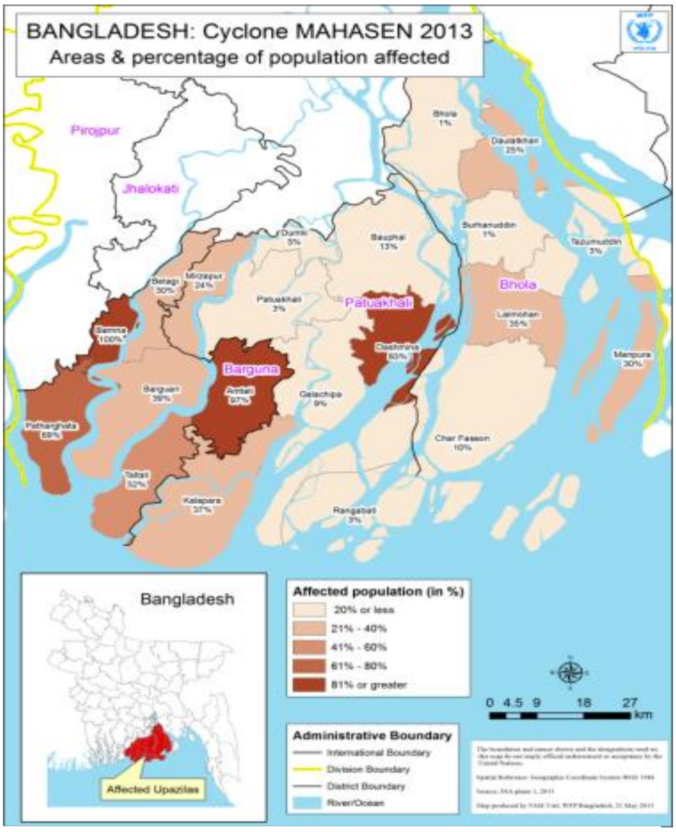

Fig 3: Areas & percentage of pop^{n} affected (Source: WFP)

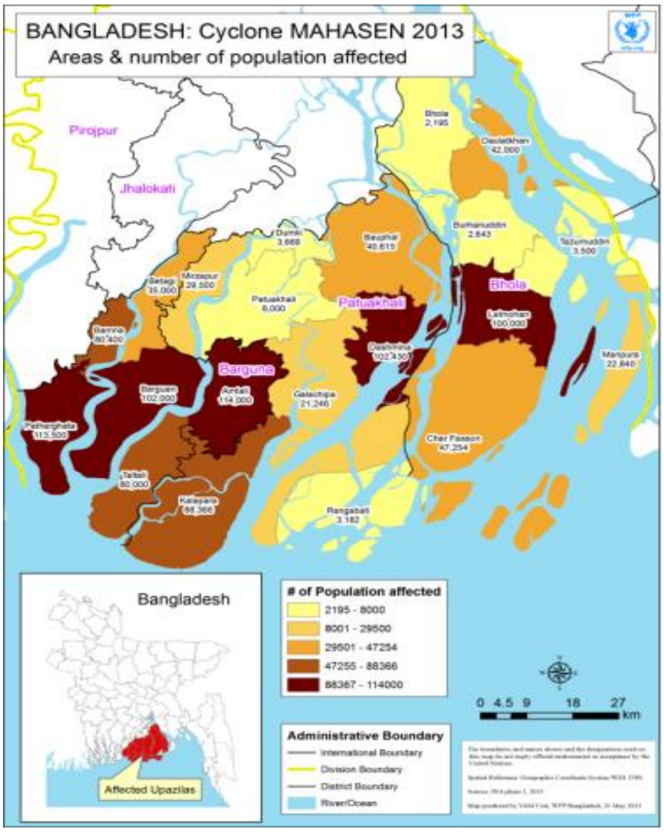

Fig 4: Areas & number of pop^{n} affected

(Source: WFP)

From this above Figures it is seen that Barguna, Patuakhali and Chittagong were severely affected, Bhola, Cox's Bazar, Bagerhat were moderately affected and Khulna, Shatkhira, Noakhali were low affected area of tropical storm Mohasen.

Impacts of Cyclone Mohasen

Tropical storm Mohasen has impacted in environmental, social as well as economic sectors. Affected sectors are as (a) Agriculture (b) housing (c) health (d) water and sanitation (e) education (f) shelter (g) food and nutrition and so on. The Government of Bangladesh reported the following affected districts with number of death, affected people and number of household damaged as given below;

Table 1: Affected districts by Cyclone Mohasen

District	Affected Upazila (No.)	Affected Union (No.)	Deaths	Affected People	House Fully Damaged	House Partially Damaged
Chittagong	4	28	2	54,295	50	2,005
Noakhali	5	33	0	35,127	1,710	4,968
Shatkhira	2	5	0	N/A	N/A	N/A
Patuakhali	8	72	3	70,409	7,540	18,238
Jhalakathi	4	30	0	5,392	55	1,934
Barguna	6	30	7	60,000	6,856	61,812
Pirojpur	7	46	1	60,690	448	5,641
Bhola	7	64	4	167,500	6,760	14,730
Lakshmipur	4	15	0	9,890	120	359
Total	**47**	**323**	**17**	**463,303**	**23539**	**109,687**

Source: National Disaster Response Coordination Centre, Ministry of Disaster Management and Relief (2013)

Major Findings

Cyclone Mahasen caused at least 17 people death but damaged a vast number of resources in the coastal belt. Government assessments revealed that 49,178 homes were totally destroyed while another 45,825 were damaged. Tens of thousands of trees were downed, causing travel disruptions all of whom were in the southern coastal areas.

Cyclone Mohasen affected in a large scale agriculture land in four districts as Barguna, Bhola, Patuakhali and Chittagong. 59% cultivated agriculture land affected in Barguna distrcits and 48% affected in Patuakhali district as well. Most of the agriculture land is still under water. Affected crops including as Aus seed beds and broadcaste, boro, chili, mung bean, sesame, vegetable, ground nut, sunflower, sweet potato, maise betel leaf and banana. Loss of production for Boro, maise and mung bean is inadequate as maximum was already harvested.

Approximately 1,285,508 people affected by the Cyclone Mohasen in all affected districts. Most affected groups are mainly day laborers in both agricultural and non-agricultural small and marginal farmers. Barguna is worst affected district considering the scale and proportion of the population affected. Bhola is the immediate priority based on the number of people displaced and food insecurity condition. Around 6400 to 39000 people displaced in cyclone Mohasen affected area according to interviewed Upazila Chairman of these areas.

Most of the tube-wells are inundated by storm water and water became polluted. As a result people are affected in various water borne diseases such as Diarrhea, Cholera, Typhoid etc. Also the mosquito breeding ground increases due to the water logging. 69% latrines were damaged and 61% tube-wells as well.

Livestock (goats, chickens, ducks, cows, and sheep's) have been affected. Total 20,371 livestock and poultry farming affected in above mentioned four districts.

13% educational institutions also affected in tremendously. Besides that 28% transportation (Kutcha, Pucka and semi-pucka road) are damaged in four districts. According to DMIC, 118792 shelters have been damaged where 19535 were fully and 99452 partially damaged.

Day laborers who get 204 days employment opportunity annually and most of them were engaged in doing labor in agriculture field and earth cutting. Due to cyclone Mohasen they became unemployed.

Most of the farmers are now redundant and borrowing money for meeting their basic needs because above 60% lost their standing crops. Fisherman lost their fishing ground as a result of inundation and also lost the fishing equipment's.

Rickshaw pullers, poultry farming holders, Feriwala who temporary lost their earning source activities due to inundation and damage of roads.

The low lying areas of these districts were inundated by 3-5 feet water.

Environmental pollution (water pollution, soil pollution, air pollution) also created.

Various NGOs and INGOs are working for meeting the affected people's basic needs by providing essential relief assistance. Organization including Red cresent society, BRAC, CARITAS, PLAN INTERNATIONAL, OXFAM, Save the Childreen and so on. Government also extended their helping hand for the Cyclone affected areas people.

People perception on Cyclone

The frequency and intensity of cyclone can be affected in a large way to the people if the preparedness and early warning system is beyond the people. To know the people perception on Cyclone the study identified some option in this regard as depicted in Figure 3.

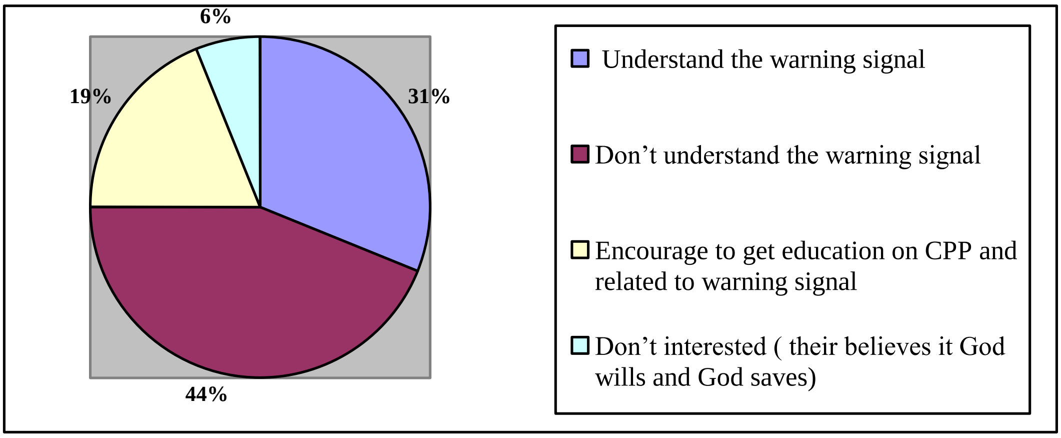

Fig 5: Respondents perception on cyclone and its Warning signals

Way of Reducing Adverse Impacts of Cyclone

For reducing the impacts of cyclone in the coastal areas of Bangladesh proper steps should be taken as follows; (a) Increase the cyclone preparedness program (b) increase awareness about cyclone warning and dissemination system (c) cyclone forecasting system should be increased (d) Reconstruct the entire fragile embankment (e) Set the block in the slop of embankment (f) Repairing the damaged roads and raising the roads elevation above the flood level (>7-8m) (g) Raising the height of the house basement above the average storm surge level and build disaster resilient house (h) more trees should be planted in road side to protect from land slide because storm water make saturated soil (i) adequate number of cyclone shelter should be established considering the vulnerability of zone (j) cyclone shelter should be established in the right place considering people easy accessibility in the time of cyclone (k) to plant salt tolerant agriculture variety (l) raising the bank of pond to protect fish species from the inundation (m) Tube-wells and latrines should be built above the storm water level (n) more Income generation activities and alternative income source should be created for supporting during and after the crisis period (o) increase the knowledge about the rain water harvesting system (p) Establish ward wise cyclone early warning and disaster information center (q) should be started more community radio center which will be used in cyclone time by providing warning signals and dissemination. More option should be considered as strengthen the indigenous practice to cope with cyclonic hazard such as tree plantation around the houses, raised houses platform, store seeds, grain, foods and valuable documents in digging a whole, open disaster insurance, increase public awareness about cyclone and its preparedness, response and recovery procedure. Government and Non-government organization should more concentrate on this issue.

CONCLUSION

Cyclone Mohasen impacted in social, economic and environmental aspects in the coastal areas of Bangladesh. The study revealed that three districts were worst affected named Barguna, Bhola and Patuakhali. Agriculture is mostly affected sector by cyclone Mohasen. This caused more than fifteen people death and vast coastal resources damaged in the coastal belt. Most affected groups are mainly day laborers (in both agricultural and non-agricultural) small and marginal farmers. Due to the poor disaster management system and ignorance of the physical plan, every year the country has experienced not only huge economic lose but also lives. Government and some Non-government Organization (NGO) are working for Mohasen affected people for improving their livelihood and minimize their loss. It should be built cyclone resilient house, increase awareness on early warning and dissemination system, need more cyclone preparedness program for reducing the impact of cyclone as well as sustainable development.

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THE INFLUENCE OF METEOROLOGY ON PARTICULATE MATTER CONCENTRATION IN THE AIR OF DHAKA CITY

S. Afrin[1], M.M. Islam[1*], T. Ahmed[1] & M.A. Ali[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <maksimulislam075@gmail.com>*

ABSTRACT

This paper aims at understanding the meteorological influences in ambient air quality of Dhaka city by analyzing air quality and meteorological data. Trends in air quality over the past decade show large seasonal variations in both PM2.5 and PM10 concentrations, exceeding the national standards during dry season, while remaining somewhat below the standards during rainy season. The number of rainy days in a month during the wet period has been found to have strong negative correlation with average monthly PM concentrations. However, the daily amount of rainfall during the rainy period does not seem to have any effect on the daily PM concentration as PM2.5-10 showed insignificant negative correlation, while PM2.5 showed insignificant positive correlation. This indicates that although precipitation in general may promote wet deposition of particulates, the daily rainfall itself does not significantly dictate the magnitude of PM and therefore it may not be a significant indicator of air pollution potential. A weak albeit positive correlation of PM with wind speed from north-western direction suggests that dilution of PM due to increased wind speed is very low. The coarse fraction of PM exhibited positive correlation with incident solar radiation suggesting increased formation of coarse nitrate particles by enhanced photochemical activity. Strong negative correlation of PM with relative humidity indicates possible absorption of moisture by particulate matter promoting settling and removal from suspension. Significant negative correlation between PM and temperature indicates that higher PM tends to reduce atmospheric temperature due to its net negative radiative forcing. Results from this study emphasize the need for understanding air quality under the context of local meteorological conditions.

Keywords: Particulate Matter, Air Pollution, Meteorology, Correlation Coefficient

INTRODUCTION

Among the air quality parameters in Dhaka city, the Particulate Matter (PM) has been the most widely studied due to its strong correlation with health outcomes and more recently due to its high concentrations in Dhaka city exceeding national ambient air quality standards during most parts of the year. In Dhaka city, the major sources of PM_{10} have been found to be soil dust, road dust and motor vehicle emissions, while brick-kilns and motor vehicles have been found to be the main contributors to $PM_{2.5}$ (Guttikunda, 2009; Begum et al., 2004).

Bangladesh has a subtropical monsoon climate characterized by wide seasonal variations in precipitation, moderately warm temperatures, and high humidity with minor regional climatic differences. Air quality of Dhaka is also characterized by high seasonal variation (Fig. 1).The daily PM_{10} and $PM_{2.5}$ concentrations exceed the national standards by a factor of about two during the dry season, whereas during the wet season the ambient concentrations of PM have been found to be somewhat lower. Meteorological parameters such as wind speed and direction, precipitation, temperature, relative humidity and solar radiation can affect dispersion processes, removal mechanisms and formation of atmospheric particles and hence govern PM_{10} and $PM_{2.5}$ concentrations. Many studies indicated that TSP and PM_{10} concentrations in ambient air are affected by wind speed, wind direction, solar radiation, relative humidity and precipitation (Alpert et al., 1998; Galindo et al., 2011; Giri et al., 2008; Owoade et al., 2012). A number of studies have been carried out on source apportionment (Begum et al., 2004, 2005; Biswas et al., 2001; Guttikunda, 2009), elemental characterization (Boman et al., 2005), and spatio-temporal distribution of PM in Dhaka city. However, a systematic study examining the relationship of various meteorological parameters with air quality is yet to be done. In this paper, the relationship between meteorological parameters and PM of Dhaka city has been investigated.

MATERIALS AND METHODS

$PM_{2.5}$ and PM_{10} are collected from Shangshad Bhaban CAMS in which high volume PM samplers are used. Daily data of $PM_{2.5}$ and PM_{10} are available from April 2002 to May 2004; and monthly data of $PM_{2.5}$ and PM_{10} are available from April 2002 to April 2014. The PM_{10} concentrations have been divided into two size fractions, $PM_{2.5}$ and $PM_{2.5\text{-}10}$ (=$PM_{10} - PM_{2.5}$) for this study.

Meteorological data for Dhaka city have been collected from two sources: Shangshad Bhaban CAMS and Bangladesh Meteorological Department (BMD). In Shangshad Bhaban CAMS, recorded data on atmospheric temperature, rainfall, relative humidity, and solar radiation are available from October 2002 to May 2004 as hourly averages. 24-hour average data have been used in this study. Daily wind speed and wind direction data have been collected from Bangladesh Meteorological Department (BMD); these data are not available at the Shangshad Bhaban CAMS. The Shangshad Bhaban CAMS meteorological data is used for the period during which it was available, whereas for the remaining periods, meteorological data from BMD were used. For the analysis on meteorological influences, the October 2002 to May 2004 period was further subdivided into three periods which are referred to as Dry period-1 (October 2002 to March 2003), Wet period (April 2003 to September 2003) and Dry period-2 (October 2003 to March 2004). Generally each period has its own unique meteorological conditions that can affect the concentrations of the particulate matter. Dry period is characterized by dry soil conditions, low relative humidity, low or no rainfall and prevailing winds of low speed from the northwest. Pearson's correlation was used to determine the degree of association between different climatic variables and PM concentrations. A significance level of 5% ($p = 0.05$) has been chosen to be the threshold for determining the significance of the correlation coefficient.

RESULTS AND DISCUSSIONS

The number of rainy days in a month during the wet period has been found to have strong negative correlation with average monthly PM concentrations (Fig. 1) with correlation coefficients -0.85 and -0.94 for $PM_{2.5}$ and $PM_{2.5\text{-}10}$, respectively. This indicates that during the closure period of the brick kilns in the wet season, the prevalence of PM in air is strongly associated with rainfall period. However, the daily amount of rainfall during the wet period does not seem to have any effect on the daily PM concentration as $PM_{2.5\text{-}10}$ showed insignificant negative correlation ($r = -0.15$, $p = 0.379$), while $PM_{2.5}$ showed insignificant positive correlation ($r = 0.06$, $p = 0.734$) (Fig. 2). This indicates that although precipitation in general may promote wet deposition of particulates, the daily rainfall itself does not significantly dictate the magnitude of PM concentrations and therefore it may not be a significant indicator of air pollution potential. Figure 2 also suggests that the high concentration of PM during the dry season cannot be explained by lack of precipitation during the dry season; extrapolating the trend of PM variation (with number of rainy days) to days without precipitation (i.e., zero rainy days) yields PM concentrations far below those actually observed during the dry season.

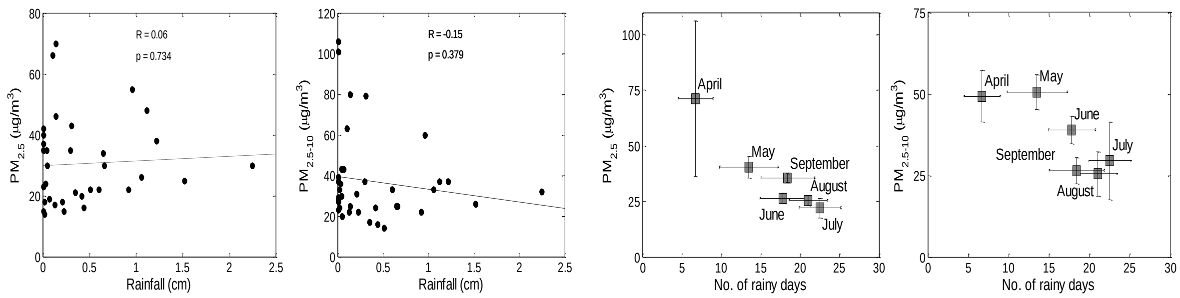

Fig. 2. Average monthly PM vs. number of rainy days in a month during wet period showing a decrease in PM with increased no. of rainy days. Fig. 3. Scatter plot of different PM fractions and rainfall intensity during wet period.

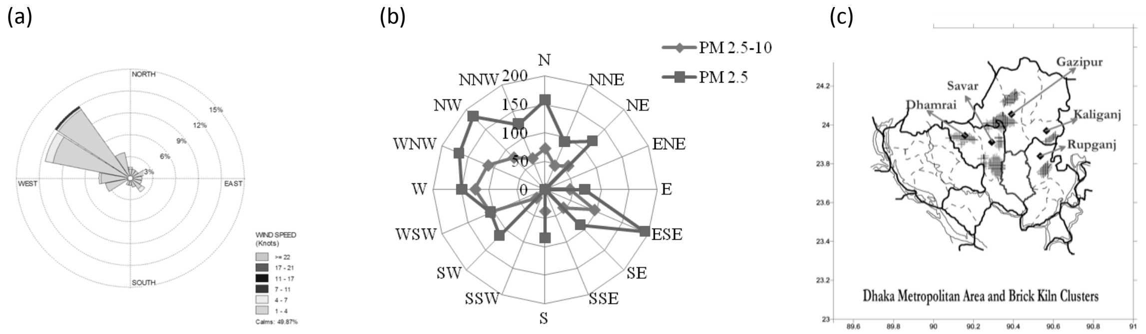

Fig. 3. (a) Wind rose plot in period, (b)Radar plot showing the concentration of $PM_{2.5}$ and $PM_{2.5\text{-}10}$ corresponding to different wind directions during the dry period and (c) Brick clusters around Dhaka City (Guttikunda, 2009)

The residence of PM in the atmosphere and their transport are also controlled by certain physical parameters such as wind speed and direction. The direction and speed of the prevailing wind may significantly affect the concentration, distribution and translocation of the particles (Giri et al., 2008; Van der Wal et al., 1996, 2000). Wind speed and direction provide real-time information on pollutant transport in a region and can be used to assess the relationships between sources and pollutant levels. Wind rose plots (Fig. 3a) show that the predominant wind direction for dry is north-west (NW) and for wet period it is south-east (SE). In order to study the effect of wind speed and direction on PM concentrations, only dry period dataset has been used as the major pollution sources located outside the city boundary (i.e., the brick kilns) remain closed during wet period. Winds coming from different directions are grouped into 16 categories such as N, NNE, NE, ENE, E etc. and for each specific direction, the days on which winds were coming from that direction have been identified and the concentrations of PM for those particular days have been averaged. Both $PM_{2.5}$ and $PM_{2.5\text{-}10}$ show higher concentrations when winds come from NW direction (Fig. 3b). This is expected as the major brick clusters of Gazipur, Savar and Dhamrai are situated along the northwestern periphery of Dhaka city (Fig. 3c), and wind blowing from those directions will likely carry the particulate pollution load. There is a slight confounding factor of increased concentration due to southeastern wind direction because there are no known brick kiln clusters in that direction. This indicates that there might be industries in that direction other than brick kilns that might contribute higher particulate concentrations. The likely contributors are the industrial installations in Narayanganj which are mostly located along the banks of the Sitalakhya River. The strong association between wind blowing from the north and northwestern direction and PM concentrations supports the hypothesis that during the dry period brick clusters of Gazipur, Savar and Dhamrai significantly dominate the air quality of Dhaka city. In order to understand whether wind speed has any bearing on the PM concentrations the wind from North (N) and Northwestern (NW) directions were correlated with the corresponding daily $PM_{2.5}$ concentrations and the scatter plots are shown in Figure 4. For winds blowing from the north

(N) direction, a weak negative correlation was found between wind speed and $PM_{2.5}$ for both dry period-1 (r = -0.18) and dry period-2 (r = -0.34). A weak positive correlation was found corresponding to wind blowing from north-west (NW) (r = 0.19 and 0.27 for dry period-1 and dry period-2 respectively). However, all of these correlations were found to be insignificant (p>0.05) indicating that wind speed does not appear to have a significant influence on PM concentration in Dhaka.

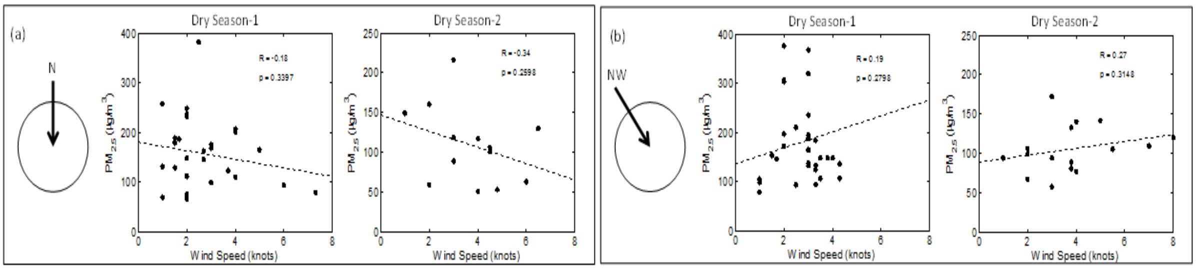

Fig. 4. Scatter plot showing the relationship between $PM_{2.5}$ and wind speed in dry season for two different wind directions: (a) north, and (b) and northwest;

$PM_{2.5-10}$ showed fairly good positive correlation (Fig. 5) with solar radiation (r = 0.36, 0.32 and 0.30 respectively for dry-1, wet and dry-2 periods, p>0.05 in all cases) while $PM_{2.5}$ showed no significant correlation (not shown in figure, see Table 1). The correlation coefficients corresponding to $PM_{2.5}$ are -0.004, 0.02 and -0.08, respectively for dry-1, wet and dry-2 periods. This could be due to formation of coarse nitrate particles by photochemical activity (Giri et al., 2008; Nicolas et al., 2009). Nitrogen oxides are oxidized to nitric acid in the atmosphere, which in turn form nitrate particles (Matsumoto and Tanaka, 1996). This conversion is dependent on the degree of photochemical activity (Chang et al., 1979) i.e. the intensity of incident solar radiation. Mehlmann and Warneck (1995) reported that particulate nitrate existed mainly in the coarse size range and it can be expected that larger fraction of PM would be positively correlated with the degree of solar radiation.

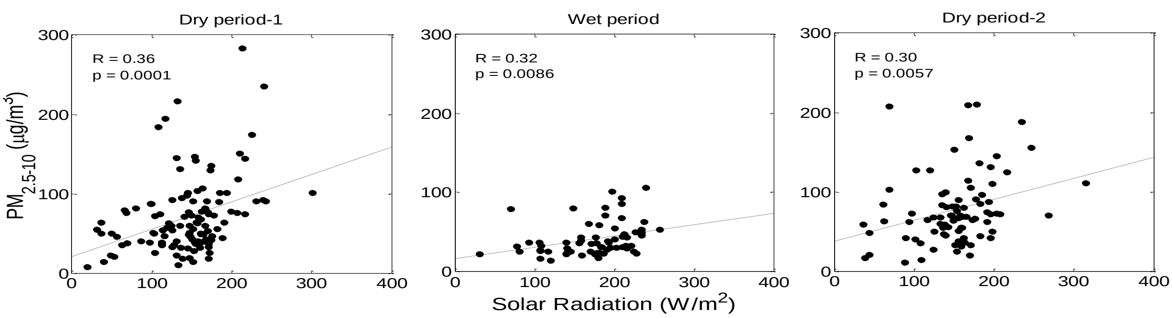

Fig. 5. Scatter plot of $PM_{2.5-10}$ and Solar Radiation during dry period-1, wet period and dry period-2 showing positive correlation between the two variables in all three periods.

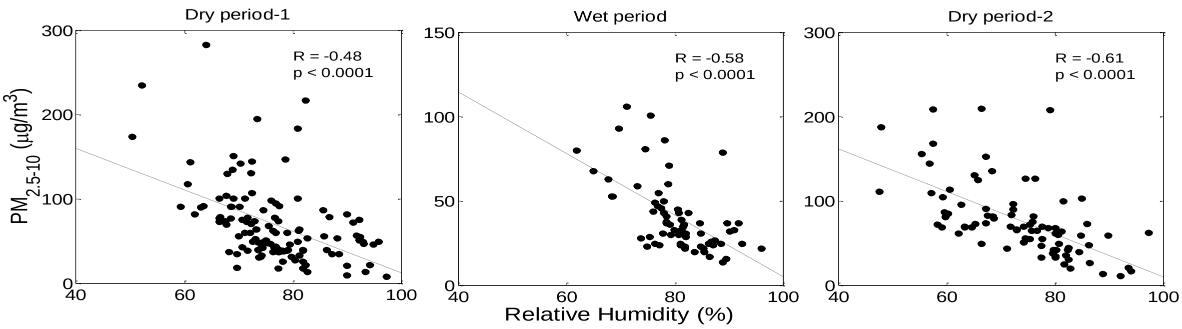

Fig. 6. Scatter plot of $PM_{2.5\text{-}10}$ and Relative Humidity during dry period-1, wet period and dry period-2 showing negative correlation between the two variables in all three periods.

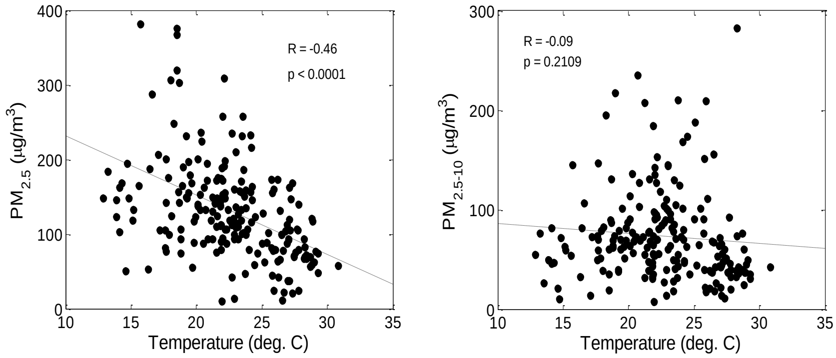

Fig. 7. Scatter plot of different PM fractions and temperature. The dashed line represents the trend of the relationship as determined through linear regression.

Humidity influences particles to gather mass and settle down on the ground. Because of this, the average PM concentrations have been found to show a decreasing trend with the increase in ambient relative humidity (Giri et al., 2008). From the air quality data of Dhaka city, the coarser fractions ($PM_{2.5\text{-}10}$) of particulate matter show comparatively better negative correlation with humidity (Fig. 6) in all periods (r = -0.48, -0.58 and -0.61 respectively for dry-1, wet and dry-2 periods, $p<0.05$ in all cases), compared to finer fraction (not shown in figure, see Table 1). The correlation coefficient of finer fractions for the three periods have been found to be -0.09,-0.42 and -0.17, respectively. This may be due to the fact that coarser PM fraction dominated by soil/road dust are likely to absorb more moisture compared to finer PM fraction coming mainly from vehicular and industrial (including brick kiln) sources.

A wider range of temperature prevails during the dry periods and $PM_{2.5}$ show strong negative correlations (r = -0.46, $p<0.05$) with it, while a weak negative (r = -0.09, p = 0.2109) is obtained for $PM_{2.5\text{-}10}$ (Fig. 9). Similar observations have been reported for Cairo (Elminir, 2005) and Kathmandu Valley (Giri et al., 2008). Significant negative correlation between PM and temperature implies that higher PM tends to reduce atmospheric temperature due to net negative radiative forcing, which induces a cooling effect. Wet period shows narrow range of temperature in Dhaka city which is not significant to establish any trend.

Table 1. Pearson's correlation coefficient (r) of Solar Radiation (SR), Relative Humidity (RH), Precipitation, NO_x and Wind Speed (WS) with PM fractions

Time Period	PM Fractions	SR	RH	Precipitation	NO_x	WS	
						N-Direction	NW-Direction
Dry Period-1	$PM_{2.5}$	-0.004 (0.965)	-0.09 (0.3)		0.63 (<0.0001)	-0.18 (0.339)	0.19 (0.279)
	$PM_{2.5-10}$	0.36 (0.0001)	-0.48 (<0.0001)		0.16 (0.136)		
Wet Period	$PM_{2.5}$	0.02 (0.859)	-0.42 (0.0005)	0.06 (0.734)	0.62 (<0.0001)		
	$PM_{2.5-10}$	0.32 (0.009)	-0.58 (<0.0001)	-0.15 (0.379)	0.07 (0.595)		
Dry Period-2	$PM_{2.5}$	-0.08 (0.512)	-0.17 (0.109)		0.64 (<0.0001)	-0.34 (0.259)	0.27 (0.315)
	$PM_{2.5-10}$	0.30 (0.006)	-0.61 (<0.0001)		0.31 (0.0015)		

CONCLUSION

In this study, the probable association between meteorological parameters and particulate matter (PM) pollution dynamics in Dhaka city has been explored Daily precipitation appears to have trivial influence on PM reduction through wet deposition, though the number of rainy days in a month showed significant negative correlation with PM concentration. The insignificant correlation between wind speed and PM for winds coming from northern and northwestern direction indicates that wind speed does not appear to have a significant influence on PM concentration in Dhaka. Statistically significant negative association between relative humidity and coarser PM fraction ($PM_{2.5-10}$) indicates the reduction of coarser PM concentrations, possibly through promotion of wet deposition. Significant negative correlation between PM and temperature implies that higher PM tends to reduce atmospheric temperature. The solar radiation appears to contribute to increase coarse PM fractions, possibly by increasing formation of coarse nitrate particles. These results indicate that apart from prevailing wind direction and prevalence of rainfall, solar radiation, relative humidity and air temperature can be contributing factors in governing the PM concentrations in Dhaka city. Some of these meteorological events are not completely dissociative (e.g. increased humidity would naturally be positively correlated with rainfall events) and hence drawing a conclusion based on a particular meteorological parameter to predict the PM concentration would be unwise.

Thus, meteorological parameters should be given due consideration while selecting locations of industrial clusters that are major sources of emissions. This type of study can be useful in suggesting mitigation measures to control ambient PM concentrations. Such mitigation measures may include relocating the brick kiln clusters to a location which will cause the minimum impact on Dhaka city under the prevailing meteorological conditions.

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NOISE POLLUTION IN CHITTAGONG CITY

M. A. Kader[1*], M.A.I Khan[1] & S.K. Palit[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering &Technology, Chittagong-4349, Bangladesh.*
**Corresponding Author, <mak.ce.cuet@gmail.com>*

ABSTRACT

Chittagong is the second largest city of Bangladesh where people are increasing day by day in search of their earnings and livings. As a result, the numbers of various types of vehicles are also increasing for their movement. These vehicles are mainly responsible for noise pollution in the city. This noise pollution causes various types of physiological & psychological disorders. So an attempt has been made by the Department of Civil Engineering, CUET, to investigate the present traffic noise scenario for the Chittagong city. A sophisticated digital Sound Level Meter (SLM) has been used for the assessment of noise levels at some important locations like some intersection points, academic institutions & hospitals, commercial area and residential area. From the study, it has been observed that noise level in Chittagong city exceeds the noise standards suggested by the Environmental conservation rules, 1997 Bangladesh (ECR'97) and the noise level is highly alarming and unsafe in various locations for human. This study also highlighted the noise polluted and vulnerable areas through the tabular analysis. It has also been observed from the study that places with high traffic congestion, narrow roads, heavy constructional activities and poor traffic management areas are more vulnerable to high noise levels. Some of the educational institutions, hospitals and nursing homes are also in the grip of high noisy environment.

Keywords: noise pollution; noise level dB (A); sound level meter (SLM); intersection.

1. INTRODUCTION

Background

The word 'noise' is derived from Latin word "nausea" implying 'unwanted sound' or 'sound that is loud, unpleasant or unexpected'. It is one of the most immediate and identifiable environmental problem associated with rapid industrialization, urbanizations and population growth.

However due to its invisible impacts and associated health problems, people are least concern about noise pollution and ignore its menace. Generally, high exposure to noise level can causes feeling of annoyance and irritation, damage to auditory mechanisms, number of health related effects like physiological disorders, psychological disorders, disturbances of daily activities and performances, hypertensions and ischematic heart diseases. The most serious health hazards associated with high level of noise exposure is deafness which initially causes temporary hearing problem or deafness while prolonged exposure to high noise level causes permanent hearing damage. Primary cause of hearing loss due to exposure to noise is termed as Temporary Threshold Shift (TTS) and due to very high level exposure to noise may cause Noise Induced Permanent Threshold Shift (NIPTS).

Previous Study

Various studies have been carried out in Bangladesh revealed that some of the Bangladeshi cities are having high noise level than the standards prescribed by ECR'97 and MOEF (Ministry of Environment and Forest, Govt. of Bangladesh). The extensive noise assessment carried out recently showed high noise levels in various locations of Chittagong city. The study carried out by Department of Environment revealed that noise levels exceeded 109 dB (A) in commercial areas, 92 dB (A) in residential areas and 91 dB (A) in silence Zones during Eid, Durga puja and other religious and cultural festivals. Department of Environment, Chittagong also indicated higher noise levels in many locations of the city than the noise standards prescribed by ECR'97. Again an individual study carried out by a Bangladeshi environmentalist named Anwar Hossain Manju in the year 2001 (Source: Holiday, September 14, 2001). His study has been indicated that out of 45 areas, 28 showed noise level exceeding the permissible limit and 17 showed noise level fluctuating in an around the permissible noise level.

Objectives

To assess the noise pollution problem in Chittagong city the following objectives have been selected

To determine the noise level of some specified locations in Chittagong city

To compare the noise level with ECR'97 guideline value

To identify the mostly polluted area

2. METHODOLOGY

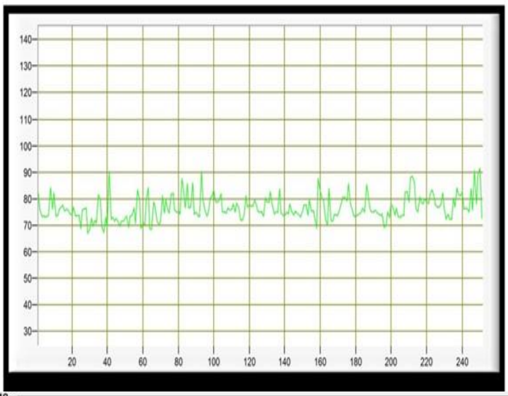
A noise level survey has been carried out to assess the noise environment of Chittagong city in some selected i.e. in heavy traffic zones, highly populated areas and areas with educational institutions, hospitals and nursing homes. The noise levels have been assessed with Sound Level Meter (SLM) (model: TES 1353H Integrating Sound Level Meter) has been shown in Fig.1 and Fig.2 shows one of the noise pollution sources. This instrument measures the sound pressure level in dB (A) i.e. decibels in A weighted scale.

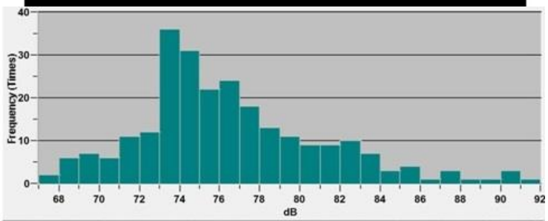
Fig.1 Integrated sound level meter

Fig.2 Noise pollution source

The sound pressure level or sound level measured in decibel (dB) is a logarithmic measure of the effective sound pressure of a sound relative to a standard reference value. The dB (A) Leq denotes the time weighted average of the sound pressure level in decibels on scale 'A' which is relatable to human hearing.
A weighting is the most commonly used curves defined in the International standard IEC 61672:2003 and various national standards relating to the measurement of sound pressure level. The sound level meter consists of microphone or receiver, preamplifier, weighting network, amplifier, rectifier and digital LCD display meter.

By connecting this integrating sound level meter to the computer via USB, it can generate bar graph and plain graph with the help of software supplied by the manufacturer. The graphical representation of their software has been shown in Fig.3 & Fig.4.





Ln	0	1	2	3	4	5	6	7	8	9
L(00)	90	90	87	87	85	85	84	83	83	83
L(10)	82	82	82	82	81	81	81	80	80	80
L(20)	80	79	79	79	79	78	78	78	78	78
L(30)	77	77	77	77	77	77	77	76	76	76
L(40)	76	76	76	76	76	76	76	75	75	75
L(50)	75	75	75	75	75	75	74	74	74	74
L(60)	74	74	74	74	74	74	74	74	73	73
L(70)	73	73	73	73	73	73	73	73	73	73
L(80)	73	73	72	72	72	72	72	71	71	71
L(90)	71	71	70	70	69	69	69	68	68	67

Fig.3 software generated plain graph

Fig.4 Software generated bar graph

3.INVESTIGATION

Physical Investigation

For the purpose of our investigation, we visited Chittagong city physically. From our physical investigation it has been observed that various locations have the higher number of traffic and they seemed to be noisier. Among them the following locations have been selected for the experimental investigation-

1. Kaptai rastar matha
2. BAWA School & College
3. Bahaddar hat
4. Hazera-Taju College
5. Muradpur intersection
6. Chittagong metropolitan hospital
7. 2no gate intersection
8. Khulsi residential area
9. GEC intersection
10. Sugandha residential area
11. New market intersection
12. Agrabad commercial area
13. Andorkilla intersection

Experimental Investigation

The sound level of our study areas have been measured for few consecutive months. Average time of 10 minutes was followed for measuring noise levels at each location at different time of the day. The sound level meter was handled at proper orientation to receive the maximum sound intensity. The same procedure was maintained for all the survey points in the city. The noise assessment was conducted during morning (9 to 11 am), noon (12 to 2 pm) and evening (6 to 8 pm) of the day. An appropriate distance and proper orientation of microphone of the sound level meter (SLM) was maintained for each sampling locations. Fig.5 shows the sound level measurement in the study areas.

Fig.5 Sound level measurement in the field

4. RESULTS & DISCUSSION

This noise survey reveals that noise environment of the Chittagong city is not so satisfactory as per the standards. The measured noise level at various locations in the city have been presented in Table 1 together with standard value specified by "The Noise Pollution (Regulation and Control) Rules, 2006".

In Table 1, day time mean from 6.00 am to 9.00 pm and night time from 9.00 pm to 6.00 am. & the silence zone is an area comprising not less than 100 meters around hospitals, educational institutions, courts, religious places or any other area which is declared as such by the competent authority. From Table 1, it is observed that the noise level in commercial location (more than 80 dB(A)) of the city during the day time at 12 noon to 2 pm is higher due to plying of more vehicles and huge accumulation of traffic than the noise level at morning (9 to 11 am) and evening (6 to 8 pm).

Similarly, intersection points also showed higher noise levels than the prescribed noise standards. In some residential areas like Sugandha and Khulshi situated nearby roadside showed higher noise level i.e. more than 65 dB(A). Most serious problem associated with high noise levels of the study areas are in silence zones of the city. The study showed that the silence zones including hospitals, nursing homes and educational institutions in the city are in the grip of heavy traffic noise and disturbances.

Table 1: Comparison between measured and standard sound level

Type of area	Locations	Day time, dB(A) Leq		Night time dB(A) Leq	
		Measured	Standard	Measured	Standard
	Kaptai rastar matha	71		68	

Intersection points(Mixed area)	Bahaddar hat	78	60	67	50
	Muradpur	75		65	
	2no gate	76		68	
	GEC	81		72	
	New market	83		70	
	Andorkilla	75		67	
Silence zones	BAWA school & college	69	50	58	40
	Hazera-Taju college	64		55	
	Chittagong Metropolitan hospital	62		53	
Commercial area	Agrabad	82	70	74	60
Residential area	Khulshi	66	55	57	45
	Sugandha	68		61	

5. CONCLUSION

The noise assessment of the Chittagong city indicated that the noise levels in the city is escalating at a very fast rate with growing population and heavy traffic accumulation. Noise levels obtained at different locations of the city viz. intersection points, commercial, residential and silence zones are found to be exceeding the noise level /limits. It also observed that higher noise level in the city is due to rapid and unplanned urbanization, improper management of city roads and traffics, lack of sufficient parking spaces and exponential growth of both private and public vehicles in the city.

Narrow linking roads, absence of arterial roads and lack of flyovers & over bridges in some locations of the city are responsible for huge gathering of vehicles resulting in a chaotic and noisy environment. Thus it is felt that noise environment of Chittagong city may pose as a great threat to the health of dwellers of Chittagong city in long term. This is because high level of noise may not cause serious or immediate effects but if, such noisy environment prevails, it may impact the population in many ways.

Therefore, a strict enforcement of law and regulation is felt in this regard. Also, creating diversion for heavy vehicles outside the city, planting of more trees in roadside for sound cushion, constructing noise barrier wherever necessary, broadening the roads, constructing flyovers and over bridges, creating more parking spaces are some remedial and preventive measures which can be considered for improving the noise environment of the Chittagong city.

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PHARMACEUTICAL WASTEWATER MANAGEMENT (WWM): A REVIEW

M. F. Zaman[1*], M. S. Akter[2] & I. B. Muhit[3]

[1] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong, 4349, Bangladesh.*
[2] *Department of Pharmacy, Gono University, Dhaka, 1344, Bangladesh.*
[3] *Department of Civil Engineering, Chung-Ang University, Seoul, 156-756, Korea.*
**Corresponding Author, <ferdouszaman85@yahoo.com>*

ABSTRACT

The wastewater which is generated from the pharmaceutical industry of Bangladesh is a big concern for the environment nowadays. There is an enormous amount of the pollutants that is being dumped constantly from various pharmaceutical industries which causes severe damages to the environment and public health. Because, it contains high organics as well as inorganic loadings, that's why it demands proper treatment before final disposal to the environment. This paper contains a review of the hazard that may cause to the environment due to the disposal of untreated pharmaceutical wastewater. It also proposes an assessment of the treatment and disposal systems that are being used by all developed world for treating pharmaceutical wastewater. Moreover, a discussion about the efficiency of currently used methods and the future development make this paper a special contribution to the wastewater management sector of Bangladesh.

Keywords: COD, Treatment, Aerobic, Anaerobic, Reactor

INTRODUCTION

By the term 'pharmaceutical' a wide-ranged class of compounds with substantial variability in structures, function, behavior, and activity are covered. They are developed to evoke a biological effect, and are used in both humans and animals to cure disease as well as fight infection and/or reduce symptoms (Abou-Elela *et al*, 2012). Pharmaceutical wastewater contain a variety of organic and inorganic constituents including spent solvents, catalysts, additives, reactants and small amounts of intermediates and products and sometimes it may therefore be high in chemical oxygen demand (Fent *et al,* 2006; Oktem *et al,* 2007). Some pharmaceutical wastewater can have COD as high as 40,000 mg/L (Fox and Venkatasubbiah., 1996). These wastewaters also contain relatively high levels of suspended solids and soluble organics (Chelliapan *et al*., 2011). Approximately half of the pharmaceutical waste waters produced worldwide are discarded without specific treatment (Lang, 2006; Enick and Moore, 2007). The presence of these hazardous wastes can cause serious damage to the environment especially surface water (Abou-Elela *et al*, 2012). Hence proper treatment of pharmaceutical wastewater is essential before disposal.

Anaerobic processes have become a viable option for the treatment of medium high strength industrial wastewaters (Lohchab and Snehlata, 2013). A number of reactors are available for anaerobic treatment of pharmaceutical wastewater such as upflow anaerobic sludge blanket (UASB) reactor, up-flow anaerobic stage reactor (UASR) and anaerobic packed bed reactor. In last few decades the aerobic biological treatment technology has been developed and used in different industrial wastewater treatment process. The treatment process of the bioreactor largely depends on aeration (Khan and Mostafa., 2011). An alternative to the conventional process is electro-coagulation, which consists of the in situ generation of coagulants by the electro-dissolution of a sacrificial anode (Dixit and Parmar., 2013).

The objective of current study is to analyze the conventional techniques of pharmaceutical waste water treatment and assess their efficiencies. Besides electro-coagulation technique will also be analyzed and its performance will be assessed in the current study.

METHODS OF TREATMENT

Anaerobic Treatment

a. Packed Bed Reactor: In a laboratory scale the anaerobic packed bed reactor is a PVC cylindrical reactor with a capacity of 22.5 L with plastic media packing. The influent wastewater is entered through an internal down-comer tube in the head-plate that extended to within 20mm of the reactor base and allowed feed to flow up-ward through the sludge bed. The walls of the reactors are wrapped with a tubular PVC water-jacket which has 15mm internal diameter in order to maintain the reactor temperature at 37^0C (Chelliapan *et al*., 2011). The start-up of reactor is established initially with a brewery wastewater feed due to its ease of degradation as well as high COD values and well-established use in continuous anaerobic reactors (Sallis *et al*., 2003). The reactor is seeded with anaerobic digested sewage sludge from an anaerobic sludge digester. Once the reactor had reached steady state the feed to the reactor is supplemented incrementally with pharmaceutical wastewater (Stronach *et al*., 1986).

b. Up-flow anaerobic stage reactor (UASR): In a laboratory scale the UASR (Up-flow anaerobic stage reactor) system comprise four identical cylindrical compartments (stages) linked in series. Each stage of the reactor has a 3-phase separator baffle with an angle of 45^0 and placed 50 mm below the effluent ports in order to prevent floating granules from washing out with the effluent. The walls of the reactors are wrapped with a tubular PVC water-jacket to maintain the reactor temperature at 37^0 C. In general, this study is carried out in four major steps: 1) start-up of UASR, 2) acclimatization to pharmaceutical wastewater, 3) increase in OLR (organic loading rate) by altering feed COD at constant HRT (hydraulic retention time) and 4) increase in OLR by reducing HRT at constant feed COD. Supernatant liquor, gas and sludge samples are taken separately from each stage for analysis (Chelliapan and Sallis., 2011).

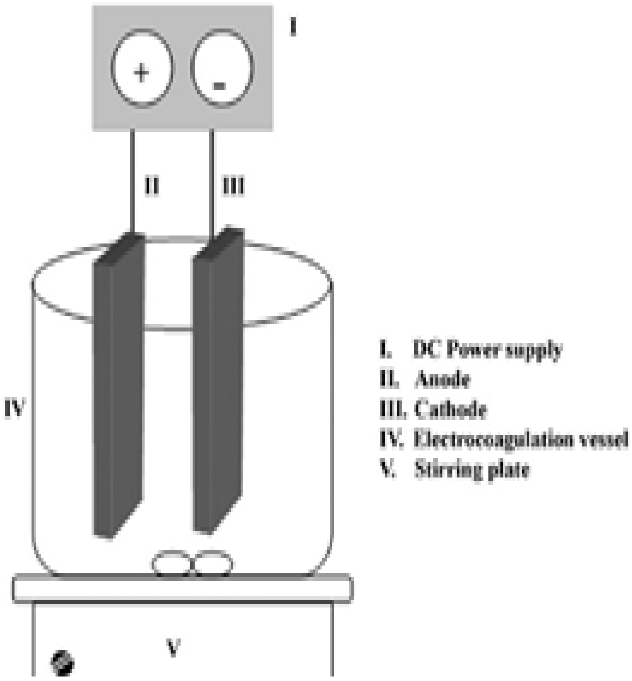
c. Upflow anaerobic sludge blanket (UASB) reactor: UASB reactor is a popular anaerobic reactor for both high and low temperature and among the recently developed high-rate processes (Lettinga *et al*, 1980). In a laboratory study treatment is carried at room temperature in glass UASB reactor having volume 3.5 liters. The UASB reactor is seeded with 500 mL filtrate of sludge of biogas plant, 500 mL of pharmaceutical effluent and the remaining volume is filled with distilled water and kept in same condition for 2 weeks after which 500 mL sample of effluent is collected from the sampling port and same volume is replaced by pharmaceutical effluent on weekly basis till steady state condition i.e. maximum COD removal is observed. The reactor is then run in continuous mode during which the reactor is operated in three phases. In phase I and II, the reactor is fed 10% of pharmaceutical waste water at HRT of 24 hours and 20 hours respectively and 50% of waste water is fed to reactor at HRT of 20 hours (Lohchab and Snehlata, 2013)

Aerobic treatment in a biological reactor

A batch type biological wastewater treatment process is carried out in a cylindrical open tank, which acts as an integrated bioreactor and air is passed through the bottom of the tank. To provide the mixing and ensure optimal contact between wastewater and biomass, reactor is designed to inject air through its bottom. Air is passed through the wastewater in the form of course bubble and the entire process is carried out at room temperature (30 ± 1^0 C). Biodegradable bacteria are used, consisting of maximum aerobic and minimum facultative bacteria. The samples are collected and analysed everyday using standard methods (APHA, 1998) in order to monitor the biodegradation process taking place inside the reactors (Khan and Mostafa., 2011).

Electro-Coagulation and Natural Coagulation process

Coagulation is a traditional physicochemical treatment of phase separation to precipitate colloidal and ionic species by adding coagulating agents such as Fe^{3+} or Al^{3+} ions in the form of chloride salts. By electro-coagulation similar effects can be achieved (Chen, 2004), where a current is applied to dissolve Fe (or steel) or Al anodes immersed in the polluted water to release the corresponding metal ions yielding different Fe(II) and/or Fe(III)) or Al(III) hydroxide species depending on the pH of the medium. These ions act as coagulants or destabilizing agents that neutralize charges and separate colloids and ionic products from the wastewater by sedimentation, producing some sludge. The coagulated particles are also separated by electro-flotation when they are attached to the bubbles of H_2 gas evolved at the cathode, being transported to the solution surface where they can be withdrawn.



[Fig. 1] The Electro-Coagulation Apparatus (Dixit and Parmar., 2013)

The setup consists of (1) Pulse generator; (2) reaction tank; (3) pump; (4) electrodes. A Pulse current is supplied and the cell voltages are recorded. After that the COD and the concentrations of BH of the wastewater is determined. Electrodes are connected to a dc power supplier (Dixit and Parmar., 2013). The electro-coagulation experiments by Pablo Canizares et al (2009) have been carried out in a bench-scale plant with a single compartment electrochemical flow cell, Aluminum electrodes were used as the anode and cathode. Both electrodes were square in shape (100 mm side) with a geometric area of 100 cm^2 each and with an electrode gap of 9 mm.

EFFICIENCIES OF TREATMENT

Packed Bed Reactor

The pH levels in the effluent of the anaerobic packed bed reactor are generally stable (pH 7.5 – 8.2) during the operational period. In experiments carried out by Chelliapan *et al* (2011) at a reactor OLR of 1.58 kg COD.$m^{-3}d^{-1}$ (HRT 5.6 d), the average soluble COD reduction was around 73%. However, when the OLR was increased to 2.21 and 4.66 kg COD $m^{-3}d^{-1}$ the COD removal efficiency decreased gradually until 60 - 70% soluble COD removal was observed. A low concentration of total VFA (average 350 mg L^{-1}) was present in the reactor effluent when operated at OLR in the range 0.50 to 1.58 kg COD $m^{-3}d^{-1}$. However, the VFA (volatile fatty acid) concentration was increased to 1000 mg L^{-1} when the reactor OLR was increased to 4.66 kg COD.m^{-3}.d^{-1}.Further increase in reactor OLR, by reducing the HRT results in higher VFA concentrations being produced in the effluent. The highest of these was found when OLR was 5.71 kg COD $m^{-3}d^{-1}$ with an average value of 1200 $mg.L^{-1}$.

Up-flow anaerobic stage reactor (UASR)

In a study it was found that at a reactor OLR of 1.86 kg COD $m^{-3}d^{-1}$ (HRT 4 d), the soluble COD reduction was around 70 - 75%.(Chelliapan and Sallis., 2011) Over 90% COD removal was observed at OLR of more than 10 kg COD m^{-3} (Uyanik *et al*., 2002).In an experiment it was found that the COD removal efficiency was 90% at an OLR of 1.5 kg COD $m^{-3}d^{-1}$ and HRT 11 d (Rodriguez-Martinez *et al*., 2005). It is reported from an experiment that when HRT of an ABR treating pharmaceutical wastewater containing antibiotics was extended from 1.25 to 2.5 d, the COD removal efficiency increased from 77 to 85% (Zhou *et al*., 2006).

Upflow anaerobic sludge blanket (UASB) reactor

In an experiment it was found that the steady state conditions i.e. 80% COD reduction was reached after 10th week of batch study. During continuous mode 97%, 93% and 89% COD reduction was achieved in phase I, II and III, respectively. In phase-I, II and III, pH varied between, 5.96 to 8.09, 6.87 to 8.00 and 5.09 to 7.01 respectively. Alkalinity of phase I, II and III varied from 700 mg/l to 200mg/l (71% reduction), 1200mg/l to 500mg/l (58% reduction) and 7500mg/l to 4700mg/l (37% reduction) respectively. Higher alkalinity in phase III was due to increase in OLR (Mohan *et al*., 2005). VFA showed variation from 825.67mg/l to 270.91mg/l (67% reduction) in phase I, 996.625mg/l to 450.987mg/l (55% reduction) in phase II and 730.87mg/l to 420.8mg/l (42% reduction) in phase III. The variation in phosphate concentration in phase I, II and III varied from 33mg/l to 10.1mg/l (69% reduction), 29.60mg/l to 16.01mg/l (45% reduction), and 78.29mg/l to 45.01mg/l (42% reduction) respectively. The VSS/TSS ratio is important in determining the sludge characteristics and reflects biomass growth and its quality. When VSS/TSS ratio was in between 0.78 to 2, the COD reduction was 97%, when it was 0.80 to 1.17, COD reduction was 93% and when it was in between 0.85 to 0.95 then COD reduction was 89%, It indicates that higher the VSS/TSS ratio higher will be the COD removal (Lohchab and Kumar, 2010; and Lohchab and Snehlata; 2013).

Aerobic treatment in a biological reactor

In a laboratory experiment it was found that TDS continued to increase with time and TDS concentration of 600 mg/L was obtained on 15 days of retention time. It was observed that liquid was vaporized from the bioreactor and this vaporization of liquid was the main cause of TDS increased. But in a continuous process, TDS would not be increased with time. In effect of aeration EC continued to increase with time and EC value was found to be 1200μs/mol on 15 days of retention time. Generally EC increases when TDS increase. Here EC gradually increased, as a consequence of TDS increased. It was seen that TSS (Total suspended solid) increases with time. In a study TSS was found to increase from 280 mg/L to 380 and 480 on day 3 and 5. About 30 and 50% TSS was increased on day 3 and 12. The maximum TSS will be removed after settling, and thus an improved water quality will be achieved. With the interval of time COD (chemical oxygen demand) decreased, which decreased rapidly between fourth and ninth day. In a study it was found that in optimum conditions COD was reduced to over 60% in 9 days and 75% in 15 days of retention time.

Electro-Coagulation and Natural Coagulation process

It is seen that the removal efficiencies of COD and BH are significantly higher by using Fe electrode than by using Al electrode (Dixit and Parmar., 2013). With Fe electrode, the removal efficiency reaches above 80% within 3.5 h, which is about twice as high as with Al electrode; the dissolution current efficiency is far higher for Al than for Fe electrode (S. Zodi *et al*., 2009). Electrode distance plays a significant role in the process. The removal rates of COD and BH reaches 68.0% and 72.0% separately at the optimum electrode distance of 2.0 cm. Short electrode distances inhibits the ion diffusion between electrodes, and reduces the coagulation efficiency, the electric resistance and voltage increases with the increase of the electrode distance (Dixit and Parmar., 2013)

CONCLUSION

Being a leading industrial sector of the country wastewater from pharmaceutical industries is a big concern to the environment of Bangladesh nowadays. However, proper treatment can eliminate the hazard of pharmaceutical waste water to the environment. Different pharmaceutical wastewater treatment methods are used worldwide. Some of those methods are discussed in above sections which can be applied in Bangladesh perspective. From above discussion it is seen that, in anaerobic treatment the efficiency of COD removal can be 60-97% depending upon various factors and reactor used. In packed bed reactor this range is 60-70% depending upon OLR and HRT. In USAR 70-90% of COD removal can be achieved depending on OLR and HRT whereas in USAB reactor up to 97% COD removal can be achieved. In aerobic treatment 60-75% COD removal can be achieved depending upon retention time. In electro-coagulation COD removal of up to 80% can be achieved depending upon the constituent of electrode and electrode distance. So these treatment methods are quite effective for treating pharmaceutical wastewater and can be a good option for pharmaceutical industries of Bangladesh.

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A STUDY ON SOLID WASTE MANAGEMENT OF DHAKA CITY: AN EMPHASIS ON REDUCTION, RECYCLE & REUSE OF WASTE

Rafiur Rahman Khan Taim[1] & Rafat Tanvir Ahmed Shuvo[1]

[1] *Department of Civil Engineering & Environmental Science, IUT-OIC, Gazipur 1700, Bangladesh.*
**Corresponding Author, <taim21@yahoo.com>*

ABSTRACT

This study has analysed the characteristics & generation of solid waste in Dhaka city, along with the environmental impacts and existing solid waste management practices prior to reduce, reuse & recycle. An estimate of the future generation rate indicates that the present generation rate of 3500 tons/day may exceed 30 thousand tons/day by the year 2020 which indicates an alarming threat. Organic waste disposal, improper collection of solid waste & lack of space of landfilling sites are the main reasons behind this problem. Solid waste data of an Etp was collected and different aspects were analyzed for reduction purpose. A survey area was selected for building composting toilet and specific materials were selected for the recycle & reuse purpose. From the analysis, it has revealed that present forms & techniques of managing solid waste are not adequate & insufficient because,

the sludge are discharged from the etp without any treatment just in solid cake form by the filter press machine .Therefore, adequate steps should be taken for efficient reduction, re-use and re-cycle of solid waste.

Keywords: Etp, Solid waste, Composting.

INTRODUCTION

Solid waste management has become a burning question especially for a developing country like Bangladesh. .Rapid growth of population, life style improvement , improper management of solid wastes and urbanization are the main reasons behind the problem. According to the adjusted population of the 2001 Census; the size of Dhaka's population is 10,712,206 of which 5978482 are male and 4733724 are female. This makes Dhaka a megacity.

The growth of the urban population in Bangladesh since 1901 is depicted through the following periods. In 1901 only 2.43% of the country's population lived in urban centres. During the next two decades the urban population remained almost static. During the 1951-61 decade there was a 45.11% increase in the urban population, more than twice the previous decade's 18.4%. The most phenomenal urban population growth in Bangladesh occurred during the 1961-74 inter-census period .The growth rate of the urban population was 5.4% during the 1981-1991.The total urban population increased to 28.6 million by 2001.

In recent decades, Dhaka has been experiencing an influx of people from across the nation, making it one of the fastest growing metropolitan areas in the world. The current population of Dhaka is 14,399,000 approximately with density of 45,000/km2 (115,200/sq mi). This increasing people leads to discharge of more solid waste creating several diseases. Estimates of solid waste generation vary widely ranging from 1040 tons/day (1985-86) to 5000 tons/day (in 1997)[1]. According to the predictions of BCAS (1998) the generation of solid waste would be around 8,478 tons/day by the year 2020 where as the second estimate predicts that the waste generation will reach over 30,195 tons/day by 2020. [2]

Certain chemicals if released untreated, e.g. cyanides, mercury, and polychlorinated biphenyls are highly toxic and exposure can lead to disease or death. Industrial solid wastes are disposed from coal and mineral industries, mining industries, metal industries, engineering industries and thermal power stations.

Biomedical waste includes anatomical waste, syringes, gauze, absorbents, glass. Serious health problems are caused by such disposals. Rotting of it in front of clinics, street corners is not uncommon due to which rag pickers, mostly children are attacked by the hepatitis virus or jaundice-causing virus. The dumping of solid wastes spoils the beauty of cities and towns, causing health problems.[3]

The "Waste Management Hierarchy" (minimization, recovery and transformation, and disposal) has been adopted by most industrialized nations as the menu for developing solid waste management strategies. As a result, reduction, recycle & re-use of solid wastes has become a crucial issue.[4,5,6[

The objectives of the study is to give an emphasis on the reduction, re-use & re-cycle of solid waste with a view to minimizing the adverse effects on human life & environment.[,7,8]

MATERIALS AND METHODS:

Data collection: A retrospective study involves the collection of information from [JICA] the agency responsible for the management of solid waste in Dhaka & Envoy textiles Etp situated at Mymensingh valuka sub-distrcit.

Other sources of information include personal observations, interviews. with staff of the agency. Relevant information was also sourced from reports, books and journals. Field surveys were carried out in some areas and the existing official dumpsite on the various samples of waste generated.

1.Etp Data:

Following data were collected based on several tests:

Etp name	Envoy textiles Etp Mymensing,Valuka
Type of Etp	Physio-chemical followed by bio-logical
Waste water type	Indigo dying Waste water
Etp capacity	200 m^3/hr
Amounts of solids	8000- 10000/day
BOD	450-500 mg/l
COD	4000-4500 mg/l
TDS	2800-3000mg/l
TSS	450-500 mg/l
Waste water colour	3000-3500 Pt-Co Unit
Waste water Ph	10-12.5

Waste water Temperature	33-35 degree
Solid-sluge disposal rate [daily]	8000kgs
Solid-sluge disposal rate [monthly]	240000kgs
Solid-sluge disposal rate [yearly]	2880000kgs
Overall cost	20 taka / m^3

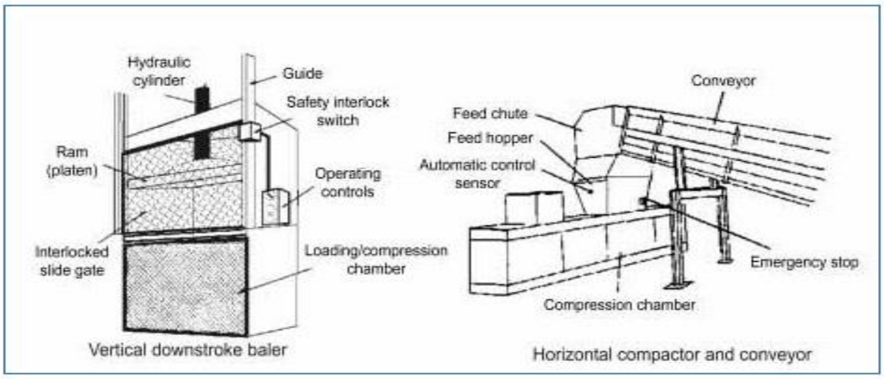

Figure: Solid compressing machine

Treatment Procedure: After treating of waste water, sludge or solids are produced and pressed by filter into solid cake forms. Then sludge are released into the nature without any further treatment.

2 **Composting Toilet Data collection:**

A composting toilet can be built according to ‘’Clean Up America Company in several villages in Bangladesh”

Following data were collected during construction of double pit composting Toilet in Atipara

Village in Barisal:

Slab length	1.2 m
Slab width	.9m
Mixtrues	10 litre cement with 50 litre river sand
Reinforcing wire thickness	3-4 mm
Vent pipe length	110 mm
Concrete beam external dimension:	1.3 * 1m
Size of hole	1 * .7 m
Minimum construction time	12-14 days

Construction Procedure:

1 .It is made by mixing 10 litres of cement with 50 litres clean river sand.

2. Half of the mix is added to brick mould first.8 reinforcing were laid within the mould.

3. Then second half of the mix is added & smoothed down with a wooden float .Finally it is finished with a steel float for 7-10 days.

4. Then rectangular concrete beam is produced :

5. Half the mix is added first. Wire reinforcing is used within the concrete mix with two strands of 3

- 4mm wire down each length, making a total of 8 pieces. The total length of wire required is approximately 9 metres.

6. Then the second half of the mix is added and smoothed down with a wooden float. The beam is covered and left to sure for at least 7 days.

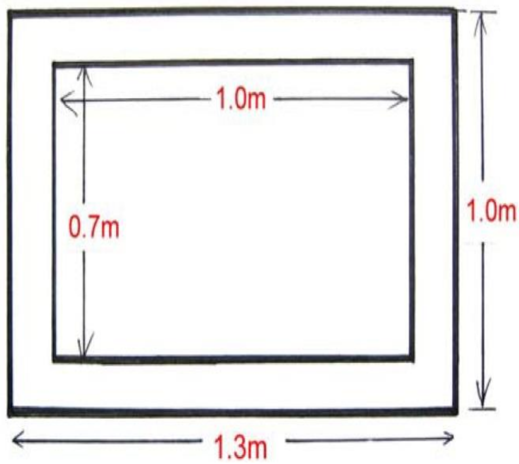

Figure:Rectangular Beam

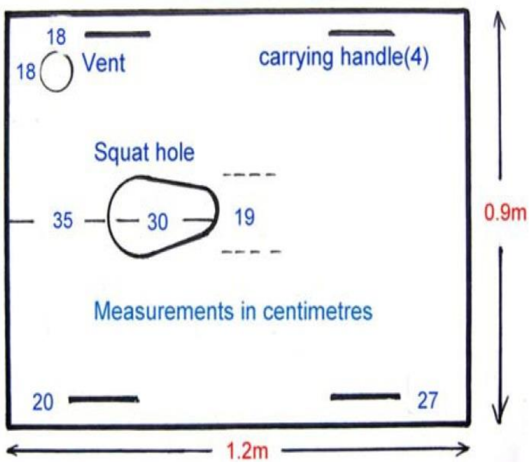

Figure: Concrete Slab

8.When constructing the double pit composting toilet, the two ring beams canbe cast on the actual toilet site directly on the ground, at least 0.5 metres apart.

9.A level piece of ground, preferably on a slightly elevated site, is best. Alternatively the two ring beams can be cast away from the toilet site and moved on to the site after curing. In this case a plastic sheet should be laid on the ground on which the ring beams can be made.[,9,10]

RESULTS AND DISCUSSION:

From the Etp data we found that :Solids are not treated in any manner. They are just released in the Cake form.
Thus more funding is needed for the additional treatment of solids along with water in a Etp of Dhaka. By constructing the Composting toilet we would have following benefits in Dhaka:

Water Use Reduction (20-50%): A significant savings in water storage will result if the household is not on reticulated water supply. Combine this with wastewater re-utilisation in irrigation and other household water reduction techniques and water storage costs can be cut by up to 60%..

Shock Loading Capacity : Loading shock for large gatherings is achieved easily with correctly sized composting toilet systems.

Odour Problems Reduced: The suction air flow in most composting toilets takes toilet and bathroom odor out of the room and acts like a constant extraction fan.

Lower Household Maintenance Costs: Sewage rates and water rates (metered) can be in the order of $500 per year, a significant cost. This will only increase if the demand for sewage system upgrading increases. Other on-site systems have annual maintenance costs that are obligatory. Local authorities will be increasingly paying rebates to households who own composting toilets.

End Product Recycled: While only small in amount, the solid end product is a valuable humic fertiliser that can be utilised around trees and gardens.

Reduced Greywater Loading :

Where composting toilets are installed instead of septic and mini-treatment systems, there is a large reduction in the "loading" on the effluent treatment system by the removal of "blackwater." Smaller, less maintenance, grey water systems are possible.

Again following steps should be taken for re-use of solid waste:

1.Curbside collection: Requires common people to separate re-cycles from their garbage.

Clean recyclables need to place in special container while the garbage in standard container.
Both should be placed at the curb for collection for collection by separate trucks.
2.Re-cycling organic waste: The main forms of organic wastes are household food waste, agricultural waste , human & animal excreta.

'Composting' is the process through which organic waste can be converted to a rich, dark colored compost or humus in a matter of a few weeks & months.

Composting provides a useful way of reclaiming nutrients from organic refuse.
Can be used as fertilizer for improving soil condition.
3.Using High Density Poly –Ethylene (HDPE) plastics :HDPE plastics can be used for recycling bottle, toys pipes ,crates from the soft drink can .
4.Reduced packaging: Package box or corrugated boxes from the customers can be reused to make the package box .
5.Using scrubbers in the industry: Industry should include scrubber system to scrub out pollutant before releasing wastes.

CONCLUSION:

As it is difficult to get land for recycling near the community, entrepreneurs should be encouraged and enabled to get lease of government land.

1. Recycling has both tangible and intangible benefits, the intangible benefits should be quantified or clearly presented to help the municipality understand the importance of recycling

2. Compost should be supplied free of charge initially to community to encourage their interest in participation

3.4Source-segregated waste is essential for good quality compost, so households should be

motivated accordingly.

4. The importance of organic compost should be well demonstrated to farmers by the Ministry of Agriculture.

5.Governmental funding is necessary for the treatment of sludge in the treatment plant.

6.Every one should encouraged to build & use composting toilet.

Above all co –operation from every one in the Dhaka city is needed along with the mentality to give importance to the factors of reduction, reuse & recycle of solid waste.

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ID: EE 062

POND SAND FILTER: AN ALTERNATE WAY OF SAFE DRINKING WATER

Mostafa Zaman[1*], Liakot Hossain[1] & Swapan Kumar Palit[1]

[1] *Department of Civil Engineering, CUET, Chittagong-4349, Bangladesh.*
**Corresponding Author, <mzshakil.cuet@gmail.com>*

ABSTRACT

Bangladesh, a flood plain delta, is a land of rivers and canals. The country is sloping gently from the north to the south, meeting the Bay of Bengal at the southern coastal end. As drinking water sources, people in coastal areas use shallow tube-well, deep tube-well, Pond Sand Filter (PSF) and rain water harvesting. PSF is popular option of potable water in coastal areas, though modified PSF has not been implemented in this area. Due to high concentration of iron and salinity in tube-well water of coastal area, people prefer PSF which treats pond water.

A survey data have been collected for the water quality parameters of PSF water from different parts of the Bangladesh. And it has been observed that most of the PSF are not performing well due to lack of teechnical knowledge and awareness and hence treated water of some PSF is slightly contaminated. But this contamination can be minimised if proper management and maintainance on the operation system of PSF and pond as well, be assured. In the present investigation, water samples have been collected from the various sources of the selected coastal area for evaluating drinking water parameters in the laboratory. Finally the result have been compared with Bangladesh standared. The laboratory result shows that no natural source of water in coastal region can be used for drinking purposes except perched acquifer if available. So a sustainable alternative way of getting safe drinking water in the coastal area will be a POND SAND FILTER.

Keywords: Potable water, Pond Sand Filter, Water source, Natural disaster.

INTRODUCTION

Bangladesh is a south Asian country with an area of 143,998 sq km. Bangladesh borders India on three sides north, east and west. The population of Bangladesh is about 100 million, of which 20 million live in coastal districts. The density of population in the coastal districts is higher than that of the rest of the country. The density of population has increased as much as four times during the century. The high rate of population growth pushes millions of people to live in the low lying coastal areas which are highly vulnerable to various types of environmental hazards. And thus it is essential to find out the alternative sources of pure drinking water.

An alternative and popular option of portable water supply in coastal problem area is the pond sand filter (PSF). It is a package type slow sand filter unit developed to treat surface waters, usually low-saline pond water, for domestic water supply in the coastal areas. Pond sand filers are installed near or of a bank on the pond, which does not dry up in the dry season. The water from the pond is pumped by a manually operated hand tube well to feed the filter bed, which is raised from the ground, and treated water is collected through taps. It has been tested and found that the treated water from a PSF is normally bacteriologically safe or within tolerable limits. On average the operating period of a PSF between cleaning is usually two months, after which the sand in the bed needs to be cleaned and replaced.

NATURAL DISASTER OCCURRED IN BANGLADESH

Natural disaster is a common matter in our country, its lik a kith and kin to the people, every year many types of disaster visits in our country .A large number of people have been died and a huge amount of people are affected in different calamities, some records are in bellow

Affected People

Disaster	Date	Affected (no. of people)
Flood	1988	45,000,000
Flood	2004	36,000,000
Flood	1984	30,000,000
Flood	1987	29,700,000
Drought	1983	20,000,000
Storm	1991	15,438,849
Flood	1998	15,000,050
Flood	2007	13,771,380
Flood	1995	12,656,006
Flood	1993	11,469,537

Killed People

Disaster	Date	Killed (no. of people)
Storm	1991	138,866
Storm	1985	15,000
Storm	2007	4,234
Epidemic	1982	2,696
Flood	1988	2,379
Flood	1987	2,055
Epidemic	1991	1,700
Flood	1984	1,200
Flood	2007	1,110
Flood	1998	1,050

STUDY AREA

Bangladesh is located at the head of the Bay of Bengal, and the country has a coastline of approximately 710 km. The Coastal Zone Policy considers three indicators for determining the landward boundaries of the coastal zone of Bangladesh, which are: influence of tidal waters, salinity intrusion and cyclones/storm surges (Ministry of Water Resources, 2005). The policy regards the exclusive economic zone as the seaward coastal zone. Bangladesh has an area of 147,570 sq.km. with one-third of the country belonging to the coastal zone (Ministry of Water Resources, 2005).

General situation

Location: Eastern coastal zone

Annual average rainfall: more than 3000mm

Ground water: very saline (almost can’t be drunk)

Surface water: sweet water but contained biological micro-organism.

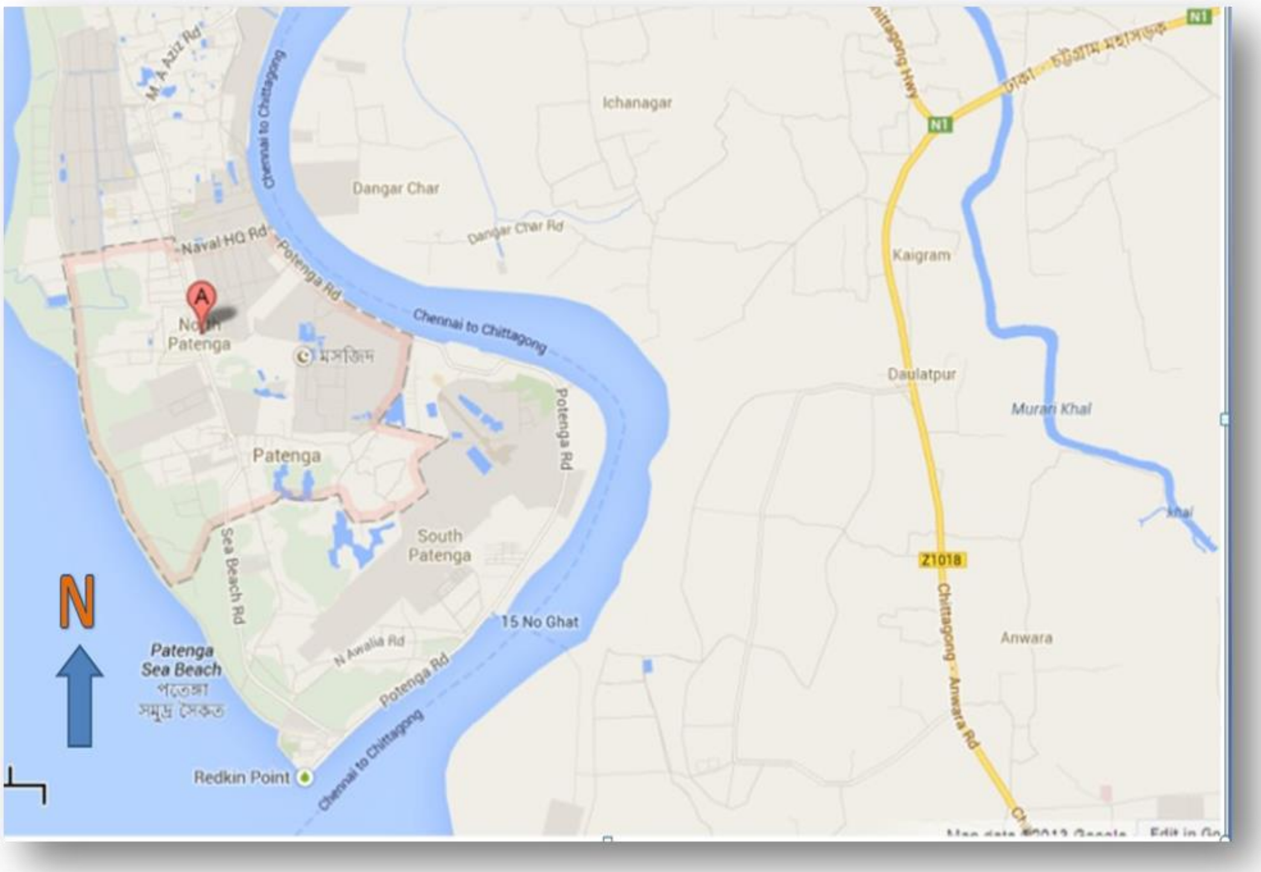


Fig: Location of north potenga

PHYSICAL CONDITION OF PONDS

There are six ponds, three ponds are owned by individuals and rest threes are joint owner ship. Among the studied six ponds, three ponds are badly managed. Often garbage, wastes are disposed in the pond also direct run-off enters into the pond i.e. banks are not protected. However other three ponds are managed nicely; banks are protected so that run-off cannot enter into the pond. Also bathing is prohibited in these ponds.

Sample test and Graphical representation

Various sources of water are collected from the study area. The collected water are evaluated in the environment laboratory of civil engineering, CUET. Table shown various laboratory test result and figure shows graphical representation of the test result and it was compared with Bangladesh standards

Water quality parameters	Location: Potenga					BD standards
	Tubewel(1)	Tubewell(2)	Small pond	Big pond	sea	
pH	5.69	5.67	5.19	5.13	6.5	6.5-8.5
color(PCU)	80	70	75	60	2.5	15
Turbidity(NTU)	1.33	3.79	52.3	67.8	27	10
Alkalinity(mg/l)	525	840	520	668	470	200-500
CO2	67	85	25	27	134	----
Chloride(mg/l)	4049	4512	250	265	4655	600
Iron	8.5	11.5	.3	.3	13	1
TC	---	---	----	---	---	0
FC	---	---	---	----	----	0

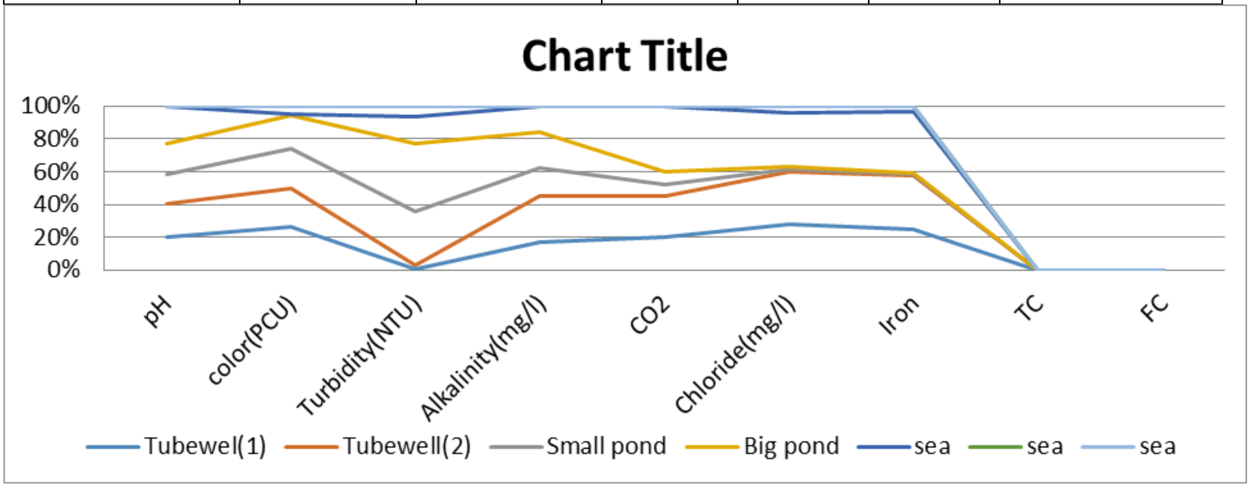
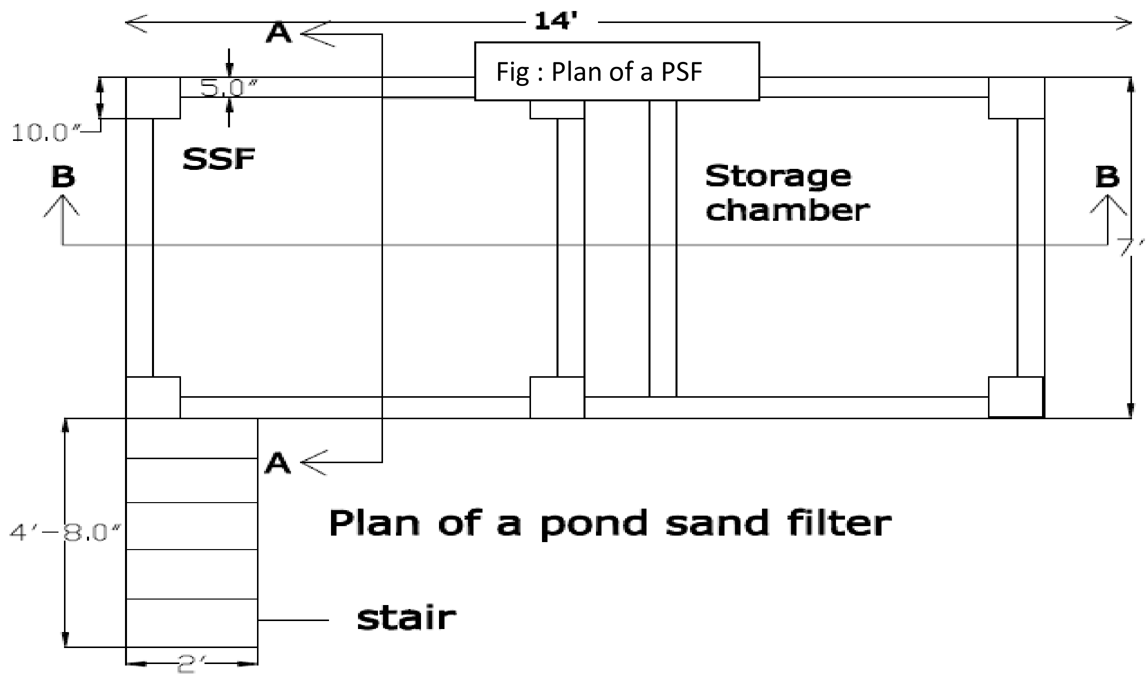
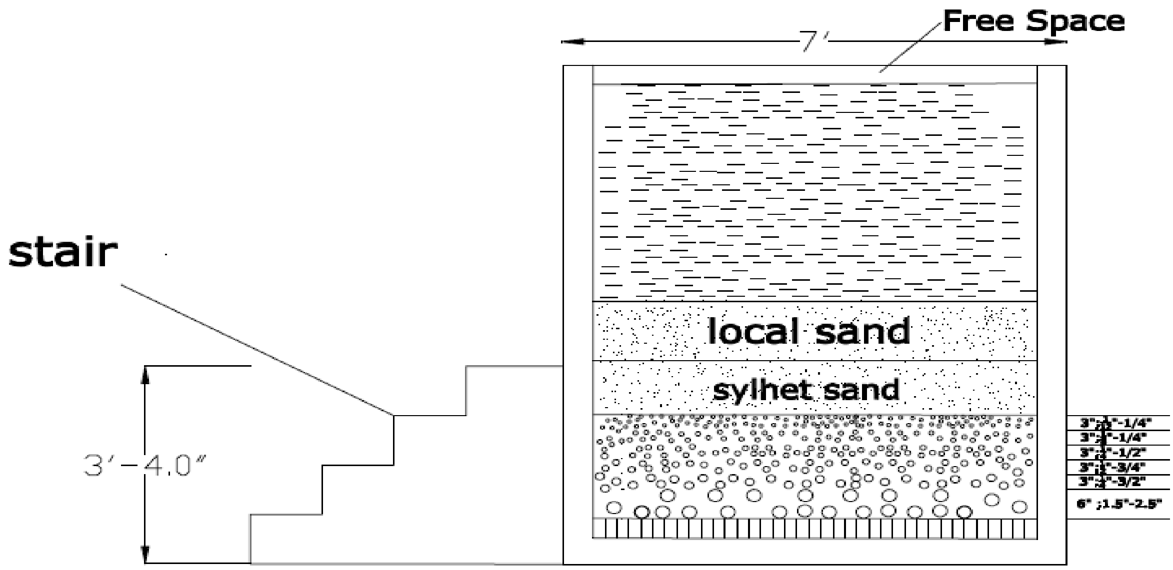


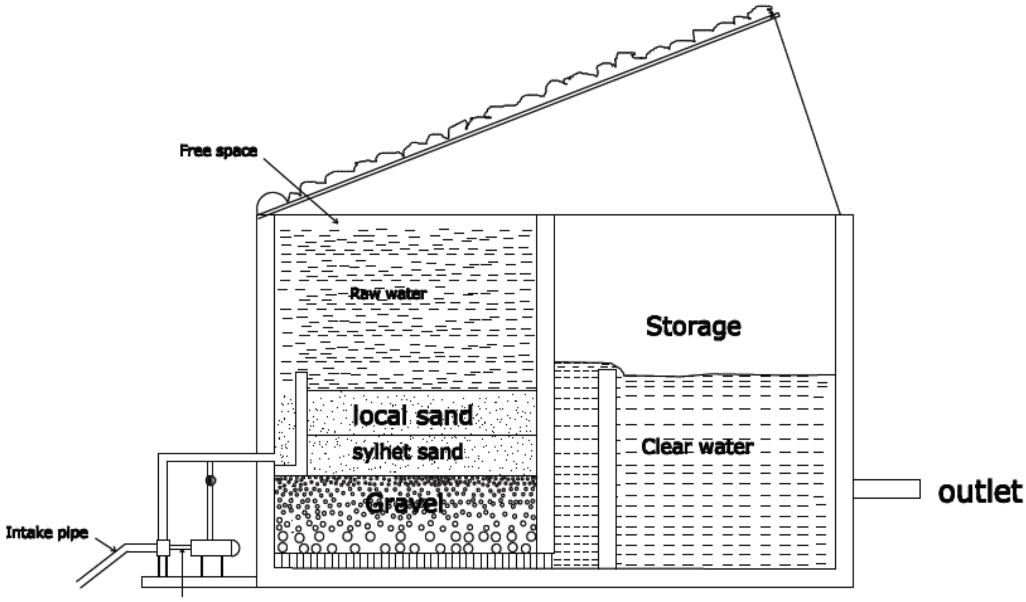
Fig : Graphical representation of different sample

DESIGN PROCEDURE & CALCULATION





Section of A-A



Section of B-B

Estimation

Total amount of cement = 38.40 cft = 40 bag

Total amount of local sand =167.8cft

Total amount f sylhet sand = 49 cft

Total amount of bricks (including khoa) =1072+2100 = 3172 no of bricks

Total length of #4 steel bar = 255’ + 112’ +112’ + 96’ = 575’

Length of #3 steel bar = 195’+ 80’ + 84’ = 356’

Nominal weight of #4 steel bar = 575’*.668 = 384lb

Nominal weight of #3 steel bar = 356’*.376 = 133.56lb

Total weight of steel = 384+133 = 517lb = 235k

CONCLUSION

From literature review it can be said that recently government and some NGO’s working together for coastal areas peoples. But it is not sufficient. More activity should be taken in future.

In coastal areas many people’s are living together. But there facilities are confined . during any type of disaster safe drinking water is not available due to high salinity of ground water.

Generally the people in coastal areas use tube-well water for domestic purpose and pond water by heating in drinking purpose.

Rainwater is safe for drinking purpose for the coastal areas people, but it is time dependent.

Pond sand filter is the best possible way that can be used in surface water treatment process.

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SOLID WASTE MANAGEMENT SYSTEM: A STUDY ON COX-BAZAR POURASHAVA

K. M. Rahman[1], N. A. Hashi[1*] & A. Habib[1]

[1] *Department of Urban and Regional Planning, Chittagong University of Engineering & Technology, Chittagong- 4349, Bangladesh.*
** Corresponding Author, <nhashi.cuet@yahoo.com>*

ABSTRACT

Solid waste management becomes a great problem now-a-days. Rapidly growing population is responsible for producing large amount of solid wastes. To develop an efficient solid waste management system the existing scenario of solid waste of a city should be known which enables to find out the problems happening regarding mismanagement of solid waste within the city. Cox's Bazar is a well-known tourist spot both in home & abroad. As a tourist spot different types of solid wastes produced everyday by the people living in the city as well as the people coming to visit it. Low collection coverage, unavailable transport services, and lack of suitable treatment, recycling and disposal facilities are responsible for unsatisfactory waste management, leading to water, land and air pollution, and for putting people and the environment at risk. In such a circumstance this study has intended to unfold the institutional dimensions of solid waste management including the types & amount of wastes produced in this city and to identify solid waste management problems. A field survey has conducted to identify the current scenario of solid waste management system in Cox's Bazar. It helps to identify the problems regarding solid waste management system which helps in this study to give some recommendations to improve the solid waste management system of Cox's Bazar.

Keywords: Solid waste management, Cox's Bazar Pourashava (CP), Institutional dimension, integration, community development

INTRODUCTION

Background of the Study

The growth in municipal solid waste generation the world over as a consequence of urbanization, industrialization, and population growth, together with improved living standards has been widely reported (Rao et al., 2007). Municipal solid waste has also been recognized as one of the major problems confronting governments and city planners the world over (Rahman, 2000). It is estimated that Bangladesh averagely generates 13,332.89 ton waste per day including average per capita urban waste generation rate is estimated as 0.41 kg/capita/day. Solid waste management (SWM) today is considered to be one of the most immediate and serious problems of environment, confronting urban local government in developing countries. Cox's Bazar is one of the most-visited tourist spot in Bangladesh. The significance of the city is increasing day by day for its tourism, which help to the overall development of Bangladesh. The growth in population, urbanization, industrialization, and waste generation in the developing countries calls for proper solid waste management as it has become a necessity for environmental conservation (Rao et al., 2007) and Sustainability. For a sustainable solid waste management system policies and techniques such as waste recycling, reuse, waste reduction, thermal treatment, land filling etc. must be in place. As Cox's Bazar is one of the main tourist spot of Bangladesh, it is necessary to take actions to make a sustainable solid waste management system for Cox's Bazar. The findings suggest that the solid waste hazard is mainly a function of Socio-economic vulnerability that should be overcome through taking some initiatives. As such, there is a need for integration of solid waste management activities with community development. It has also identified that the planning and implementation of community development and solid waste management activities should be done at the grassroots level with the provision for direct participation of the people concerned.

Goals and Objectives

Waste generation is not a natural phenomenon rather generating solid waste depends mostly on human activities. Huge generated wastes has become a struggling matter to manage. The total environment of the study area is degraded due to ill or unscientific management of solid waste. Cox's Bazar is the main tourist spot of Bangladesh and many foreign tourists come to visit the city, it is, therefore, highly desirable to study solid waste management system of Cox's Bazar for better planning and management of solid wastes. The main objectives of the study are to view the types and current amount of wastes generated in the city and to identify the existing solid waste management problems of Cox's Bazar Pourashava (CP).

STUDY AREA

Cox's Bazar Pourashava has taken as the study area which covers an area of 32.72 sq. km with 27 mahallas and 9 wards. The Pourashava falls under the category of 'A' class. The Pourashava is located at 21°35′0″N 92°01′0″E and bounded by Bakkhali River on the north and East, Bay of Bengal in the West, and Jhilwanj Union in the south. The existing landfill site is at Kasturighat and the proposed is at Khurushkul Union near airport area and sea beach which shown in fig. 1

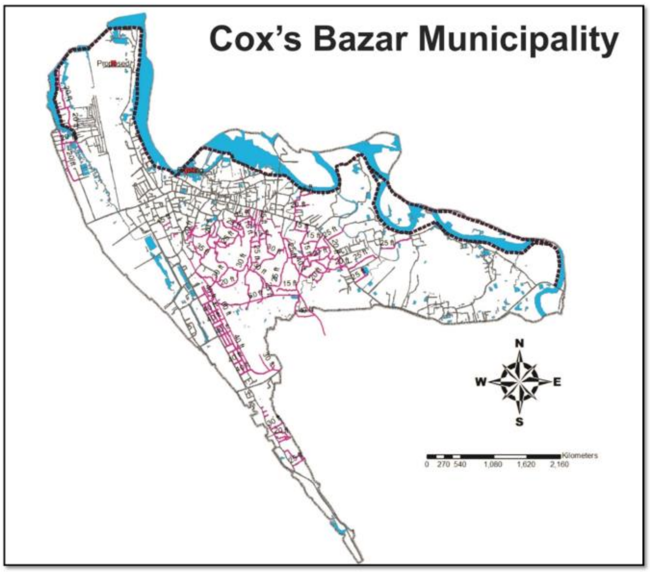

Fig. 1. Study area map

METHODOLOGY

A proper methodology is always necessary for any research, which helps to organize experiences, observations, examinations, analysis of data and information and their logical expression in a systematic process to achieve the ultimate goals and objectives of the research. The methodology adapted in this study is a combination of empirical and case study method. To conduct this research, the study followed sampling and observation techniques. At first, to make the clear concept about the frame work of the study and get the overall information about the study area, a reconnaissance survey was conducted. The questionnaire was pre-tested in the field and necessary modification was done before finalizing. Valuable primary information regarding the selected case is gathered from key informants, executives & beneficiaries. Besides this direct observation field visit was done to get a clear picture of various aspects of the study relevant to social management. It helps to observe the actual situation and to know people's opinion about the solid waste management. Most of the people in that community have found cooperative in answering the question. The description and unstructured materials from interviews had been arranged in a sequential order. The collected data both from primary and secondary sources were analyzed properly using standard statistical tools to explore the hidden dimensions behind the findings.

EXISTING SOLID WASTE MANAGEMENT SCENARIO

Organogram of Conservancy Department

The organogram of Cox's Bazar Pourashava is shown in fig. 2

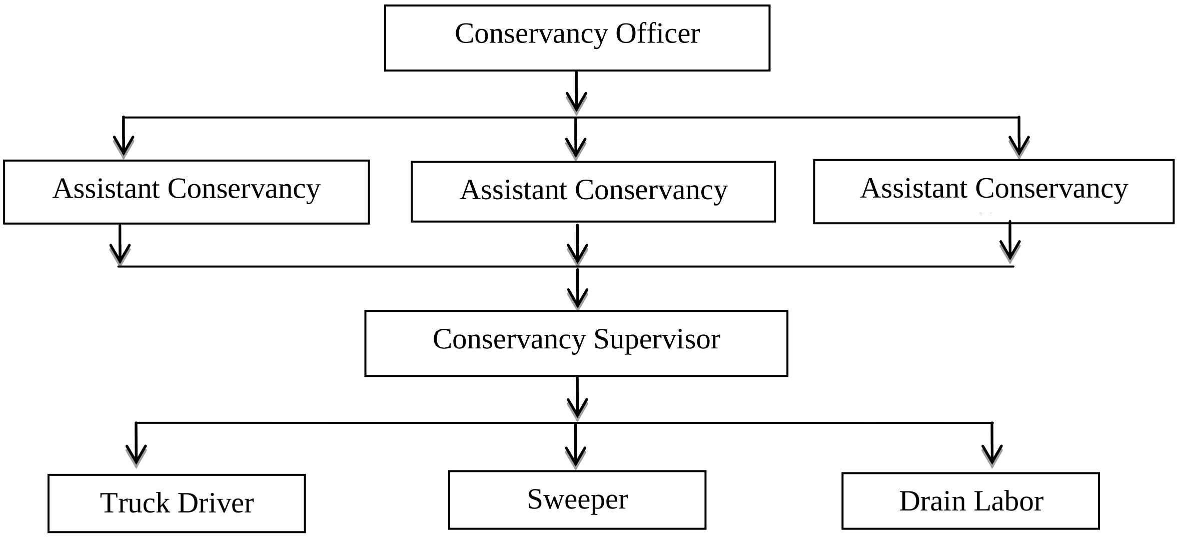

Fig. 2: Organogram of Conservancy Department of Cox's Bazar Pourashava

Manpower of Conservancy Department

At present there are 157 workers and about 4 officials are engaged by conservancy department (CP) for collection and disposal of refuse. The conservancy department has not enough manpower for street sweeping, drain cleaning, garbage collection and disposal. Still the ratio of workers are about 1:372 Considering the population of 51918(BBS, 2001) which is low as compared to the minimum requirement of 4 - 5 workers per thousand populations for satisfactory manual collection and disposal of garbage (Alamgir, 2007).

Financial Aspects of Cox's Bazar Pourashava

The Revenue and accounts sections look after the financial matters of CP. Revenue section is responsible for the assessment and collection of local taxes, charges, rents and license fees is headed by Cheap Revenue officer and the accounts section responsible for budgeting, accounting and payments is headed by Chief Accounts Officer. The responsibility for overall financial management of CP shared function between the Chief Revenue Officer, Chief Accounts Officer and Audit sections. The sources of revenue of CP can be divided into three groups – 1st internally based revenue, 2nd the government grants and 3rd loans & advances. The present financial outlay of CP with regard to solid waste management is quite inadequate and there is an urgent need to improve this situation. Only about 7% of total budget is used for conservancy purpose for CP.

Waste Collection

Cox's Bazar Pourashava has a collection area of 32 sq. kilometres with a total number of conservancy staff of about 189**.** A fleet of 4 trucks with an average payload of 6 tons, 24 vans are engaged in garbage disposal. The reported frequency of garbage collection is once a day. Duration of waste collection is 8 AM to 2 PM.

Solid Waste Disposal Site

The present garbage disposal site is at Kasturighat, consists of an area of 1 acres. The wastes are dumped on to open land. Either a natural of artificial barrier such as ridge of ground or a belt of trees does not screen the site from view. Levelling and compacting the dumped wastes without following any standard specification. No soil cover is provided between the waste layers. Uncontrolled dumping of solid wastes at Kasturighat severely affects both surface waters and ground water of the area. Proposed dumping ground of CP has in the same site with the area of 3 acres.

Case Study on Solid Waste Management of Slum Area

Table 1: Comparison between this two slums of their solid waste management system

Criteria of Comparison	North Baharsora Slum	Jhawtala Slum
Household and Population	About 150 household and population is 500	About 50 household and population is 200
Waste Generation	Around 140 kg per day	Around 50 kg per day
Waste Management System	None	Communal dustbin system
Drainage Facility	There is no drainage facility	There is poor drainage facility

Source: field survey, 2013

Case Study on Solid Waste Management in Residential Area

Table 2: Comparison among residential solid waste management system

Criteria	Case Study-1	Case Study- 2	Case Study- 3
Para/ Mahalla Name	Suja Soudagor Para	Baharsora	Majher Ghat
Ward no	4	11	4
Dustbin Facility	No dustbin facility	No dustbin facility	No dustbin facility
Waste Generation Per Household	About 0.5kg	About 0.5kg	About 1kg
Waste Collection System	None	Communal collection	Curb side collection
Waste Management Organization	None	Pourashava	Pourashava
Collection Procedure	No collection	By Van	By Pick up
Waste Gathering Way	Waste is thrown directly to the drain	Waste gather in waste box	Waste gather in the road side
Drain Management System	Poor (every month)	Good (per 2 days)	Good (per 15 days)
Payment per month	No payment	30 tk per household	No payment

Source: field survey, 2013

RECOMMENDATIONS

Base on the existing situation the following recommendations are provided for better solid waste management.

CP should established its accountability in this respect and should allocate more funds for providing more dustbins and appoint more waste conservancy stuffs.

NGO’s should also extend their programs in solid waste management in other parts of the city.

CP should develop a standard for dustbin for waste collection with the context of area, location and population size.

Most of the staffs of conservancy department of CP are not highly educated and also have no idea about waste management. CP should provide proper training about waste management to the stuffs.

For long benefits awareness program should be taken.

Active participation of community is essential for proper waste management. Active participation can be ensured with the involvement of community based organization for waste management.

With the present level of conservancy stuffs and finance, it is very difficult for CP to improve solid waste management system. So house to house collection system by NGO’s should be promoted.

Modern motorized vehicles and technological methods may be used by CP to improve the environmental quality of the whole city.

To improve the waste disposal system it is possible to use sanitary land filling method because it is low in operating cost and easy to operate and also less chance of water pollution.

Illegal road side dumping should be discouraged

Some treatment and recycling plants are to establish in the periphery area of the city. Government should create interest among the people about the business option from the solid waste recycling.

CONCLUSION

Community based management is an arising issue for sustainable solid waste management. City corporation have no proper solid waste management system especially in the conservancy department the manpower is not available for waste collection, transportation and final disposal of waste. The urban population is increasing day by day tremendously. For this reason private organization started solid waste management program by the community participation with the help of city corporation. The image of Cox-Bazar city is much depends on the proper management of the generated waste. Pourashava authority is the concern authorities for solid waste management of its residence. According to the respondents of the study area it can be conclude that the present system of community based solid waste management of the study area is satisfactory. So, it is proved that with the help of government organization, non-government authorities and community participation can take proper steps to establish solid waste management system which can develop outstanding aesthetic beauty and glorious image for the study area (CP) and at the same time will upgrade the living standard of the people of the city.

ACKNOWLEDGMENT

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STUDY ON SOLID WASTE RECYCLING AND REUSE IN BARISAL CITY

Md. Hamidul Islam[1*], Protiva Adhikary[2], Susmita Howlader Muna[2], Ayesha Siddiqua[2], Shampa Reza Katha[2] & Md. Sabbir Ahmed[2]

[1] *Department of Building Engineering & Construction Management, KUET, Khulna-9203, Bangladesh.*
[2] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <hamidcekuet@gmail.com>*

ABSTRACT

Solid waste commonly known as trash or garbage (US), refuge or rubbish (UK) is a waste type consisting of everyday items that are discarded by public. Recycling is the process of collecting and processing materials that would otherwise be thrown away as trash and turning them into new products. Reuse is the procedure of using any disposed material by proper processing (Haque et al., 2011). Our study area was Barisal city and the main surveyed wards are 5, 6 and 25 because most of the industries are located in these wards. Barisal city is an old port on the Kirtankhola, former Ariel Khan on the northern shore of the Bay of Bengal in southern Bangladesh. It is 142 km (373 km by road) from capital city Dhaka. It's Latitude: 22°38′ and 22°45′ north, longitude: 90.36′. The city consists of 30 wards and 50 mahallas. The area of the town is 20 km². Barisal municipality was established in 1957 and was turned into a city corporation in 2000. Its population is about 224389; male 123402, female 100987 (Banglapedia, 2014). This study focuses on the estimation of the total quantity of recyclable solid waste, to identify the component of solid waste and know the present recycling pattern with the percentage of solid waste components. Recycling and reuse is important for reducing the amount of waste sent to landfills and incinerators. These are done for preventing pollution by reducing the need to collect new raw materials and to sustain the environment. Recycling and reuse method also conserves natural resources and create new well-paying jobs in the recycling and manufacturing industries. The key findings of this study are recyclable solid waste before and after processing as approximately 44930 kg/week and 29002 kg/week respectively. Before and after processing the percentage of recyclable solid waste components as approximately plastic (60.38%; 84.20%), paper (15.80%; 0.34%), metal (13.13%; 3.03%), e-waste (0.45%; 0.006%), and vehicular parts (10.24%; 12.41%) respectively. The recycling pattern or processing type of different solid waste is dissimilar. The recycling pattern of e-waste, metal, plastic and paper is called scrap, cot-pitch or gorda, pet or cutting and belt respectively. In the time of survey work faced different problems mainly people negligence to share information, didn't allow taking photos and recording their interviews. Also the interviewers didn't deliver accurate information because they feel risky to pay more taxes as their industries are non-governmental or private. Raising public awareness about the amount of waste generation and careful on storage system needs for managing waste. Effective government assistance like providing transport facilities or funding that inspire more people to engage in waste management system and which will improve this field as well as the environment.

Keywords: Solid waste, Recycling and reuse, Barisal city, Recycling pattern, and Component of solid waste.

INTRODUCTION

With the economic advancement and population growth, the generation of solid waste is also increased tremendously in different areas of Bangladesh. Improper solid waste management is one of the most serious local environmental problems in our country. Waste mainly any material that is discarded, useless or unwanted. 'Wastes' are substances or objects, which are disposed of or are intended to be disposed of or are required to be disposed of by the provisions of national law" (Basel Convention). The waste can be solid or hazardous but the authors give special focus on solid waste (Haque et al., 2011). Materials are solid waste if they are abandoned by being disposed of or burned or incinerated; or accumulated, stored, or treated (but not recycled) before or in lieu of being abandoned by being disposed of, burned, or incinerated (Environmental Protection Agency, 2012). Some of the common solid waste materials observed in Barisal city include paper, food waste, metals, plastics, glass or ceramics, vehicle parts and a small amount of e-waste. Most of the cases which are known to be hazardous to the environment. These materials can have an economic benefit if managed accordingly in addition to protection of the environment from pollution. Solid waste management refers all activities pertaining to the control, collection, transportation, processing and disposal of those in accordance with the best principles of public health, economics, engineering, conservation, aesthetics and other environmental consideration (Alamgir & Ahsan, 2007). The amount of solid waste in Barisal city is approximately 175000-180000 kg/day. Many approaches are used for reducing the amount of waste in that area such as recycling, reuse, landfill etc. Recycling is a process to change (waste) materials into new products to prevent waste of potentially useful materials and reuse is the process of frequent use of any product.

Fig. 1: Recycling steps in different waste recycling shops in Barisal city.

MATERIALS AND METHODS

The study area is Barisal City Corporation which comprises 30 wards and 50 mahallas. It is situated on the south side of Dhaka. It is not so recent established area as a city corporation but it is one of the crucial municipal areas among the southern part as well as whole Bangladesh. The population density of municipal area of Barisal city is also very high which is about 5000 per sq km. Its population is almost 5 lakhs among which 54.99% male and 45.00% female (Banglapedia, 2014).

The study was conducted from February to August 2014. The main points of view of the study are recyclable and reusable solid waste pattern. It also focuses on the present status of solid waste and its respective management practices. The study was carried out using several municipal inorganic wastes. The related term of this study is different types of solid waste collection, transportation, storage and disposal system in selected area. These studies identify the lacking of waste management and the authority future management plan with the proper consciousness of municipal residents about waste management. During the survey facing the ignorance and unwilling cooperation to the field workers and officials as well as inadequate transparency of their information. Both qualitative and quantities data were collected. Primary data collected by field observation of the study area, focus group discussion with the stakeholders. Secondary information collected for proper documentation like research articles, books, and periodicals.

Primary Data Collection through Questionnaire Survey

To find out the solid waste management practice in the study area the primary data was collected from various classes of people of the selected sector and the respondents were selected randomly. The primary data was collected through questionnaire survey from hawker, day labor, rickshaw/van puller, business man, job holder to assess the exact situation of solid waste management with direct field observation. Primary data was also collected by visited the waste collection site and different small private agency. Visiting the selected ward as well as sighting the dustbin of the road site and its situation was also observed.

Secondary Data Collection

Secondary data about population, volume of waste generation, activities exiting on solid waste management in selected study area were collected. It was collected from Barisal city corporation office. For assessing expert opinion the key informant interview was conducted with the various stakeholders who were expert and associated with solid waste management practice in the selected area. The engineer of municipality of Barisal and the planner are given an opinion about present solid waste management with their upgrading process for better solid waste management.

Fig. 2: Processing of solid wastes in the study area.

RESULTS AND DISCUSSIONS

Though solid can be organic or inorganic but the study mainly based on the inorganic solid waste. The findings of the study on solid waste are that the waste consists of several components mainly plastic, paper, metal, e-waste, vehicle parts. Different elements and sources are shown in Table1.

Table1: Elements and sources of solid waste

Types of solid waste	Sources of waste
Paper	Paper scraps, cardboard, newspapers, magazines, bags, boxes, wrapping paper, telephone books, shredded paper, and paper beverage cups.
Plastic	Bottles, packaging, containers, bags, lids, cups.
E-waste	Broken TV, radio, computer, wires.
Metal	Tins, non-hazardous aerosol cans, appliances (white goods), railings, fishing tools.
Vehicle parts	Bicycles, Automobile parts, shipping unusable parts.

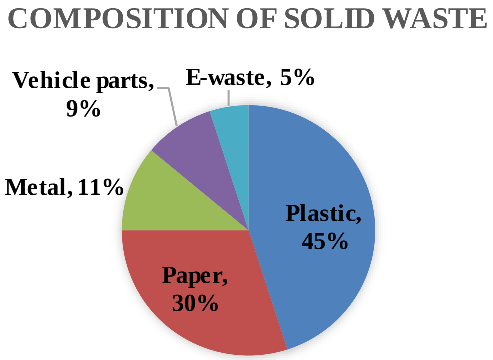


Fig. 3: Composition of various solid wastes in percentage in Barisal city.

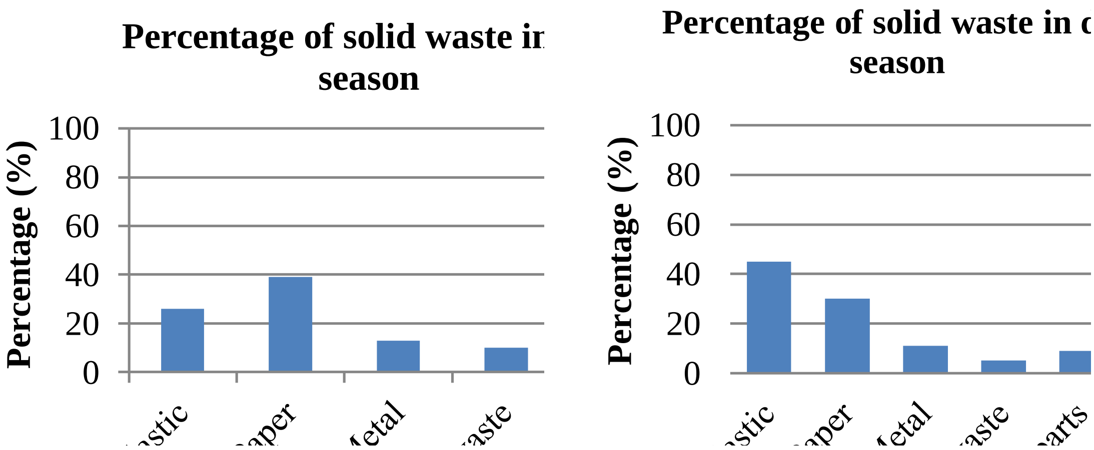


Fig. 4: Percentage of different solid wastes in wet and dry season respectively.

As our study time was March to August it contains summer and rainy season. The amounts of different solid wastes in different season as wet and dry season respectively are shown in figure 4.

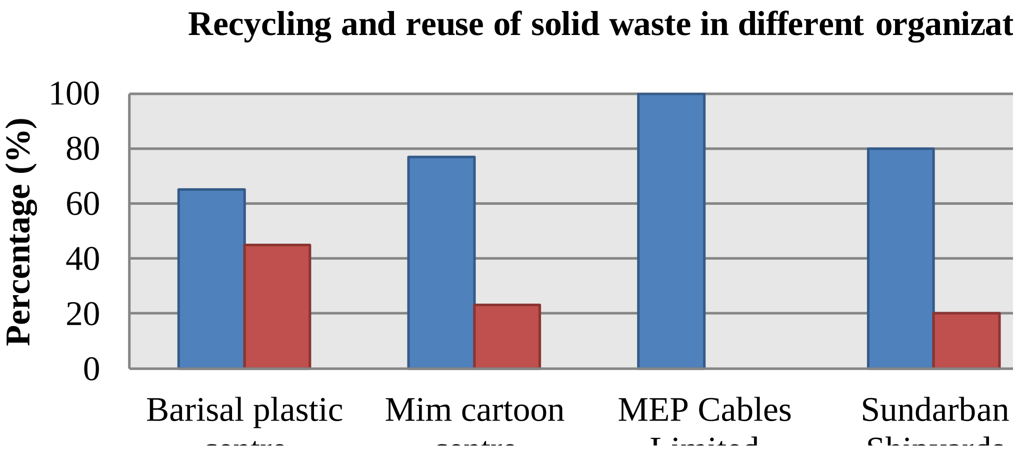


Fig. 5: Percentage of recycling and reuse of solid waste in different organization in the study area.

In Barisal city many process are using for management of waste. From those recycle and reuse the most using process for management of waste in that area. Most of the organizations are NGOs and GOs. Different organizations have different status of the amount of recyclable and reusable wastes as shown in figure 5.

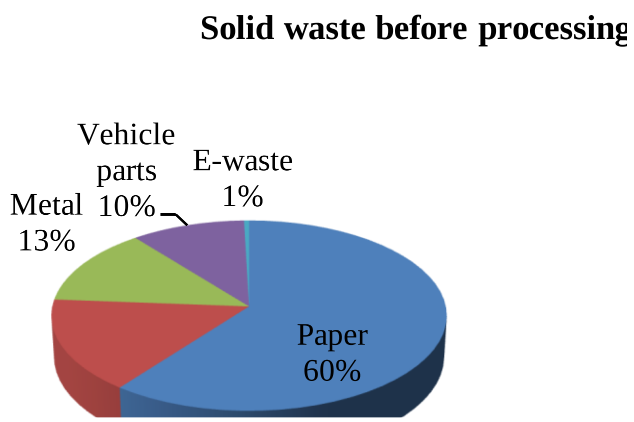
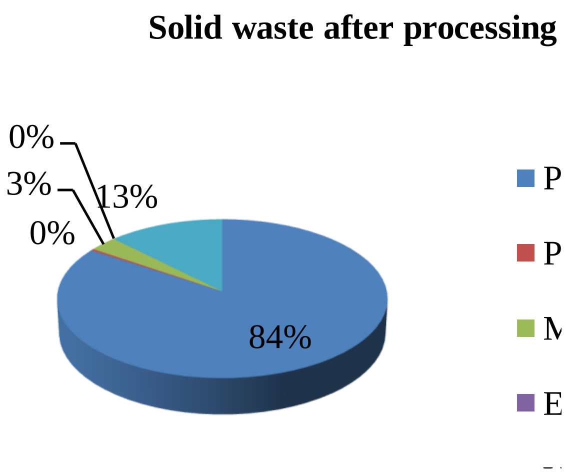


Fig. 6: Percentage of different wastes before and after processing in Barisal city.



Waste cannot be removed but can be managed. So that after making a things or goods waste must be produced and then that waste can be used as a raw material in making different products again. From this study the statistical records in figure 6 shows the amount of different solid waste before and after processing respectively.

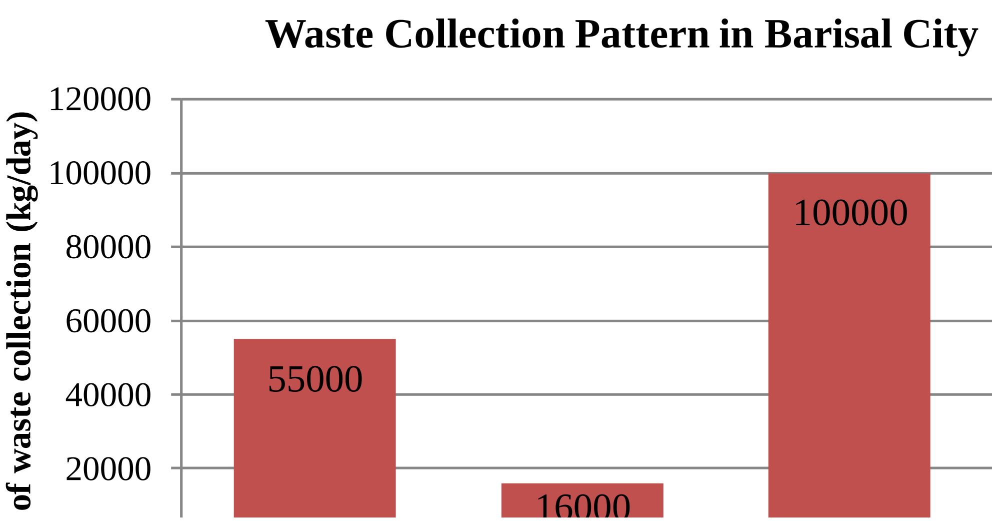


Fig. 7: Amount of collected waste from different sources kilogram per day in Barisal city.

CONCLUSION

Waste is said to be a mirror of the society since waste generation and disposal reflect a range of aspects of the society such as its economic, historical, cultural and environmental components. In Barisal city area the amount of solid waste is increased day by day but its management system and disposal site are not quite enough developed. The study said that the estimated value of the total recyclable solid waste is 44930 kg/week. Among all waste the recyclable components of solid waste and their percentage are plastic 60.38%, paper 15.80%, metal 13.13%, e-waste 0.45%, and vehicular parts 10.24% respectively. The recycling pattern of plastic, paper, metal, e-waste is called pet or cutting, belt cot-pitch or gorda, scrap respectively. Most of people in the Barisal city said that the municipal waste collection with management system is very poor and weak than the necessity. Although, there is a little help and investment of NGOs in these sector and they take huge expense. Most of the time private company show their interest on this sector. Whenever they invest on waste management it would not economical viable for local government as well as local day labourers. To improve this condition the government should increase self-investment for the concern on waste management and encourage NGOs and private agencies to work and legislate on their activities. Moreover, public awareness is crucial for waste management. Training should be required on personal hygiene as most waste collectors are presently unaware of the consequences of garbage sorting without adopting safety guidelines.

ACKNOWLEDGMENTS

The authors would like to express their deepest gratitude to Almighty God firstly. Then the authors also would like to extend their heartfelt appreciation and thank to Barisal city corporation officials, other agency owners and workers for their cooperation, giving information, valuable comments and suggestion.

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STATE OF THE FUNCTIONAL ELEMENTS OF SOLID WASTE MANAGEMENT IN CHITTAGONG CITY CORPORATION AREA

Md. A. Bashar[1*], T. M. A. Rahman[1], A. Rahman[1], I. Haider[1], S. A. Mohit[1], Md. Mohiuddin[1], A. Chowdhury[1], Md. A. Rahman[1] & Md. S. Sarkar[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong - 4349, Bangladesh.*
**Corresponding Author, <muhammadbashar@hotmail.com>*

ABSTRACT

Solid waste management is a very vital issue for a developing city. By the passage of time, the waste generation of a city increases day by day in conjunction with the increase of population and economic development. But as the authorities and the inhabitants are often found neglecting the proper management of solid waste, it hampers the urban environment both economically and hygienically. Though Chittagong is a very important city, it also faces this kind of itchy situation which results in various pollutions, unhygienic sanitation and also economic losses sometimes. This paper represents an attempt to investigate the state of the solid waste management of this port city. Physical surveys were conducted to accumulate the data of present state of the storage, collection, transportation, processing and disposal system and management along with the cooperation of city dwellers. Survey and data analysis shows a clear picture of lack of public awareness and an insufficient management and improper legislation which is not fully capable of satisfying the demand of a sound solid waste management along with the growing population and rising development. This study also reveals that improper placement and unhygienic use of dustbins or containers accompanied with unplanned collection and transportation schedule creates an unhealthy condition. There exists a recycling and processing system of biodegradable wastes, only in the southern zone of Chittagong city. Rest of the non-biodegradable waste are left untreated and unprocessed excluding some partial incineration and recycling. Moreover, the location of the dumping zones is not far from the locality and the Bay of Bangle that may need to be transferred in a new location in future. Clear and specified legislation, adequate manpower and active equipment, generation of mass awareness and above all strict monitoring by the authorities concerned can change the present situation into a healthy city through energy efficient and environment friendly solid waste management system.

Keywords: Solid waste management, Collection, Transportation, Processing, Disposal, Legislation, Environment.

INTRODUCTION

Solid waste management is a very important affair for any city. Especially a city like Chittagong, which plays great role in the development of Bangladesh, should possess a sound and pollution free city environment by ensuring the proper solid waste management. To maintain a clean and healthy environment a city should have the total system, functionality, administrative and financial control and supervision on waste generation, storage, collection, transfer and transport, processing and disposal of solid wastes. Since solid wastes pollutes the environment of the city and creates nuisance among the lives of people, the functions followed should be under proper planned and engineered management. But in comparison with the rising development and increasing population it is often quite difficult to keep harmony with the waste handling and management properly. Last few decades the population of our country increased. The population of Chittagong district has increased to 7616352 in 2011 with an annual growth rate of 1.40%, in 2001 which was 6612140 with the annual growth rate of 2.24% (BBS, 1997 and Population Census 2011). With the increase of population the generation of waste increases. In 2012 the total generation of waste is as close as 1161.113 tons/day

(BMDF Report, 2012). Present condition of SWM is not good due to inadequate implementation of social, technical, institutional and financial issues. Almost no on-site processing, negligence in transportation, wrongly selected disposal site is the governing issues in Chittagong city (Alam and Sohel, 2008). To improve the solid waste management it is important to evaluate the present condition. So an initiative has been taken to find out the current situation of the solid waste management of Chittagong City corporation area. The objective of this study is to represent an overall scenario of the solid waste management of Chittagong city corporation area.

METHODOLOGY

To understand the present condition of solid waste management of Chittagong City Corporation area both primary and secondary data have been collected. Primary data has been collected from field investigation. Questionnaire survey has been conducted in residential areas and Chittagong City Corporation personnel and different NGOs. The volume of generated waste has been measured by the volume of the bin. Secondary data has been collected from corresponding authorities, NGOs and previous literature. The collected data is analyzed and summarized to assess the state of the solid waste management.

PRESENT CONDITION OF FUNCTIONAL ELEMENTS OF SWM

Population growth of Chittagong city is depicted in the Table 1 with comprising the population growth in Bangladesh. This represents the population and urban development growth which greatly effects the waste generation and management.

Table 1: Urbanization and Urban population growth in Bangladesh and Chittagong

year	Bangladesh			Chittagong	
	Total Urban Population	% of Urban Population	Avg. Annual Growth rate (%)	Total Urban Population	Avg. Annual Growth rate (%)
1981	13535963	15.54	10.63	1,025,846	7.17
1991	20872204	20.15	5.43	1,392,860	3.57
2001	28808477	23.39	3.27	2,023,489	4.53
2011	-	-	-	2,971,102	3.91

Source: BBS, 1997; BBS, 2001 and Population Census 2011

The state of handling and managing the generated waste by this increasing number of population as well as increasing number of mills and industries of Chittagong City Corporation area is presented below within the functional elements of solid waste management.

Waste Generation

Obtained solid waste generation in typical types of residential areas is listed in the Table 2 and also the total waste generation and calculated per capita waste generation is also listed later in Table 3.

Table 2: Residential waste generation rate in Chittagong City Corporation (Mass-volume analysis)

Sample area Type	Locations	Population	Volume (m^3)	Mass (kg)	Generation Rate (kg/c/day)
Residential Area	Chadgaon Residential Area	51	1.5	13	0.26
Colony	Eastern Refinery Colony	459	10	110.16	0.24

Apartment	Jamal Khan Area	53	1.45	16.4	0.31
Unit Family Dwelling	GEC Area	16	0.47	4.6	0.29
Slum	Tigerpass Area	10	-	2.3	0.23

Source: Field Survey

Table 3: Waste Generation Rate of Chittagong City Corporation

Description	**Generation Rate/day**
Per Capita Domestic Waste Generation Rate (kg)	0.266
Population in CCC	4.03 million
Population with 10% floating	4.44 million
Total Domestic Waste (ton)	1181 ton
Market Waste (ton)	194.67 ton
Street Sweeping Waste (ton)	184 ton
Total Waste (ton)	1559.67 ton
Per Capita Waste Generation (kg/c/d)	0.351

Source: Field Survey, BMDF Report, 2012

On-site Handling, Storage and Processing

On-Site Handling

In Chittagong house-to-house residential collection service is adopted by most of the communities. Primary collection service provider (PCSP), usually from NGOs', collects the waste from the generation point. Householders store the wastes in bucket and PCSP collect those wastes and return the bucket to the householders. PCSP uses Rickshaw-van, which is generally 5x3x3 ft^3 and can accommodate 100-120 kg of residential wastes. Primary collection time starts from 11am-4pm and householders pay around 40 tk. per month for this service. Usually PCSPs' empty their vans in the primary storage points. In Chadgaon residential area PCSPs' deposits all the wastes at the only primary storage point located beside the Sadhinota Complex.

On-Site Storage

Commonly three types of storage points are used in Chittagong City Corporation area. They are Steel Containers, Brick Masonry Storage and Open Storage. Steel containers have the capacity of 3 to 5 tons of wastes. This type of container is mainly found in locations where waste generation rate is relatively higher, like near bazar, EPZ etc. Brick-masonry storages and open storages are used as secondary storage points and later they are transferred to the primary storage point.

On-site Processing

PCSP collects mainly garbage from residential area so there is no need of major on-site processing. But generally manual sorting out and recovery is carried out by informal sector.

Collection

It is estimated that 60-70% of the total cost of SWM is spent on the collection phase alone (Ahmed and Rahman, 2003). In Chittagong both communal collection and house-to-house collection systems are followed but in the city corporation area house-to-house collection system is widely used. In house-to-house collection system waste collection workers collect the bin, basket or bag, empty it into the collection vehicle (generally rickshaw van) and return the containers to the premises. This system is mostly run by various NGOs' also by the local community in some cases. Stationary container system is widely used in Chittagong city. As the container station important city and community points like Katgor, Steel Mill, Taltola, Bondor Tila Bazar, EPZ, Sagorika, Agrabad, Choumuhoni Bazar, Mile'er Matha, Postar Par (Dewan Hat), Choumuhoni Khan Bari, Madarbari, Bangla Bazar, Post Office, Hasina Oil Mill, Daily Life (Old Custom), Port Market, K-block, WAPDA PDB pole, PEPSI Manufacture, Colonel Hat are taken in south zone. In Chittagong container haul vehicle and waste carrying truck are used where trucks are manually loaded.

Table 4: Solid Waste Collection in Chittagong City Corporation

Item	Description
Production (Ton/day)	1559.67
Dumping Place	South Zone: Ananda Bazar, Halishahar, Chittagong (Area= 5 acres.)
	North Zone: Arefin Nagar, Oxyzen, Chittagong (Area=16 acres.)
Total Containers	54 nos.
Container Capacity	5-8 Tons
Total Number of Dump Trucks	25 nos. in each zone
Total Avg. Number of Trips per day	90
Total Container Movers	13
Total Number of bins	1263
Total Collection of Waste (Ton/day)	880-940
Collection Efficiency	**56-60 %**

Source: Field Survey

Transfer and Transport

Chittagong City corporation waste moving trucks are generally 14*5.3*2 cft (BMDF Report, 2012).

Most of the transportation works are operated during early morning, before 10am. In some densely populated area where the waste generation is high transportation is performed during both morning and afternoon. The capacity of the trucks is 2 tons and 5 tons. Waste workers usually visit all the points once a day except k-block, EPZ, Mile'er Matha, Daily Life. These points are visited twice or more.

Processing and Recovery

In Chittagong most of the wastes are food waste which is easily decomposable. Wastes are sorted in the south zone (Ananda Bazar, Halishahar) by the scavengers and then the decomposable part is composed in a compost plant.

Table 5: Compost Plant by Filing Box Method in Chittagong South Zone

Name	**Description**
Raw Waste	Vegetable, fruit and other food waste
Additive	10% raw dust and water

Maximum allowable Temperature	70°C
Composting Duration	45 days
Maturity Duration	7 days

Source: Field Survey

Disposal

Landfilling is the method adopted by Chittagong City Corporation. It can be called crude dumping rather than landfilling. Having a useful life greater than 1 year it doesn't seem to fulfill other factors satisfactorily. Among the two drains one is overloaded with wastes (Figure: 1) and other one carries leachate along with other toxic substances and directly meets the Bay of Bengal (Figure: 2).

Figure 6: Overloaded Drain **Figure 2: Leachate carrying drain.**

Geologic or hydrologic conditions requirements are completely absent. Because this site is located less than 100m from the sea (Figure: 5) and there is a paddy field just foots away from it (Figure: 2). Among the problems that a potential landfill site may have damage to access roads by heavy vehicles is the one to taken into account (Figure 3).

Figure 3: Damaged access route. Figure 4: Water sample collection.

Figure 5: Dumping zone and Bay of Bengal

Three water samples were collected from hand pumps at a close proximity from the disposal site and here are some water quality parameters:

Table 6: Water Quality Parameters

Test Results				**Location**		**Standard Guidelines**		
Sample NO.	DO (mg/L)	BOD_5 (mg/L)	pH	Latitude and Longitude		DO (mg/L)	BOD_5 (mg/L)	pH
1	5.6	5.465	7.0	N 22° 18.69′ E 91° 46.42′		6.0	0.2	6.5 - 8.5
2	4.8	4.25	7.0	N 22° 18.73′ E 91° 46.37′				
3	5.4	5.16	7.0	N 22° 18.74′ E 91° 46.40′				

Source: Field Survey, Amio Water Treatment Limited, 2010

CONCLUSION

The water quality parameters show a higher value of BOD which indicates the presence of micro-organisms in the underlying soil. The waste generation rate of Chittagong city corporation area is much lower than the generation rate of a developed city. It shows a proper management system can easily ensure a clean and healthy city. Lack of proper knowledge and attention of general people, improper compatibility of elementary management and unavailability of specific regulations has made the solid waste management scenario more sophisticated. Legislations should be updated to include some fines if anyone doesn't follow the rules. Storage points should be placed in proper locations where collection becomes easy and vectors do not have easy access to it. Authorities should maintain the time schedule properly and encourage the citizens to put the waste into the dustbins.

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ASSESSMENT OF BRICK KILNS IN BANGLADESH: ENERGY, FINANCIAL AND ENVIRONMENTAL PERFORMANCE

S. Pk.[1*], M. S. Hossain[1], L. C. Mazumder[1] & T. Sultana[1]

[1] *Institute of Water Modelling, Dhaka, Bangladesh.*
**Corresponding Author, <pk_shubhra@yahoo.com>*

ABSTRACT

This study has been carried out to assess Energy, Financial and Environmental performances of brick kilns in Bangladesh. The study area is greater Kishoreganj and Mymensingh districts where almost 12% of countries brick fields are situated. Field visit and questionnaire survey have been conducted in 130 brick factories to collect primary data. A field data collection sheet, containing 15 questions has been prepared to record field survey data. Summarising and analysing collected 130 data sets it is observed that, almost 72% of brick kilns are Fixed Chimney Bull's Trench Kiln (FCBTK), 23% are Zigzag kiln and only 5% are Vertical Shaft Brick Kiln (VSBK). FCBTK has the lowest energy and environmental performance but it has the highest financial performance. In VSBK opposite scenario is observed. Zig-zag kiln has moderate performance in all three types of performance assessment indexes and it would be the best option in future because it offers easy integration with existing FCBTK.

Keywords: Bangladesh, Brick kiln, Energy, Environmental, Financial, Kishoreganj and Mymensingh

INTRODUCTION

The brick production in Bangladesh is seasonal, confined to the six dry months of the year and this industry is best described as a “footloose” industry (IIDFC, 2009). In most cases brick manufacturing technology is outdate, about 150 years old; labour productivity is low; capitalization non-existent, operating mostly on equity capital; and management is informal. These phenomenon’s cause a huge energy as well as financial loss every year. In addition, brick making process is creating greenhouse gas emissions in Bangladesh, estimated to be in the order of 3.0 million tons of CO2 annually (IIDFC, 2009). Most of the Bangladesh brick industry utilizes ancient types of brick kilns. Typically, a low-grade coal from India along with firewood and rice husks and tiers are used to fire these kilns. These kilns are found to be poorly constructed and are highly inefficient in fuel consumption. They also found excessive air leakage and heat loss from the system causing high level of emissions. Although a number of energy saving, cost effective and less carbon emitting brick kiln is available, here still we are using ancient types of brick kiln due to inadequate application of rules and regulations, lack of technology and skill labour, lack of awareness, lack of initial capital and to use all kinds of fuel such as wood, tire, coal etc.

This study analyses energy, financial and environmental performance of major brick kilns those are presently using in Bangladesh. Most of the industries are using lower quality fuel to minimize production cost but due to old and inefficient production technology the cost is growing higher. These lower quality fuel and ancient production technology are producing higher volume of carbon di-oxide and other toxic gases those are polluting our environment gradually. This pollution is creating various respiratory and eye diseases to the brick industry’s labours and surrounding inhabitants. Brick kilns govern the energy consumption rate, emission of carbon di-oxide and other toxic gases and production of quality bricks. Therefore higher quality kiln ensure higher financial performance but it is important to select right kind of brick kiln for perfect matching with surrounding other conditions.

In Bangladesh, only a few numbers of brick kilns are practicing now. Fixed Chimney Bull's Trench Kiln (FCBTK) originally developed in India is the common type of brick kiln using in Bangladesh. Many new technologies, such as the Hebla or Zig-zag kiln, Vertical Shaft Brick Kiln (VSBK) and the Hybrid Hoffmann Kiln (HHK) are substantially cleaner than the Fixed Chimney Bull's Trench Kiln (FCBTK) currently used (ESMAP, 2011). These improved technologies consume less energy and emit lower levels of pollutants and greenhouse gases but the use of these technologies in Bangladesh is still in the pilot stage of implementation (ESMAP, 2011).

STUDY AREA

This research has been conducted in greater Kishoreganj and Mymensingh districts of Dhaka division those are located in the central Bangladesh. This area contains huge numbers of brickfield, almost 12% of the country. Kishoreganj district is situated in 24°26′ N to 24°61′ N latitude and 90°47′ E to 90°87′ E longitude (Md. Enamul haque, 2012). Total area of this district is 2688.62 sq. km which is bounded by Netrokona and Mymensingh districts on the north, Narsingdi district on the southwest and Brahmanbaria district on the southeast, Sunamganj and Habiganj districts on the east, Gazipur and Mymensingh districts on the west (LGED, 2014) (Fig. 1). It consists of four municipalities, 39 wards, 145 mahallas (sub-ward), 13 upazilas (sub-district), 105 unions, 946 mouzas (sub-union) and 1775 villages (Md. Enamul haque, 2012). Mymensingh district is situated on the west bank of Brahmaputra river with an area of 4363.48 sq. km and located in 24° 0'' N to 24° 40' N latitude and 90° 40' E to 91° 15' E longitude (Md. Khaled Ibrahim, 2011) (Fig. 1). Mymensingh districts is bordered on the north by Meghalaya state of India and Garo Hills, on the south by Gazipur district, on the east by districts of Netrokona and Kishoreganj, and on the west by districts of Sherpur, Jamalpur and Tangail (Banglapedia, 2008) (Fig. 1). Several numbers of major and minor rivers such as Old-brahmaputra, Meghna, Kalni, Dhanu, Ghorautra, Baurii, Kangsa, Bhogai and Narasunda are flowing over the study area. This area is very important for the development of capital Dhaka and surrounding developing areas because most of the bricks and other construction materials are supplied from Kishoreganj and Mymensingh districts.

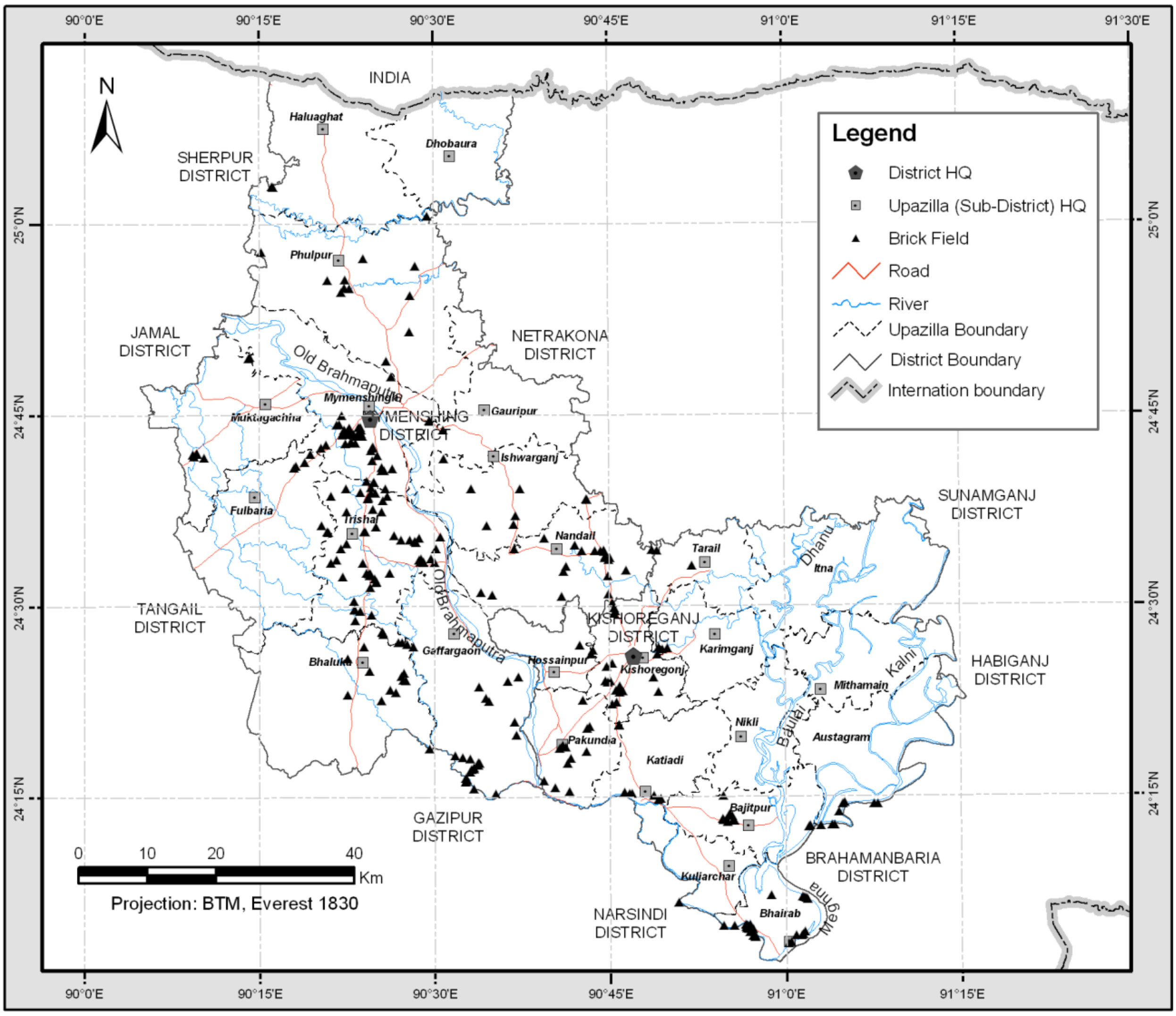

Figure 1 Base map of the study area showing major brick fields location

STUDY METHODOLOGY AND DATA COLLECTION

Field visit and questionnaire survey are the methodologies of primary data collection in this study. A data collection sheet containing 15 questions has been prepared to collect field data (Table 1). Total 130 brick manufacturing factories from every upazilla (sub-district) of Kishoreganj and Mymensingh districts have been surveyed in July, 2013. Financial and energy performance related information have been collected from factory owner or manager of the brick factory. Interviews are taken of brick field's workers and surrounding inhabitants to collect environmental impacts data. All the collected data are summarised according to brick kiln types and analysed to calculate fuel consumption rate, fuel consumption efficiency, production cost and percentages of brick field where workers are suffering in various respiratory, skin and eye diseases.

Table 1 A sample field data collection sheet

Date: ……………………………………………………	Time: ……………………………………
Brick field name: ………………………………………………………………………………	
Brick field locations: ………………………………………………………………………..	
Number of production months per year	: ………………………………………
Average number of hampered working day due to weathering conditions per year	: ………………………………………
Kiln types	: ………………………………………
Height of the chimney	: ………………………………………
Diameter of the chimney	: ………………………………………
Fuel types	: ………………………………………
Amount of fuel used per million of bricks burning	: ………………………………………
Amount of waste fuel per million of bricks burning	: ………………………………………
Average cost of production per million of bricks	: ………………………………………
Are the workers suffering in any respiratory, eye or skin diseases?	: ………………………………………
Are the surrounding inhabitants suffering in any types of respiratory, eye or skin diseases?	: ………………………………………
Is there any labourer died in any disease, if yes, what was the disease?	: ………………………………………
Future improvement plans	: ………………………………………
Respondents	
---------------------- Name	---------------------- Signature

RESULT AND DISCUSSIONS

Among 130 monitored brick kilns, the number of FCBTK, Zig-zag kiln and VSBK are 94, 30 and 6 respectively. Most of the factories are using FCBTK because of its lower construction cost and facilities of using all kind of fuels. On the other hand, Zig-zag kilns are getting popularity in recent years because of its higher quality products. The practice of VSBK using is not started very well in Bangladesh and only 6 are found in the study area. It is observed that, the size of FCBTKs and Zig-zag kilns are comparatively larger than VSBKs and the average chimney height of FCBTK, Zig-zag kiln and VSKB are 30-35m, 18-22m and 20-25m respectively. Summarising and analysing collected information, the energy, financial and environmental performances of three types of brick kiln are assessed.

Energy performance

A large variation is observed in the energy performance of the monitored kilns. VSBK (most efficient kiln) consumes approximately half energy compared to FCBTK (least efficient kiln) to produce same volume of brick. The energy consumption rate of Zig-zag kiln is more than VSBK but lower than

FCBTK. Zig-zag kiln distributes heat uniformly, resulting in uniform temperature across kiln cross-section, results in a higher class-I brick output.

Financial performance

The FCBTK is very cost-effective because of its simple technology, less payback period, other than coal and firewood; discarded tires can also be used as fuels and no need of expert worker. Due to these advantages this is very popular types of brick kiln in terms of financial performance. Zig-zag kilns require lower investments and have shorter pay-back periods but due to fuel types the production cost is slightly higher than FCBTK. Zig-zag kilns offer easy integration with the existing production process of FCBTKs (SHAKTI, 2012). VSBK can be used for small-as well as medium-scale production, but has problems of low productivity and poor fired brick quality with certain clays. It is suitable for firing only solid bricks and has a longer pay-back period compared to Zig-zag and FCBTK kilns.

Environmental performance

VSBKs emit lowest amount of Suspended Particulate Matter (SPM) and Particulate Matter (PM 2.5) followed by Zig-zag kiln, and FCBTK (SHAKTI, 2012). VSBKs have very low black carbon (BC) emissions, followed by Zig-zag whereas FCBTKs have the highest BC emissions (SHAKTI, 2012). In this research, it is observed that in 27 brick field sites the workers and in few cases the surrounding inhabitants are suffering in various respiratory and skin diseases. Among this 27 brick field sites the FCBTK sites are 25 and Zig-zag kiln sites are 2. These result illustrates the FCBTKs have lowest environmental performance whereas VSBKs have the highest. Zig-zag kilns have moderate environmental performance but it is much better than FCBTKs.

CONCLUTIONS

Fixed chimney bull's trench kilns are the most common types of brick kiln in Bangladesh. The popularity of this kiln is governed by the facility of using all types of cheap and locally available fuels, large scale production, no need of much technical knowledge, good financial performance and no restriction from environmental performance. FCBTK has lowest energy and environmental performances. On the other hand VSBK has highest energy and environmental performances but due to some limitations in production raw materials quality this kiln has lowest financial performance in Bangladesh. Zig-zag kiln produces excellent quality bricks and its energy, financial and environmental performances are higher than FCBTK. This kiln can be the best option for Bangladesh because, it has better energy and environmental performance than FCBTK and it gives the facility of easy integration with existing FCBTK. If it is possible to convert our existing FCBTK into Zig-zag kiln, the quality of brick will improve and the emission of carbon di-oxide will reduce.

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ROLE OF GREEN ROOF ON TEMPERATURE REDUCTION AND SOCIO-ECONOMIC DEVELOPMENT IN A CITY, BANGLADESH

S. Halder[1], M. A. A. Pavel [1]*, M. B. Uddin[1], K. Harada[3] & S. K. Bala[2]

[1] Department of Forestry and Environmental Science, Shahjalal University of Science and Technology, Sylhet, 3114, Bangladesh.
[2] Institute of Water and Flood Management, Bangladesh University of Engineering and Technology, Dhaka,1000, Bangladesh.
[3] Graduate School of Bioagricultural Sciences, Nagoya University, Japan.
**Corresponding Author, <pavel.sust@gmail.com>*

ABSTRACT

Our green surrounding gradually disappears as our villages turn to cities. Consequence, horizontal green is decreasing at a large scale. Moreover, the green cover in urban areas around the world is being replaced with concrete and brick. They do not find plant and trees so abundantly around them. Accordingly, it creates many environmental problems such as temperature rising. However, roof gardening is a new concept for urban greenery. By introducing roof gardens, sustainable benefits regarding environmental, social and economic can be achieved. It can improve urban environmental problems, enhance community functions and ensure urban food security. The study analysed the environmental economic aspects through cost-benefit analysis of roof gardening at the city with respect to temperature reduction. The people of the city are practice roof gardening mainly for aesthetic value and for passing leisure time. However, this study showed that net benefit from roof garden was 5,710.17 Tk. (US$ 73.85), which was financially attractive as a small scale venture where no active labor required. It was also found that green roof reduces temperature to a limit of 6.7° C. It was also a small scale business to accelerate additional family income for their livelihood. This study, therefore, is aim to portray the overall status of roof garden, financial benefit and environmental service through roof garden.

Keywords: Climate Change, Greenery, Temperature, Valuation.

INTRODUCTION

On the turn of the 19th to 20th century many South-Asian countries have experienced high economic growth accompanied by rapid urbanization like Bangladesh. Rapid population growth, migration and poor urban planning resulted in unhealthy and fragile environment which is alarming to the city dwellers (Kohler and Keeley, 2005). Consequently, the total amount of forest cover of the country is decreasing at a large scale (Saiz et al., 2006). Sylhet is one of the fastest growing cities in the Bangladesh (SCC, 2012), rapid population growth has created severe pressure on the land use planning. Agricultural lands or forest lands have given way to housing developments and roads for exceeding population as well. With rapid and unplanned urbanization, incidence of urban poverty and food insecurity has been also increasing alarmingly. Furthermore, roof gardening (RG) is an art and science of growing plants on the fallow spaces within, surrounding or adjacent to the residence, most often referred to as a garden, other conventional areas include atrium, balcony and window boxes. Clearly, plants are grown for a variety of utilitarian and non-utilitarian purposes. RG differs with farming in terms of scale i.e. gardening can be a hobby or an income supplement, but farming is a full-time commercial activity involving more land and quite different practices (Cultip, 2006). RG can be an effective method in ensuring food supply and satisfying nutritional needs of the inhabitants. RG, although is being practiced in the SCC in many form for years in the past, there have been hardly any concerted effort on part of the Government, community organizations and as well the general citizens to integrate in to urban agriculture. Proper understanding of the problems and prospects associated with the adoption of policies will contribute, to a great extent, to increased food supply in the city. Our study is an effort in this direction to identifies the long-term policy measures for RG that can become the basis for a sustainable approach for urban agriculture (Islam, 2004). Urban growth resulted in tremendous increase of energy consumption of building (Mahdeloei et al., 2012). More use of green roofs can reduce the severity of some of the environmental problems of modern cities. Buildings that have roof gardens-are, in comparison with the building ordinary roofs, require less heating in winter and in summer are much less cold; and can lead to significant savings in energy consumption. The roof gardens have the potential to act as insulation for the roof because of heat exchange with the outside environment to prevent (Kohler and Keeley, 2005). Increase in CO^2 over the past, however, most of the rest is due to land-use change, in particular deforestation and degradation. The finding of this research is green roof will be one of the best solutions against deforestation of urban areas in Bangladesh, without a doubt these cost of a green application on building lot less than if climate change continues its trend (Baas and Baskaran, 2003). Some cities are trying to enhance sustainability by improving urban greenery and promoting urban agriculture or farming (Beattie and Berghage, 2004). By installing RG with urban farming, it is possible to achieve environmental, social and economic sustainability for the buildings in urban cities because it can contribute to the improvement of environmental problems, enhancement of community functions and development of urban food systems (Greenstone, 2009). This study is aim to portray the overall status of roof garden, financial benefit and environmental service through roof garden.

RESEARCH METHODS

STUDY AREA

Sylhet is a main city in north-eastern Bangladesh, and was granted metropolitan city status in March 2009. The city has a high population density, with nearly 500,000 people. The Sylhet is well known for its tea gardens and tropical forests, the city however is currently known for its business boom-being one of the richest cities in Bangladesh, with new investments of hotels, shopping malls and luxury housing estates.. Sylhet consists of 27 Wards and 210 Mahallas, and has a total area of 26.50 km². During the colonial period, experienced rapid growth and expansion of the city (SCC, 2012).

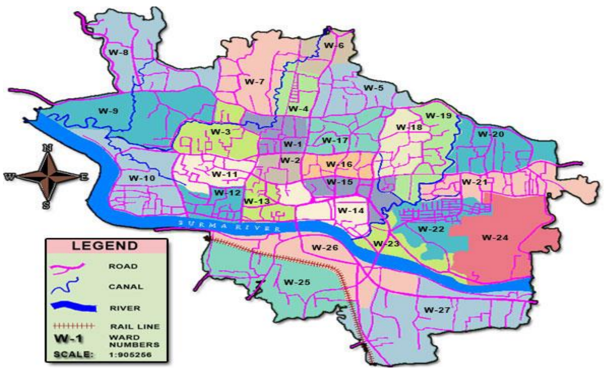

Fig. 1. Location map of study areas (SCC, 2012)

SAMPLING DESIGN AND ANALYSIS

Sampling design was conducted into two stages, cultural and financial, and then environmental survey in the study areas. Simple random sampling was used for the cultural and financial survey. About 7 wards are randomly selected from 27 wards which almost covered 25% area of SCC. The selected wards were 1, 3, 4, 9, 12, 18 and 21. Then 100 households were selected randomly from each ward.

Qualitative and quantitative both data were collected for cultural and financial survey except environmental data. Using a semi structured questionnaire, information on roof gardening like, establishment year, size, composition, cost, out turn, etc. was gathered through individual household survey. In case of environmental data, temperature readings were taken by using "weather meter". For this two roofs were selected purposively for the study-one of them having roof garden and another without having roof garden or reference roof. Data were collected purposively from 5 points of the roof garden and the reference roof which was the mirror reflects of the roof having roof garden. For analyzing qualitative and quantitative data, used different types of software such as MS Excel, statistical software SPSS.

RESULT AND DISCUSSION

STATUS OF ROOF GARDENS

Estimated number of housing plots in Sylhet City Corporation (SCC) is about 29,381 in 26.50 square kilometers (SCC, 2012). Among the houses, more than 90% are residential buildings and 10% are institutional buildings (private and public). The residential buildings are mostly in private possession and few residential buildings are government official staff quarters. The survey shows that out of 500 households, on an average only 7.286% of the houses are bestowed with gardens either in roofs or in balconies (Table 1).

Table 1.Status of roof gardening in study areas

		House with Roof Garden			
Name of Zone	**House without Garden**	**Roof**	**Balcony**	**Total**	**House having Garden (%)**
W1	90	8	2	10	10
W 3	96	4	0	4	4
W4	92	5	3	8	8
W9	89	9	2	11	11
W12	95	5	0	5	5
W18	94	5	1	6	6
W21	93	5	2	7	7
Total	**649**	**41**	**10**	**15**	**7.286**

Note: w-ward

PURPOSE OF ROOF GARDENING

The result shows that major purposes of roof gardening are passing leisure time (96%), creating aesthetic values (94%), showing luxury (25%), getting psychological health (27%), achieve financial gain (12%), contributing in environmental amelioration (12%), and awareness being a very minor portion (4% only) (Fig 1). Roof gardening is a leisure time activity that involves minimal off farm physical labor. The plants contributes in reducing thermal temperature to its immediate surroundings, intercepts or act as a screen for dust reservoir, abate noise pollution, reduce reflection, reducing atmospheric carbon (Wenger, 1984) Pant, 1994; Sajjaduzzaman et al., 2005).

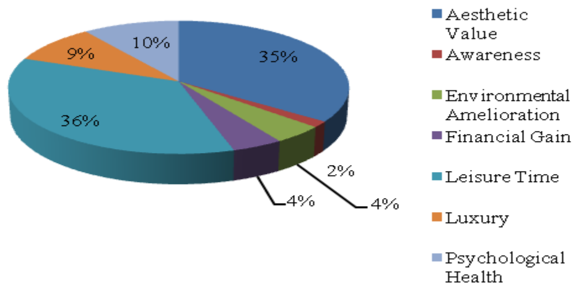

Fig. 2. Purpose of raising roof garden.

TREND OF ROOF GARDENING

Practice of RG is comparatively an ancient idea of urban forestry but is greatly neglected by the urban people for many reasons (Rashid, et al., 2010). From recent years people of SCC are conscious and interested of roof garden, interesting information is found that the number roof garden is increasing day by day except 2005. From 2006 to 2007 the increasing rate of roof garden establishment was maximum and only from 2004 to 2005 it was downward. Maximum of RG established in 2011 and 2012 are found from the survey. The overall result shows that tendency of roof gardening is rising consequently (Fig. 4).

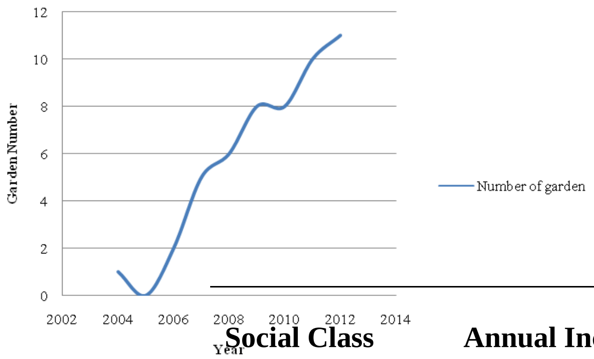

Fig. 4. Trend of roof gardening according to year.

ECONOMIC VALUATION

The analysis is done in order to have an insight whether RG can be considered as a supplementary economic activity in urban society. It is found that a large portion of the roof gardener belongs to middle class including upper middle class (55%) and lower middle class (21%). No roof garden is found in lower class households within the study area because they lack own residence and even if they possess, most of them are not *pucca* building i.e. not suitable for roof gardening (Table 1).

Table 1. Distribution of roof gardeners with respect to income

Social Class	**Annual Income (Tk.)**	**Percentage (%)**
Lower class	<81,000	0
Lower middle	82,000-3,30,000	21

Upper middle	3,40,000-10,00,000	55
High income	<10,00,000	24

Variable costs for roof gardening are the cost for weeding, hoeing, watering, manuring, insecticide, replanting etc. The study identifies the following variable cost needed for roof gardening (Table 2). The return from the gardening starts within three months after investment, very few people consider roof gardening commercially to get profit. The return varies due to size and number of species. This analysis is done on the basis of average production and current price derived from current financial market. It is found that on an average 10,150 (Tk./Yr) (Table 3) is derived from each roof garden against an average yearly cost of Tk. 4,440 (Table 2) excluding fixed cost. So, yearly neat profit from roof gardening is Tk.5,710 (i.e. 10,150 – 4,440) which is financially attractive as a small scale venture as no active labor is required.

TEMPERATURE REDUCTION

Result shows that maximum temperature was recorded in reference roof (roof without roof garden) and the minimum temperature was recorded in roof with roof garden. The mean temperature value of reference roof is 39.02^0C, top of the planting tray in roof garden is 33.42^0C and bottom of the planting tray is 31.15 ^{0}C. The reference rooftop recorded and reached temperatures between 26.60°C and 53.90°C during the test period. However, the roof directly bottom of the planting tray displayed a significantly lower temperature ranging from 22.70°C to 45.90°C. The thermometer placed on top of the planted tray also showed temperature reduction in comparison to the reference roof which ranges from 25.10° C to 45.70° C. At no stage during the five day period did the temperature for the readings under or on top of the planting trays reach the maximum temperature recorded on the reference roof. It is found that green roof reduce 6.7° C temperatures in comparison to reference roof (Table 4). According to Peck et al.,(1999) there is reflected heat that comes off rooftops, especially in flat concrete rooftops. Rooftop gardens can shield off as much as 87% percent of solar radiation while a bare roof receives one hundred percent exposure (Kohler and Keeley, 2005). Rooftop gardens (for example, Tokyo city), which is a highly dense city, have been constructed on apartments and it has reduced the heat generated by hardened surfaces (Dunnett and Kingsbury, 2004). Similarly, Wong et al., (2003) found that by constructing green rooftop gardens (for instance, Singapore), the solar radiation, external temperature, relative humidity and winds are slowed down.

Table 2. Variable costs for roof gardening

Types of Cost	Garden Size (sq.m)	Cost (Tk./Yr)	Average Cost (Tk./Yr)	Total Average Cost (Tk./Yr)
Watering	>24	540	900	4440
	25-48	900		

	<48	1260	
Fertilizer & insecticide	>24	830	
	25-48	1420	1370
	<48	1860	
Seed collection, planting & maintenance	>24	1600	
	25-48	2130	2170
	<48	2780	

Table 3. Financial return scenario in roof gardening

Types of Products	Garden (sq. m)	Min.	Max.	Mean Range (Tk/Yr)	Mean (Tk/Yr)	Mean Total Return (Tk/Yr)
Flower	>24	600	1560	723.64		
	25-48	900	2400	994.17	1156.77	
	<48	1530	4260	1752.50		
Fruits	>24	3540	5900	2441.82		
	25-48	4200	8200	3107.50	3648.73	10150.17
	<48	5800	11700	5396.88		
Vegetables	>24	3250	7500	2914.55		
	25-48	4570	11500	4361.25	4677.35	

	<48	5700	17500	6756.25	
Cutting & others	>24	1250	2050	433.64	667.32
	25-48	1400	2650	513.33	
	<48	1900	6870	1055.00	

Table 4 Descriptive statistics

Variables	**Min**	**Max**	**Mean**	**SD**
Reference Roof	26.60	53.90	39.0200	8.62277
Top of the planting tray	25.10	45.70	33.4250	5.88977
Bottom of the planting tray	22.70	45.90	31.1550	6.09499

CONCLUSION

Roof garden plays an important role in the mental well being of the gardener as well as in amelioration of the environment. It has also a promising potential as a small scale business that can accelerate additional family income, however, it may generate some employment facilities through its backward and forward linkages. The study concludes that green roof contributes to reducing energy consumption for passive cooling load of residential building. It is also found that green roof reduce 6.7° C temperatures in comparison to reference roof. Green as a passive cooling mean, its related thermal benefits are essential for architectural design strategy in warm-humid tropical climate of Bangladesh.

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MOBILE WASTE AND ITS IMPACT ON ENVIRONMENT: BANGLADESH PERSPECTIVE

A. Rahman[1] & P. Paul[1*]

[1] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong, Bangladesh.*
**Corresponding Author, <prasenjit.paul.pratik@gmail.com>*

ABSTRACT

Electronic related waste has become a growing concern all over the world as it poses threat to both human health and environment; Mobile waste is no exception to this. Being an overpopulated developing country the amount of mobile waste is increasing exponentially. This study targets at finding the amount of waste produced due to disposal of mobile phone and to study the effect of this waste in the environment in Bangladesh. To find out the amount of mobile phone waste and their impact on the environment critical observation and qualitative analysis were endeavoured. A mobile phone usually contains plastic, lead, bromine, dioxins, mercury, beryllium, cadmium, lithium etc. which are hazardous for environment. The amount of mobile phone waste being generated can be determined by the amount of mobile phone imported in a year and relating that with life cycle. Life cycle of the mobile phone can be determined by surveying people from different socio-economic aspects and ages. This paper highlights that there is an urgent need to address the issues related to mobile waste in Bangladesh to avoid future detrimental consequences.

Keywords: E-waste, Mobile waste, Environment, life cycle, PBT, Lead

INTRODUCTION

In recent decades with the revolution in information and communication technology (ICT) the world has experienced an enormous change in lifestyle, economy, industry and institutions. The growth of many different ICT products is tremendous regardless of both Developed and developing countries. With the use of these products the E-waste is also increasing. An estimation of 50 million tons of electronic waste is generated every year comprising more than 5 percent of total municipal solid waste. 3.4 millions Of E-waste is produced by USA and 12 millions in Asia .According to Environment and Social Development organization (ESDO) Bangladesh discards 0.3 million tons of e-waste every year (Hossain et al., 2010).

MOBILE WASTE

Mobile phones have low life cycles and frequently changed by many users within very short time. Use of mobile phones has grown exponentially, from the first few users in the 1970s, to 1.76 billion in 2004 (Sahu, Srinivasan, 2008) to recent 6.7 billion in 2013. Mobile phones sold during this year were 1.8 billion. Most of the cases the mobile phones are not properly recycled and find their way in the landfills. Only 3 percent of the mobile phones are recycled all over the world (Miah et al., 2013). Mobile waste consists of various heavy metals, plastics, flame retardants and other toxic materials which may pose harm to both human health and environment. Bangladesh is a developing country with a population of 142 million. According to BTRC the total number of mobile subscribers has reached 116 million. So, the number of mobile phone waste is also increasing. Bangladesh has generated 10,504 metric tons of toxics waste in cell phones alone in the last 21(Hossain et al., 2010) years.

LEGISLATION

Bangladesh has no legislation regarding the generation or management of the mobile waste. Bangladesh is a signatory of Basel convention on Trans Boundary movements of hazardous waste. The rules on handling of e-waste (which includes mobile waste) are being formulated by the Ministry of Environment and Forests (MOEF).

METHODOLOGY

To find out the total amount of mobile waste we collected both primary and secondary data. Primary data was collected from field survey. Questionnaire survey was conducted among people from different ages, profession and socio economic state. Secondary data was collected from corresponding authorities, NGO's, previously published journals. The collected data then summarized to determine the amount of mobile waste generated in Bangladesh and its effect in environment.

LIFE CYCLE OF MOBILE PHONES

Mobile phones are brought into market by the manufacturers and after using they become obsolete, some of them find their place in home desks or other storage and some find their way to garbage. Some are reused, some are recycled but most of them find their way to the disposal. Many users

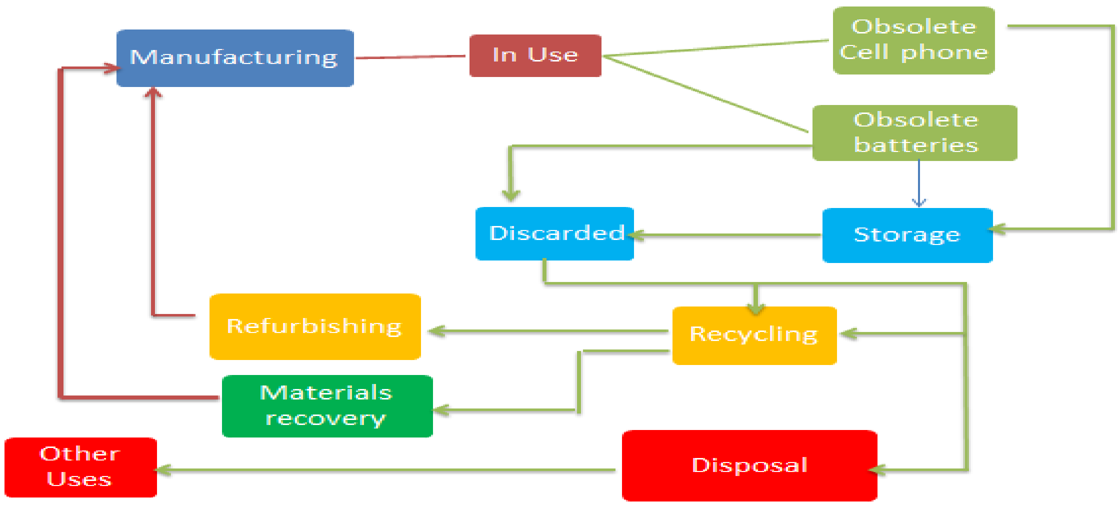

Fig.1: Life cycle of mobile phones

change their cell phone battery, they also find their way either recycling plant or disposal. The life cycle varies from country to country due to different socio-economy.

COMPONENTS OF MOBILE PHONE

A cell phone can contain over 40 elements heavy metals and persistent organic pollutants (POP), where 23% of their weight represents metal and rest are plastic and ceramic materials. The circuit boards and casings are made of plastics.

Constituents of mobile phones	Average weight per cent
Acrylonitrile butadiene Styrene (ABS-PC)	29%
Ceramics	15%
Cu and compounds	15%
Silicon plastics	10%

Epoxy	9%
Other Plastics	8%
Iron	3%
PPS	2%
Flame retardant	1%
Nickel and compounds	1%
Zinc and compounds	1%
Silver and compounds	1%
Al, Sn, pb, Au, pd, Mn, etc.	Less than 1%

Table1: chemical composition of mobile phone

The soldering of the circuit boards are made of heavy metals like Lead and Mercury, Mercury can also be found in the liquid crystal display. Another heavy metal cadmium can be found in infrared detectors and semiconductor chips. More metals like antimony, bismuth, chromium, copper, tin, cobalt, arsenic, Gold, Silver etc. consists in the mobile phone. Composition of mobile phone is given in Table 1.

The cell phone batteries consists a higher amount of lithium and also lead, cobalt chromium, thallium etc.

DETERMINING THE AMOUNT OF MOBILE WASTE BEING PRODUCED

To determine the life span of mobile phone we took a survey among people from different socio economic aspect and age. We ask them the following questions:

What is their age?
How many mobile phones do they use?
Which Brand mobile phones do they use?
How long they had used their last mobile phone?

From this survey it is found that people between 1-30 years change their mobile phones more frequently and prefer smart phones. Here the life span of the mobile phone is 1-1.5 years. People between 30-50 years likely to use both smart phones and Nokia brand (traditional) phones and the average life span of mobile phones is more than 2.5 years. And the people above 50 years prefer to use Nokia brand (traditional) phones. Here the life span is more than 5 years. And also people of different socio economic aspect life span of the mobile varies significantly. Because rich people change their handset more frequently than the others. After comparing all this data it is found that the average life span of the mobile is 3 years at present.

Table: 2 There is detail scenario of number of subscriber, imported mobile phones and their growth pattern. A rational analysis is done for preparing this table because all of these data was not found to the authority. Number of subscriber is found in BTRC (Bangladesh telecommunication Regulatory Authority) and number of imported mobile set from 2014-2010 is found from BMPIA (Bangladesh Mobile Phone Importers Association).Other value of the imported mobile was found from the rational analysis.

Year (1)	Number of subscriber (million) (2)	Increased Number of subscriber at the end of year (million) (3)	Increased Number of mobile phones at the end of year user (Million) (4)	Imported mobile set at the end of year (million) (5)	Total mobile set at the end of year (cumulative column 5) (6)
2001	0.520		0.520	0.54	.54
2002	1.075	0.555	0.4	0.56	1.1
2003	1.65	0.575	0.46	0.59	1.69
2004	2.771	1.121	0.6	0.65	2.34
2005	9.0	6.229	1.3	1.4	3.74
2006	19.13	10.13	2.2	2.3	6.04
2007	34.47	15.34	2.8	2.9	8.94
2008	44.64	10.17	3.5	3.8	12.74
2009	52.43	7.79	4	4.5	17.24
2010	68.65	16.22	5.9	6	23.24
2011	85	16.35	9.7	10	33.24
2012	97.18	12.18	11.5	13	46.24
2013	113.784	16.604	12	20	66.24
					Total 66.24

Table2. Calculating total number of mobile used in past 12 years in Bangladesh

Majority of the subscriber uses more than one sim that's why increased number of subscriber (column 3) is always greater than increased number of mobile phone user (column 4).As demand of mobile set is always higher than present users of mobile phone considering this, imported mobile set (column 6) is tabulated. From the cumulative value of column 5 total number of mobile phones imported since 2013 is 66.24 million. Because of smuggling in average 20% of mobile phone comes to the country illegally. That's why adding 20% phones to 66.24 million. And the revised amount is 79.48 million. It is assumed as the life cycle of mobile phone is 3 years than after 2016 total number of unused mobile phones would be 79.48 million. As the average weight of a mobile phone is 80 gms, the total amount of mobile waste being generated every year is 640 tons

EFFECT ON ENVIRONMENT

When the elements of mobile phones are safely encased mobile phones dangers aren't much of an issue. Problems can occur when they find their way in disposal. They can leak and contaminate their immediate environment. Over time, the toxic chemicals of a landfill's mobile waste can seep into the ground and possibly entering the water supply or escape into the atmosphere, affecting the health of nearby communities and the workers working in the disposal site.

Cell phones consists hazardous materials of the class H11 (Toxic: delayed or chronic), H12 (Ecotoxic) and H13 as classified by the Basel Convention.

The Natural Resources Defense Council observes that lead, mercury and cadmium found in personal mobile phones can release dangerous toxins into our air and water when burned or deposited in landfills improperly. In these situations, there can be significant environmental impacts. For example, the U.S. Environmental Protection Agency reports that ecosystems near point sources of lead often demonstrate biodiversity loss, decreased growth and reproductive rates, and neurological effects in vertebrates.

While, considering The environmental impact of the mobile waste not only toxicity should be taken into concern but also its exposure to environment

Risk = Hazard* Exposure

Persistent Bioaccumulative Toxins (PBT)

Most of the materials found in the mobile phone waste are Persistent Bioaccumulative Toxins. PBTs are particularly dangerous because they do not degrade over long periods of time, and can easily spread and move between air, water, and soil, resulting in the accumulation of toxins far from the original point source of pollution. Because PBTs accumulate in fatty tissue of humans and animals, the toxins are gradually concentrated, putting those at the top of the food chain at the greatest risk.

According to the United States EPA, “PBTs are associated with a range of adverse human health effects, including damage to the nervous system, reproductive and developmental problems, cancer and genetic impacts.” Children are a particularly sensitive population adversely impacted by PBTs.

Lead

Lead, ranked as number one priority on the EPA’s original list of PBTs of concern. It can be released to the environment due to leaching in the soil or by incineration. Concentration of Lead in the environment is generally low, soils and fresh water sediments contains less than 30 mg/kg. Lead is highly toxic to humans as well as many animals and plants. Lead exposure is cumulative; the effects of exposure are the same whether through ingestion or inhalation, and some appear to be irreversible. It has some wicked effects like damage to the nervous system and blood system, impacts on the kidneys and on reproduction.

Lithium

Lithium’s high degree of chemical activity can create environmental problems. When exposed to water, which is present in most landfills, the metal can burn, causing underground fires that are difficult to extinguish. Cadmium in the battery from a single phone could contaminate 600,000 litres of water.

Beryllium

Another troubling toxin in cell phones is beryllium, usually used in beryllium-copper alloys to increase flexibility and strength in components that need to be capable of flexing, such as contacts and springs. It can cause a permanent scarring of the lungs, sometimes years after initial exposure, and can be fatal (Basel Action Network, 2004).

Cadmium

Cadmium for example is considered as the 7th most dangerous substance known to man. It is a toxic heavy metal that can harm humans and animals that ingest it. It is also carcinogenic.

RISK AT DISPOSAL AND INFORMAL RECYCLING SITES:

In Bangladesh the dumping site for electronic wastes are not separate. Dumping of mobile phone waste can cause serious health injury for the workers and scavengers on the dumping zone. There are no formal practices for recycling of the mobile waste but there are many spot in Bangladesh where informal recycling of e-waste (which include mobile phones) are held, locally they are known as bhangary shop. The workers don't have any safety measure [Ahmed, 2011] and longtime exposure can cause them chronic diseases.

RESULT

As previously stated the amount of mobile waste is 640 tons which is the 0.2% of total E-waste generated in Bangladesh. The total amount of mobile waste generated in last 12 years is although the amount is not much but contains hazardous materials as classified by Basel convention and can eventually cause harm to Environment and spread fatal diseases.

RECOMMENDATION

Obsolete mobile phones can be used as source of metal if recycled. It has been found that 300 grams of gold, 140 grams of platinum and palladium and 140 kilograms of copper can be recovered from approximately 1 ton of recycled mobile phones. So it would not only beneficial economically,, but also advantageous ecologically if mobile wastes are properly recycled. The Battery recycling process focuses on maximizing the recovery of cobalt and other metals such as copper from the batteries for resale. The remaining products can then be used in smelting works, cement factories and also as road building materials (Miah et al. 2013).

Separation of cell phone waste as well as other hazardous waste from other waste and careful processing should be provided, as it poses great threat to the ecosystem near the dumping site. This can be done by separate on site storage and handling of such waste.

Lack of clarity on the issue of e-waste as well as mobile waste rules to govern and effectively monitor the mobile waste recycling are some of the prime reasons for experts and members of civil society demanding a separate set of rules to guide and control these processes.

There should be more emphasize given on the safety of the workers related to the Informal recycling of e-waste sector, as it can bring about various critical diseases. Both governments and human rights organization should pay attention to this.

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CHARACTERIZATION OF LIGNOCELLULOSIC AGRICULTURAL WASTE MATERIAL FOR BIO-FUEL PRODUCTION

Md. Niamul Bari[1*]& Saiful Islam[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology*.
**Corresponding Author, <niamulbari@yahoo.com>*

ABSTRACT

Enormous quantities of agricultural wastes are producing every year in Bangladesh. Most of them are lignocellulosic in nature, which contain cellulose, hemicellulose and lignin. For the production of any bio-product from these agricultural wastes through bioconversion, the contents of cellulose and hemi-cellulose are important. The rice straw, banana stem, banana leaf and corn cobs were taken as the experimental sample as these are abundantly available. Several laboratory experiments were conducted in order to characterize the lignocellulosic materials by determining the lignin, cellulose, hemicellulose and ash content. The results show that the cellulose contents are to be of 38.43%, 17.77%, 19.70% and 40.03% while, hemi-cellulose are to be of 29.93%, 12.55%, 12.05% and 34.23% in rise straw, banana stem, banana leaf and corn cobs, respectively. However, lignin and ash contents are varying from 13.70% to 16.10% and 5.83% to 15.90%, respectively for these agricultural wastes materials. Through this study it is evident that the corn cobs and rice straw might be the potential raw materials for the production of bio-ethanol with higher contents of cellulose and hemi-cellulose as well as lower contents of lignin and ash.

Keywords: Lignocellulosic, agricultural waste, characterization, bioethatnol, bioconversion

INTRODUCTION

Lignocellulosic material can be any material containing lignin and cellulose. Lignocellulosic materials (LM) are a natural, abundant and renewable resource essential to the functioning of industrial societies and critical to the development of a sustainable global economy. Most carbohydrates in plants are in the form of lignocellulose in which is made up of mainly cellulose, hemicellulose, pectin, and lignin. Lignocellulose is generally found, for example, in the stems, leaves, hulls, husks, and cobs of plants or leaves, branches, and wood of trees. The lignocellulosic materials can also be herbaceous material, agricultural residues, forestry residues, municipal solid wastes, waste paper, and pulp and paper mill residues. Agricultural wastes and in fact all lignocellulosics can be converted into products that are of commercial interest such as ethanol, glucose, and single cell protein (Kassebullah, 2006).

Lignocellulosic agricultural wastes are among the causes of environmental pollution. Their conversion into useful products may ameliorate the problems they cause. In many countries, these materials are generally used as animal feeds. There are various lignocellulosic agricultural wastes used for the production of cellulase enzyme. Rice husk and wheat straw are used for the production for lignocellulase enzymes specially cellulase enzyme (Masutti *et al*, 2012). There are some other lignocellulosic agricultural waste used for the production of cellulase enzyme like millet husks, banana peels, wheat bran, coir waste and saw dust were selected for the cellulase production (Jadhav *et al*, 2013).

Cellulases are a group of enzymes that break down cellulose into glucose monomers. Cellulase enzyme, having its importance due to major role in industrial applications. It is used for bioremediation, waste water treatment and also for single cell protein (Alam, 2005). It has also importance in food sciences like food processing in coffee, drying of beans for efficient purification of juices when used mixed with pectinases, paper and pulp industry and as a supplement in animal feed industry. This enzyme helpful for plant protoplast isolation, plant virus's investigations, metabolic and genetic modification studies (Shah, 2007). It has also pharmaceutical importance, treatment of phytobezons (a type of bezoar cellulose existing in humans stomach) and a key role in textile industry especially as its detergent applications to recover properties of cellulose related textiles and biofuels production from cellulosic biomass (Ali and Saad El-Dein, 2008). Cellulases producing fungi include *genra Aspergilli* (Ali and Saad El-Dein, 2008) *Aspergillus niger* and *Aspergillus terreus*, *Rhizopus stolonifer* (Pothiraj, *et al*., 2006) *Trichoderma, Penicillium, Botrytis Neurospora* etc. (Pandey, *et al*., 1999). Fungi are capable of decomposing cellulose, hemicellulose and lignin in plants by secreting multifarious set of hydrolytic and oxidative enzymes (Elzaher and Fadel, 2010).

Solid State Fermentation (SSF) is a way of fermenting substrate in the presence of excessive moisture in growth medium in spite of large amount of water being provided. SSF is an environmental friendly (less waste water production), low energy required and economical technology in synthesizing cellulase enzyme in response to submerged fermentation (Pandey *et. al*, 2003). SSF from last decade has made its importance in the production of value added products i.e., secondary metabolites, alkaloids, enzymes, organic acids, bio-pesticides (mycopesticides and bio-herbicides), biosurfactants, biofuels, aroma compounds, biopulping, degradation of toxic compounds, biotransformation, nutritional improvement of crops, biopharmaceuticals and bioconversion of agricultural waste (Pandey *et al*., 2000).

Lignocellulosic materials have also a great use to prepare ethanol. Biogas is prepared from lignocellulosic biomass. Production of ethanol from renewable sources of lignocellulosic biomass can improve energy security, decrease urban air pollution, and reduce accumulation of carbon dioxide in the atmosphere (Lynd *et al*, 1991). The production of ethanol from indigenous lignocellulosic biomass will stimulate new markets for the agriculture sector, thereby increasing domestic employment while reducing balance of payments deficits. The benefits of this unique technology can then be realized. Therefore, the aim of this study is to find out potential lignocellulosic agricultural biomass as a raw material for the production of bioethanol.

MATERIALS AND METHODOLOGY

Lignocellulosic materials contain lignin, cellulose, hemicellulose, ash content etc. The utilization can easily be identified by knowing the chemical composition of lignocellulosic materials. This section discusses about the materials and methods of determination of this chemical constituents.

MATERIALS

Widely cultivated and abundantly produced agricultural biomass viz. rice straw, banana stems, banana leaf and corn cobs are considered for characterization in this study.

Sample Collection

Lignocellulosic agricultural waste materials are collected from different parts of Rajshahi. Rice straw and corn cobs were collected from Meherchondi of Rajshahi city. Banana stem and leaf were collected from RUET campus of Rajshahi.

Sample Preparation

The collected samples were cleaned with brush and by washing. The cleaned samples were oven dried. The size of the samples were reduced with cutting mill and screened by passing through 1mm screen.

Experimental Procedure

All fibrous woody material mainly contains lignin, cellulose and hemi-cellulose. Cellulose and hemi-cellulose are the source of sugars, which might contribute to the production of bioethanol as carbon source. The characterization of collected lignocellulosic material is essential as these are considered as substrate for bioethanol production. The content of lignin, cellulose, hemi-cellulosed and ash were determined according to the following procedures.

Determination of Lignin Content

Acid insoluble lignin was determined according to TAPPI test method (T222 om-88). A known weight of 1 ± 0.001 gm oven dried (moisture free) sample was taken in a beaker. The sample was macerated with 15 ml of 72% (v/v) H2SO4 by gradual addition and stirring with glass rod. The solution was kept in water bath at 20±1oC for 2 hours and stirred frequently during this time. The material was then transferred from beaker to 1000 ml Erlenmeyer flask by rinsing with deionized water. The solution was then diluted by adding deionized water up to the mark of 575 ml to reach the final concentration of 3% H2SO4 acid. The solution was boiled for another 4 hours maintaining constant volume by refluxing. The insoluble material was allowed to settle by keeping overnight in inclined position. A portion of the supernatant was taken for the determination of acid soluble lignin content. The remaining portion was passed through the filtering crucible of known dry weight. The flask was washed with hot deionized water to collect all insoluble materials. The crucible with lignin was dried in an oven at 105±3oC to constant weight and the dry weight was taken after cooling in desiccators. The acid insoluble lignin was calculated based on oven-dried sample weight basis:

Acid insoluble lignin, (%) = $(W_3 – W_2)*100/W_1$
(1)

Where, W1 = dry weight of sample (1 g), W2 = dry weight of crucible and W3 = dry weight of crucible with lignin.

Acid soluble lignin was determined according to TAPPI Useful Method UM-250. A portion of the clear supernatant was collected during the determination of acid insoluble lignin. The sample was analyzed by using UV/VIS spectrophotometer at 205 nm wave length. Reference solution used was 3% (v/v) sulphuric acid. Acid soluble lignin was calculated as follows:

Acid soluble lignin (%) = 0.523AD/B
(2)

Where, A = absorbance, D = dilution factor and B = dry weight of sample (1 g). Finally, the total quantity of lignin was calculated by adding acid insoluble and acid soluble parts.

Determination of Holocellulose Content

Holocellulose content was determined according to method developed by Wise, et al. (1946). A known weight of oven dried (moisture free) sample was mixed in 250 ml Erlenmeyer flask with 100 ml distilled water, 1.5 gm sodium chlorite and 5 ml of 10% (v/v) acetic acid. The mixture was then heated in water bath at 70oC for 30 minutes. Subsequently, 5 ml of 10% (v/v) acetic acid was added and the solution was heated for another 30 minutes. After that, more sodium chlorite (1.5 gm) was added and the solution was heated for a further 30 minutes. The alternate addition of acetic acid and sodium chlorite, at intervals of 30 minutes, was continued until a total addition of 6 g sodium chlorite achieved. Finally, the solution was heated for another 30 minutes and cooled in an ice bath. The solution was filtered with a filtering crucible and washed with 300-400 ml distilled water followed by quick rinsing with acetone. The crucible with holocellulose was dried at 105±3oC to constant weight. The content of holocellulose was calculated based on oven-dried sample weight basis:

Holocellulose, (%) = $(W_3 – W_2)*100/W_1$
(3)

Where, W1 = dry weight of sample (1 g); W2 = dry weight of crucible; and W3 = dry weight of crucible with holocellulose.

Determination of Alpha-cellulose Content

Alphacellulose content was determined according to TAPPI test method T203os-61. A known weight of 2±0.001 gm oven dried (moisture free) holocellulose sample was macerated with 15 ml of 17.5% (w/v) sodium hydroxide solution in a water bath at 20oC for 1 minute with constant stirring (gentle). Further 10 ml of 17.5% (w/v) sodium hydroxide solution were later added and the content mixed for 45 seconds. This was repeated using an additional 10 ml of 17.5% (w/v) sodium hydroxide solution, mixed for a further 15 seconds and the resultant mixture were left to stand for 3 minutes. After this, an additional 10 ml of 17.5% (w/v) sodium hydroxide solution were again added and the content mixed for 2.5 minutes. This final stage was repeated 3 times after which the mixtures were left to stand for 30 minutes. A 100 ml of distilled water was then added, content mixed, and left to stand for a further 30 minutes. The mixtures were then filtered with vacuum filtration equipment. The solid portion was washed with 25 ml of 8% sodium hydroxide solution followed by rinsing with 650 ml of cold distilled water. Subsequently, the filtering holder was filled with 2N acetic acid and the acetic acid was removed under suction. Cold distilled water was then used to remove any residual acid from the sample prior to drying in a convection oven at 105oC till constant weight. Alpha cellulose content was calculated based on oven-dried weight of holocellulose sample:

$$Alphacellulose, (\%) = \frac{X}{Y} \times 100$$
(4)

where, X = dry weight of extract (g) and Y = dry weight of holocellulose sample (g).

Determination of Ash Content

Ash content was determined according to TAPPI test method (T211 om-93). A known weight of 2±0.001 gm oven-dried samples was taken in a porcelain crucible which dry weight was known and ignited in a muffle furnace with gradual temperature increment at 5oC per minute from 100oC (for 1 hour) to 300oC (for 2 hours) to finally 525oC (for 8 hours). The crucible was then allowed to cool to 100oC prior to place in a desiccator for further cooling to room temperature. The weight of ash with crucible was taken and recorded. The ash content was calculated based on oven-dried sample.

Ash content, (%) = $(W_3 – W_2)*100/W_1$ (5)

where, W1 = dry weight of sample (2 g), W2 = dry weight of crucible and W3 = dry weight of crucible with ash.

RESULT AND DISCUSSION

The experimental results of lignin content, hemicellulose, alpha cellulose and ash content of rice straw, banana leaves, banana stems and corn cobs are presented in the following sections. All experiments were carried out in triplicate. The lignin contents of all samples in percentage and their average value with standard deviation are presented in Table 1.

Table 1: Lignin content of rice straw, banana stems, banana leaves and corn cobs

Items	Lignin content (%)

	Sample 1	Sample 2	Sample 3	Average
Rice straw	14.6	13.6	12.9	13.7±0.85
Banana stems	16.8	15.7	15.1	15.9±0.86
Banana leaves	15.6	14.2	14.4	14.7±0.76
Corn cobs	15.3	16.6	16.4	16.1±0.70

The Table shows that the average lignin content of rice straw, banana stems, banana leaves and corn cobs are 13.7±0.85%, 15.9±0.86%, 14.7±0.76% and 16.1±0.70%, respectively. It is observe that the highest lignin content is obtained for corn cobs and the lowest lignin content is obtained for rice straw. However, the variation of lignin content for all these lignocellulosic material is not significant.

As the aim of this study to evaluate the potentiality of different agricultural wastes for the production of bioethanol through solid state bioconversion, the determination of cellulose content as carbon source is very important. The holocellulose contents of rice straw, banana stems, banana leaves and corn cobs have been determined in laboratory. The results of holocellulose content in rice straw, banana stems, banana leaves and corn cobs in percentage is presented in Table 2.

Table 2: Holocellulose content of rice straw, banana stems, banana leaves and corn cobs

Items	Holocellulose content (%)			
	Sample 1	Sample 2	Sample 3	Average
Rice straw	34.50	43.59	37.21	38.4±4.67
Banana stems	12.40	19.19	21.73	17.8±4.82
Banana leaves	7.98	19.85	19.55	15.8±6.77
Corn cobs	43.02	43.37	43.69	43.4±0.33

The results presented in Table 3 show that the holocellulose contents in corn cobs and rice straw are almost twice the holocellulose contents of banana stems and banana leaves. The holocellulose contents in corn cobs and rice straw are found to be of 43.4±0.33% and 38.4±4.67%, respectively. It is also observed that the variation of results among different samples of same material is not wide and it is varying within about 5% to 6%.

The cellulose is a major part of lignocellosic material which is almost 50% of the total material. The α-cellulose which is called true cellulose is composed of polysaccharide [(C6H12O6)n]. The fraction or quantity of this constituent is important for any metabolic product by microbial bioconversion. The percentage of α-cellulose content in rice straw, banana stems, banana leaves and corn cobs are given in Table 3.

Table 3: α-cellulose content of rice straw, banana stems, banana leaves and corn cobs

Items	α-cellulose content (%)			
	Sample 1	Sample 2	Sample 3	Average
Rice straw	26	35.43	28.37	29.9±4.91
Banana stems	10.1	12.34	15.228	12.6±2.57
Banana leaves	5.735	11.66	12.45	9.9±3.67
Corn cobs	37.19	36.53	39.086	37.6±1.33

The fraction of α-cellulose for corn cobs is showing the highest value of 37.6±1.33% among other sample tested. The second highest is shown for rice straw but α-cellulose for banana stems and banana leaves are very low. Although, the highest content obtained in corn cobs is lower than the average value of 50%.

Hemicelluloses, which occur in the cell wall, are heteropolysaccharides. These contain many different sugar monomers. In contrast, cellulose contains only anhydrous glucose. For instance, besides glucose, sugar monomers in hemicellulose can include xylose, mannose, galactose, rhamnose, and arabinose. Hemicelluloses include xylan, glucuronoxylan, arabinoxylan, glucomannan, and xyloglucan. Hemicelluloses contain most of the D-pentose sugars, and occasionally small amounts of L-sugars as well. Xylose is always the sugar monomer present in the largest amount, but mannuronic acid and galacturonic acid also present sometimes. The quantity of hemicellulose can be calculated by subtracting the quantity of α-cellulose from the quantity of holocellulose. The results are shown in Table 4.

Table 4: Hemicellulose content of rice straw, banana stems, banana leaves and corn cobs

Items	Hemi-cellulose content (%)			
	Sample 1	Sample 2	Sample 3	Average
Rice straw	8.5	8.16	8.84	8.5±0.34
Banana stems	2.3	6.85	6.5	5.2±2.53
Banana leaves	2.24	8.189	7.1	5.8±3.17
Corn cobs	5.833	6.84	4.6	5.8±1.12

The present study shows that the fraction of hemicelluloses obtained was 8.5±0.34% for rice straw is the highest value. The hemicelluloses content for other three materials are almost same which is varying from 5.2% to 5.8%. The total sugar content in corn cobs of 43.4±0.33% and in rice straw of 38.4±4.67% are represented by the quantity of holocellulose which means that about 434 gm and 384 gm of sugar could be achieved from 1 kg of corn cobs and rice straw, respectively if it is possible to hydrolyze completely. The residual part as ash content of these materials was determined and the results are presented in Table 5.

Table 5: Ash content of rice straw, banana stems, banana leaves and corn cobs

Items	Ash content (%)			
	Sample 1	Sample 2	Sample 3	Average
Rice straw	9.27	9.61	8.84	9.2±0.39
Banana stems	14.3	13.2	12.78	13.4±0.78
Banana leaves	6.3	7.3	6.2	6.6±0.61
Corn cobs	6.1	5.71	5.62	5.8±0.26

The experimental results shown in Table 2 depicted that the highest ash content of 13.4±0.78% is found in banana stem and the lowest ash content of 5.8±0.26% is found in corn cobs. However, the ash content of banana leaves is slightly more that the lowest. This variation of ash content in different parts of the same plat is might be due to the structural formation of the part of the plant. The ash content of rice straw is moderate (9.2±0.39%) compared to others.

The compositional characteristics of straw, banana stems, banana leaves and corn cobs is summarized and shown in Table 6.

Table 6: Ash content of rice straw, banana stems, banana leaves and corn cobs

Items	Constituent (%)				
	Lignin	Holocellulose	α-cellulose	Hemi-cellulose	Ash
Rice straw	13.7±0.85	38.4±4.67	29.9±4.91	8.5±0.34	9.2±0.39
Banana stems	15.9±0.86	17.8±4.82	12.6±2.57	5.2±2.53	13.4±0.78
Banana leaves	14.7±0.76	15.8±6.77	9.9±3.67	5.8±3.17	6.6±0.61
Corn cobs	16.1±0.70	43.4±0.33	37.6±1.33	5.8±1.12	5.8±0.26

The characterization study of different lignocellulosic agricultural materials shows that the average compositions of rice straw are 13.7±0.85% of lignin, 38.4±4.67% of holocellulose, 29.9±4.91% of α-cellulose, 8.5±0.34% of hemicellulose and 9.2±0.39% of ash content. Similarly, average compositions of corn cobs are 16.1±0.70% of lignin, 43.4±0.33% of holocellulose, 37.6±1.33% of α-cellulose, 5.8±1.12% of hemicellulose and 5.8±0.26% of ash content. On the other hand, the average compositions of banana stems and banana leaves are 15.9±0.86% and 14.7±0.76% of lignin, 17.8±4.82% and 15.8±6.77% of holocellulose, 12.6±2.57% and 9.9±3.67% of α-cellulose, 5.2±2.53% and 5.8±3.17% of hemicellulose and 513.4±0.78% and 6.6±0.61% of ash content, respectively. The comparison of different constituents of various agricultural lignocellulosic materials is presented in Figure 1.

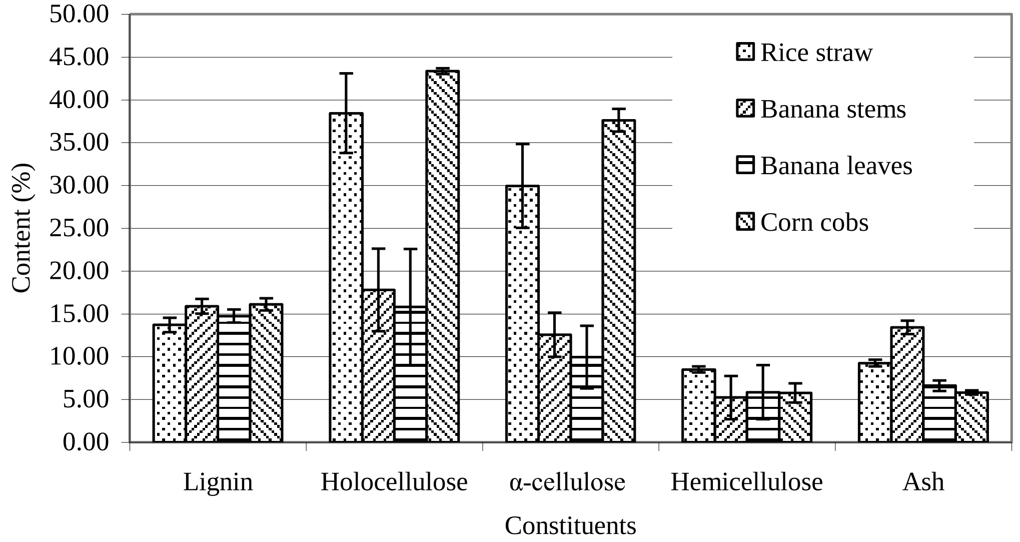

Figure 1: Comparison of lignocellulosic constituents of agricultural wastes

The Figure 1 shows that the lignin contents of straw, banana stems, banana leaves and corn cobs are around 15%, hemicellulose contents are mostly 5% except rice straw and ash content is varying within 5% to 13%. On the other hand, Holocellulose and α-cellulose content are varying in wide ranges among the materials. The holocellulose content of rice straw and corn cobs are about twice that of the holocellulose content of banana stems and banana leaves.

CONCLUSIONS

From the result it is clear that rice straw and corn cobs contain high amount of cellulose or hemicellulose and low amount of lignin. On the other hand banana stems and leaves contain low amount of cellulose and hemicellulose. Usually for the production of bio ethanol, cellulase enzyme, fermented sugar can be produced in the presence of high amount of cellulose and hemicellulose. Therefore, it can be said that rice straw and corn cobs can be used in the production of bioethanol, cellulase enzyme, fermented sugar and other bioconversion products. On the other hand banana stem and leaf are not suitable to use in the production of such bioconversion products.

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STANDARD METHODS FOR EVALUATING EARTHQUAKE INDUCED SOIL LIQUEFACTION POTENTIAL

M. S. Sarkar[1*] & S. M. Farooq[1]

[1]*Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong, Bangladesh.*
**Corresponding Author, <sakib.sarkar@gmail.com>*

ABSTRACT

Earthquake prone regions are susceptible to a very serious geotechnical hazard, termed 'soil liquefaction'. Soil liquefaction refers to the complete loss of shear strength in soil due to increase in pore water pressure during the event of an extreme vibration. Liquefaction causes devastation by inducing flow slides, lateral spreading, settlements, loss of bearing capacity, and creating sand boils. Hence a careful evaluation of liquefaction potential is very important. Different evaluation methods have been developed by different researchers following the disastrous events of liquefaction hazard that took place during the 1964 Nilgata, Japan and Alaska earthquakes. Seed and Idriss (1971) first proposed a method termed the 'simplified procedure' based on the cyclic stress approach which uses the Standard Penetration Test data. Since then this method has been modified and improved by several researchers. Also other methods have been developed based on the Cone Penetration Test (CPT), Becker Penetration Test (BPT), shear-wave velocity (Vs) measurements etc. This paper primarily reviews the following three standard methods: (1) the method based on SPT data proposed by Idriss and Boulanger (2006), (2) the method based on CPT data proposed by Robertson and Wride (1998), and (3) the method based on shear-wave velocity measurements proposed by Andrus and Stokoe (2000). All three methods have been critically reviewed and discussed in full with their respective procedural steps, various correction factors etc.

Keywords: Soil liquefaction; Standard Penetration Test; Cone Penetration Test; Shear-wave velocity.

INTRODUCTION

Soil liquefaction is a dangerous threat in terms of geotechnical aspects especially during the event of an earthquake. A moderate to major earthquake can cause enormous devastation if the liquefaction threat is not properly mitigated. Liquefaction occurs when intense vibrations from an earthquake or other force increases the pore water pressure in a saturated or semi-saturated, cohesionless soil mass to equalize the shear strength of the soil mass, and thus reducing the effective stress of the soil mass to an apparent zero. The soil mass losing all its strength and stiffness behaves like a liquid substance and is unable to carry any external load. If properly investigated soil liquefaction can be controlled and mitigated using different techniques e.g., compaction, stone columns, soil grouting etc. Hence to ensure the utmost safety of structures proper investigation should be conducted using the most standard methods. Numerous researches have been carried out by different researchers following the devastating liquefaction hazards happened during the 1964 Nilgata, Japan and Alaska earthquakes. Seed and Idriss first presented a standard method termed the 'simplified procedure' in 1971. The method was developed using SPT data from different sites. This method has been widely considered as the most standard method to determine liquefaction potential all over the world. This method has been further improved, updated, and developed by various researchers. In this review paper we will be reviewing the method proposed by Idriss and Boulanger (2006). Liquefaction potential evaluation using CPT data is much more reliable and consistent over SPT data because of its detailed and robust soil profiling. In this paper we will be reviewing the method proposed by Robertson and Wride (1998). Other methods have also been developed over the years e.g. methods using the BPT data, methods using the shear wave velocity (Vs) measurements. In this paper we will only review the method based on shear-wave velocity measurements proposed by Andrus and Stokoe (2000). Various correction factors, charts, and curves have been presented where necessary.

CYCLIC STRESS RATIO AND CYCLIC RESISTANCE RATIO

The two most important variables of liquefaction potential evaluation are Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR). Evaluation of liquefaction potential is followed by determining the Factor of Safety (FS) of consecutive soil layers. FS is defined as the ratio of Cyclic Stress Ratio (CSR) to the Cyclic Resistance Ratio (CRR).

$$\mathrm{FS} = \frac{\mathrm{CSR}}{\mathrm{CRR}} \tag{1}$$

If FS<1, then the layer is termed as non-liquefiable and if FS>1, then the layer is termed as liquefiable

Determination of CSR

The following formula proposed by Idriss and Boulanger (2006) is used to determine CSR:

$$\mathrm{CSR} = 0.65\,\frac{a_{max}}{g}\,\frac{\sigma_{v0}}{\sigma'_{v0}}\,r_d\,\frac{1}{\mathrm{MSF}}\,\frac{1}{K_\sigma}$$

0.65 is a factor to calculate the equivalent uniform stress cycles required to generate same pore water pressure during an earthquake; a_{max} is the peak horizontal acceleration at the ground surface generated by the earthquake; g is acceleration of gravity; σ_{v0} and σ'_{v0} are total vertical overburden stress and effective vertical overburden stress, respectively; r_d is stress reduction factor; MSF is the magnitude scaling factor, and K_σ is the overburden correction factor (Dixit et al., 2012).

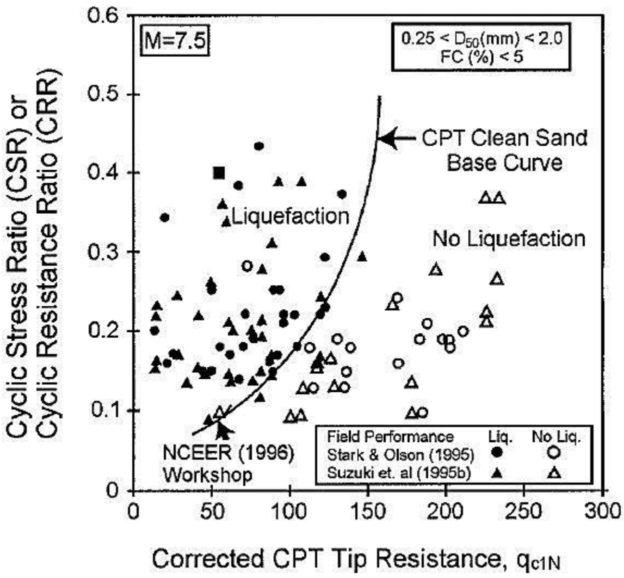


Fig. 2 Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data from Compiled Case Histories (After Robertson and Wride. 1998)

Fig. 1 SPT Clean Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (After Youd et al., 2001)

Stress reduction factor, r_d

The stress reduction factor r_d is calculated from the following expressions developed by Idriss and Boulanger (2006):

$$r_d = \exp[\alpha(z) + \beta(z)\,M_w] \tag{3}$$

$$\alpha(z) = -1.012 - 1.126\,\sin\left(\frac{z}{11.73} + 5.133\right) \tag{4}$$

$$\beta(z) = 0.106 + 0.118\,\sin\left(\frac{z}{11.28} + 5.142\right) \tag{5}$$

where z = depth below the ground surface (meters) and M_w is the moment magnitude of earthquake.

Magnitude scaling factor, MSF

Earthquake magnitude, M_w has to be adjusted to an equivalent CSR for an earthquake of magnitude, M_w = 7.5 through the use of magnitude scaling factor, MSF. MSF accounts for the duration effect on the initiation of liquefaction. The following relationship for determination of MSF for sands was given by Idriss (1999):

$$\mathrm{MSF} = 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058 \leq 1.8 \tag{6}$$

Overburden correction factor, K_σ

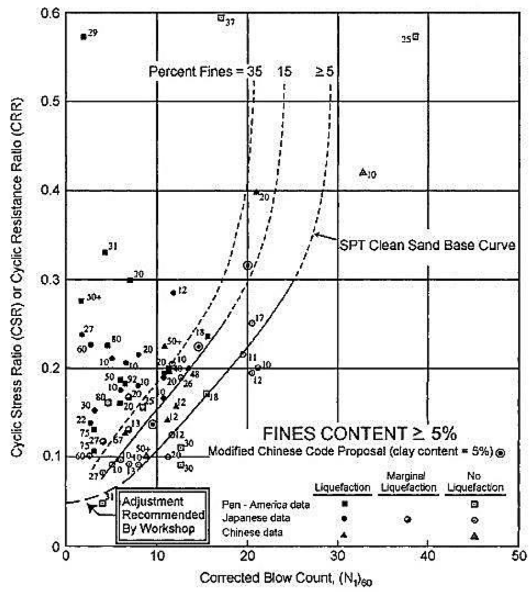
The overburden correction factor K_σ is used to adjust the value of CSR to an equivalent overburden pressure σ_{v0} of 100 kPa. K_σ can be determined from the following expressions (Boulanger and Idriss, 2004):

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right) \leq 1.0 \tag{7}$$

where

$$C_\sigma = \frac{1}{18.9 - 2.5507\sqrt{(N_1)_{60}}} \leq 0.3 \tag{8}$$

where P_a = Atmospheric pressure in kPa



Determination of CRR

The following formula was proposed by Idriss and Boulanger (2006) to determine the CRR value of cohesionless soil containing any fines content:

$$\mathrm{CRR} = \exp\left(\frac{(N_1)_{60CS}}{14.1} + \left(\frac{(N_1)_{60CS}}{126}\right)^2 + \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60C}}{25.4}\right.$$

where $(N_1)_{60CS}$ = clean sand standard penetration resistance

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60}$$

where $(N_1)_{60}$ = Corrected SPT value

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{11}$$

and $$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{\mathrm{FC}+0.1} - \left(\frac{15.7}{\mathrm{FC}+0.1}\right)^2\right) \tag{12}$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samplers with or without liners. $\Delta(N_1)_{60}$ is the correction for fines content (FC) in percent. FC = fines content in percent. The corresponding values of these corrections are listed in Table 1.

CPT

This CPT based liquefaction evaluation methods are more reliable and consistent than SPT based methods as the soil profiling through CPT is much more conclusive and detailed than through SPT. The CRR for clean sands (FC ≤ 5%) from CPT data can be determined by the curves in Fig. 2, which were proposed by Robertson and Wride (1998). Robertson and Wride (1998) also approximated the curves through the following expressions:

If $(q_{c1N})_{CS} < 50$, CRR$_{7.5}$ = 0.833[$(q_{c1N})_{CS}$/1,000] + 0.05 (13a)

If $50 \leq (q_{c1N})_{CS} < 160$, CRR$_{7.5}$ = 93[$(q_{c1N})_{CS}$/1,000]^3 + 0.08 (13b)

where $(q_{c1N})_{CS}$ = clean-sand cone penetration resistance normalized to approximately 100 kPa pressure.

Normalization of Cone Penetration Resistance

The normalization of cone tip resistance is achieved by the following equations:

$$q_{c1N} = C_Q(q_c/P_a) \tag{14}$$

where

$$C_Q = (P_a/\sigma'_{v0})^n \tag{15}$$

and where C_Q = normalizing factor for penetration resistance, values greater than 1.7 should not be used; P_a = 100 kPa or equivalent amount of pressure in the same units as used for σ'_{v0}; q_c = field cone penetration resistance measured at the tip; n = an exponent varying from 0.5 to 1.0 (Olsen, 1997).

Clean-Sand Equivalent Normalized Cone Penetration Resistance $(q_{c1N})_{CS}$

The following equation is used to correct the normalized penetration resistance (q_{c1N}) to an equivalent clean sand value $(q_{c1N})_{CS}$.

$$(q_{c1N})_{CS} = K_c q_{c1N} \tag{16}$$

where K_c is the correction factor for grain characteristics. It is expressed by the following equation proposed by Robertson and Wride (1998):

for $I_c \leq 1.64$, $K_c = 1.0$ (17a)

for $I_c > 1.64$, $K_c = = -0.043 I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75\, I_c - 17.88$ (17b)

where I_c is termed as the soil behavior type index and is calculated from the following expressions:

$$I_c = [(3.47 - \log Q)^2 + (1.2 + \log F)^2]^{0.5} \tag{18}$$

where

$$Q = [(q_c - \sigma_{v0})/P_a]\,[(P_a/\sigma'_{v0})^n] \tag{19}$$

$$F = [f_s/(q_c - \sigma_{v0})] \times 100\,\% \tag{20}$$

The value of exponent n can be used as 1.0 for clayey soils; 0.5 for clean sands, and a mid-value between 0.5 and 1.0 for silts and sandy silts (Robertson, 1990).

V_s

Shear-wave velocity, Vs method is useful for soils which are difficult to penetrate using SPT and CPT. Andrus and Stokoe (1997) proposed the following relationship between CRR and V_{s1}. A curve proposed by Andrus and Stokoe (2000) for this purpose is shown in Fig. 3.

$$\mathrm{CRR} = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right) \qquad (21)$$

Where V_{s1}^* = limiting upper value of V_{s1} for liquefaction occurrence; and a and b are curve fitting parameters.

$$V_{s1} = V_s\left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} \qquad (22)$$

Where V_{s1} = normalized shear wave velocity corrected for overburden stress
V_s = field shear wave velocity

The values of *a* and *b* used to draw the curves were 0.022 and 2.8, respectively. Values of V_{s1}^* were assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less (Youd et al., 2001).

Factor	Equipment variable	Term	Correction
Overburden pressure	-	C_N	$(P_a/\sigma'_{v0})^{0.5}$
Overburden pressure	-	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5-1.0
Energy ratio	Safety hammer	C_E	0.7-1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8-1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.80
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling	Standard	C_s	

method	sampler		1.0
Sampling method	Sampler without liners	C_s	1.1-1.3

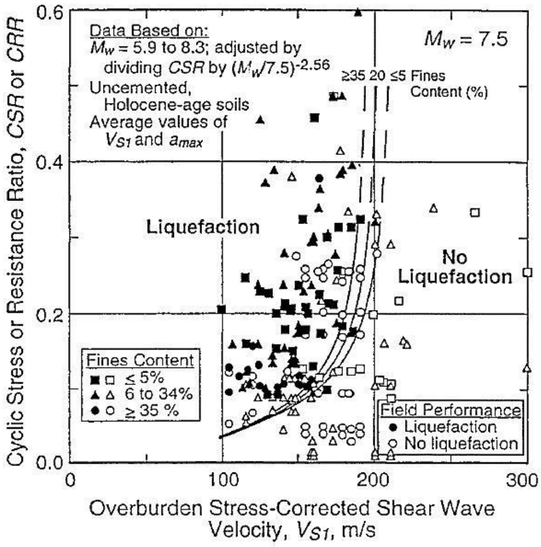


Fig. 3 Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (After Andrus and Stokoe, 2000)

CONCLUSION

Table 1 Corrections to SPT (Modified from Skempton, 1986) as Listed by Robertson and Wride, 1998 (After Youd et al., 2001)

All the three methods reviewed here have their relative advantages and limitations. The SPT has the largest compilation of case data histories from all over the world. The SPT can also be helpful to obtain disturbed soil samples which can be used for the determination of fines content and other important grain properties. The CPT provides the most continuous and detailed profile of the subsurface soil strata. The V_s measurements provide valuable information about the small-strain soil characteristics which can be used for higher level analyses beyond liquefaction evaluation. This method is also helpful at sites where SPT and CPT are difficult to perform. However, the best method should be chosen based on specific requirements viz. soil characteristics, site conditions, accuracy, consistency, ease of use, and economy etc. The correction factors should be carefully applied to obtain the most accurate results.

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A NUMERICAL STUDY ON THE EFFECT OF SURCHARGE ON THE STABILITY OF SLOPE

M. M. Sazzad[1] & M. M. Moni[1*]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi-6204, Bangladesh.*
**Corresponding Author, <mmsruet@gmail.com>*

ABSTRACT

In usual practice, the stability of slope is calculated by limit equilibrium methods (LEM). Now-a-days, finite element method (FEM) is gaining increasing popularity in slope stability analysis. This paper presents the capability of FEM in the stability analysis of slope. The stability of slope has been analyzed for homogeneous soil by FEM. The factor of safety has been computed by using the conventional methods such as Bishop (1955), Fellenius (1936) and Spencer (1967). The results obtained from these conventional methods have been compared with the calculations performed by FEM. After performing several calculations, it has been found that, for a simple homogeneous soil, factor of safety obtained from numerical method is almost same as that obtained from the conventional methods. It is also depicted that the surcharge has noticeable effect on the stability of slope. Factor of safety increases when the distance of surcharge increases from the crest of slope up to a certain level and then the effect of surcharge remains constant for a homogeneous slope. The failure surfaces for both FEM and LEM have also been reported.

Keywords: Slope stability, Factor of safety, Finite element method, Limit equilibrium method.

INTRODUCTION

Slope stability analysis is one of the most important areas of interest in geotechnical engineering. The disasters of landslide and collapse often occur in different countries of the world. On the contrary, there are a lot of engineering structures which require foundation systems to be placed near an existing slope such as bridge abutment, tower footings, basement construction of high rise building, etc. Therefore, it is of great significance to study the problem of slope failure and to know whether the slope is stable or not. Conventional limit equilibrium techniques are the most commonly used analysis methods. Cala and Flisiak (2003) performed many simulations for homogeneous and isotropic slopes using LEM and FDM. They computed the effect of slope height and slope angle on the factor of safety of slope for isotropic and homogeneous soil. Namdar (2010) studied several factors like slope geometry and gradient, geologic materials, stratigraphy, hydrology, and the local effect of shore process using LEM. Duncan (1996) depicted that the stability and deformation of slope can be analyzed by FEM. Griffiths and Lane (1999) discussed several examples of FEM based slope stability analysis with comparison against other solution methods. Totsev and Jellev (2009) performed a case study of a 56 meters high slope and compare the LEM results with FEM. He and Zhang (2012) described the stability analysis of a homogeneous slope and showed that equivalent area circle Drucker-Prager yield criterion was suitable for the stability analysis of slope. However, conventional methods have several limitations. With the rapid development of computing efficiency, several numerical methods, the finite element method (FEM), the boundary element method (BEM), the discrete element method (DEM), and the finite difference method (FDM) are widely used in the stability analysis of slope. Of these methods, the finite element method is the most widely used method. The advantage of finite element approach in the analysis of slope stability problems over limit equilibrium methods is that no assumption needs to be made in advance about the shape or the location of the failure surface, the slice side force and their directions, FEM is able to monitor progressive failure and also gives information about deformation at working stress level (e.g., Griffiths and Lane 1999). Even though many researches have been carried out for the stability analysis of slope by LEM and FEM for homogeneous soil, a few studies is found for the effect of surcharge on the stability analysis of slope. Consequently, the objective of this paper is to demonstrate the effect of surcharge on the stability of slope, to compare the FEM based stability results with limit equilibrium methods (LEM), to examine the mode of slope failure and compare with limit equilibrium methods.

GEOMETRIC MODEL OF SLOPE

Fig. 1 shows the model slope used in the present study for a homogeneous soil. In this model, β, W and x represent the angle of the slope, the width of the surcharge and the distance of surcharge from crest of the slope, respectively. Here, x and β are variable and W is constant. The soil parameters used in this study is presented in Table 1. Mohr coulomb yield criterion and Drucker-Prager yield criterion have been used. The finite element model of slope is shown in Fig. 2.

Table 1 Soil parameters used in this study

Soil Parameters	Value
Cohesion, c (kN/m^2)	10
Friction angle, ϕ (°)	18
Unit weight, γ (kN/m^3)	20
Modulus of elasticity, E (MN/m^2)	8
Poisson's ratio, υ	0.3
Dilation angle, ψ (°)	0

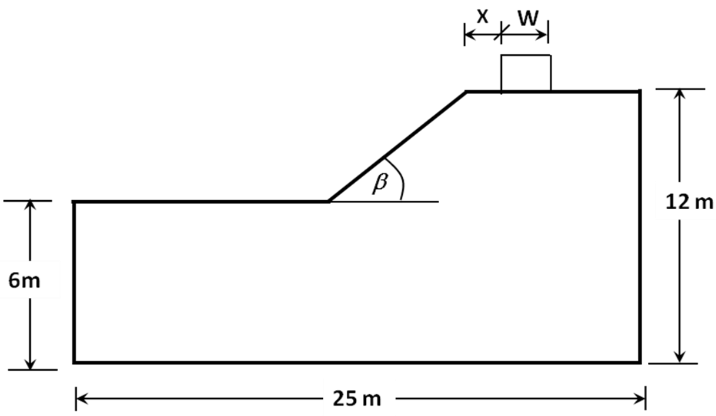

Fig. 1 Geometric model for homogeneous soil used in the study

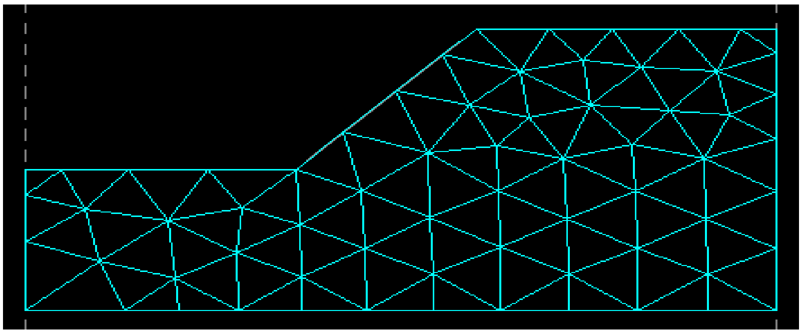

Fig. 2 Geometry and mesh for a homogeneous slope with surcharge

SOLPE STABILITY ANALYSES BY LEM AND FEM

The software GEO5 (2013) has been used for both limit equilibrium analysis and finite element analysis. For LEM, the geometric model (Fig. 1) is incorporated in the GEO5. After incorporating the model, the properties of soil are assigned for the specified interface. In the analysis stage, a slip surface is added. The slip surface may be circular or polygonal. In this paper, circular slip surface is used. After assigning all properties and slip circle, the analysis method is chosen. In this study, the methods proposed by Bishop (1955), Fellenius (1936) and Spencer (1967) are used. Then, analysis type is chosen which includes standard and optimization. In this paper optimization type is taken. Finally, a surcharge is added on the terrain of slope and analysis is carried out.

For stability analysis using FEM, the analysis type is first set. The geometric model (Fig. 1) is incorporated in the GEO5 same as in LEM. After incorporating the model, the properties of soil is assigned for the specified interface. The material model is to be set, which includes elastic, elastic modified, Mohr-Coulomb, modified Mohr-Coulomb, Drucker-Prager etc. In this paper, Mohr-Coulomb and Drucker-Prager model are used. For FEM analysis, meshes are generated. After generating the meshes, a strip surcharge is added on the terrain of slope. Finally, analysis is performed using strength reduction method (Griffiths and Lane, 1999).

RESULTS AND DISCUSSION

The geometric boundaries are horizontally constrained on the left and right sides and completely fixed at the bottom of the geometry. Factor of safety between FEM and LEM is compared and shown in Table 2. Factor of safety by Fellenius method (1936) is lower than that by Bishop (1955) and Spencer (1967). Drucker-Prager model shows greater factor of safety than that of Mohr-Coulomb model and LEM results are close to FEM for a homogeneous 45° slope when the ratio of x/W =2. Fig. 3 shows the effect of surcharge on the factor of safety of slope with LEM and FEM (β =45°). The figure depicts that Fellenius method (1936) represents lower factor of safety than that of Bishop (1955) or Spencer (1967) irrespective of the position of surcharge for homogeneous soil. Note also that the factor of safety is a function of the position of surcharge varying from the crest of the slope. However, it does not depend on the position of surcharge varying from the crest of the slope when x/W >3 for homogeneous soil. It is also observed that the factor of safety by Bishop (1955) and Spencer (1967) method are almost same. Fig. 4 shows the comparison of factor of safety for various slope angles with surcharge (x/W =0) and without surcharge. The instability of slope increases when a surcharge load is applied to the slope. The trend of the reduction of factor of safety with increasing slope angle for surcharge and without surcharge condition is similar. Fig. 5 shows the effect of slope angle on the factor of safety of slope with LEM and FEM. The values of factor of safety decrease with the increases of slope angle. Fig. 6 shows contours of the equivalent plastic strain for homogeneous slope by finite element method while Fig. 7 depicts failure of a homogeneous slope with surcharge using limit equilibrium method. It is obvious from Figs. 6 and 7 that slip surfaces obtained from FEM are localized deeper than LEM.

Table 2 Comparison of factor of safety between FEM and LEM (x/W =2, β =45°)

Material models and methods	Factor of safety
Drucker-Prager	1.28
Mohr-Coulomb	1.20
Bishop	1.09
Fellenius	1.05
Spencer	1.09

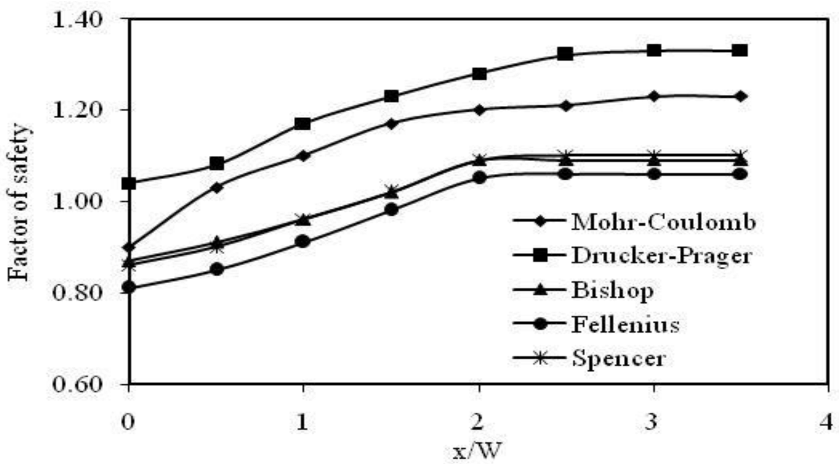

Fig. 3 Effects of surcharge on the factor of safety of slope with LEM and FEM (β = 45°)

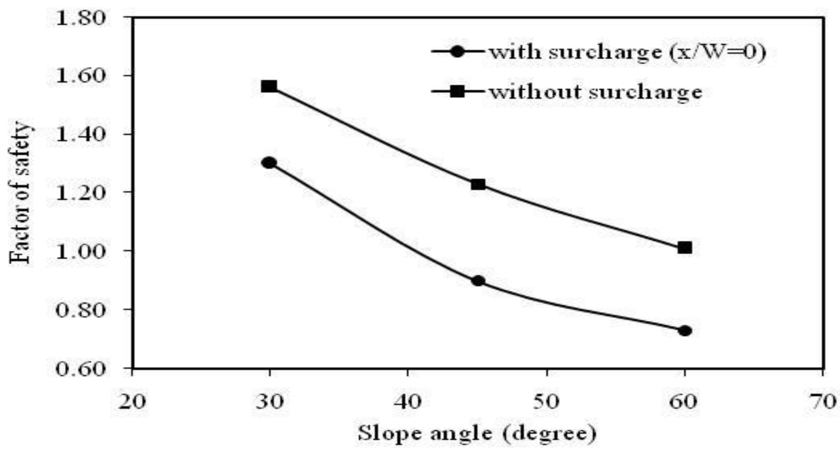

Fig. 4 Comparison of factor of safety for various slope angles with surcharge (x/W =0) and without surcharge

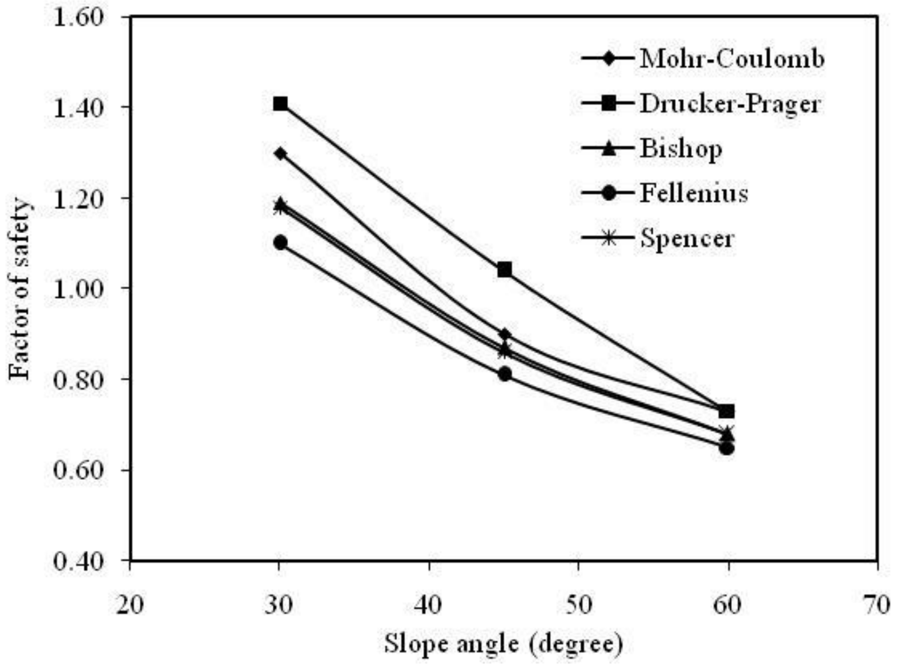

Fig. 5 Effect of slope angle on the factor of safety of slope with LEM and FEM

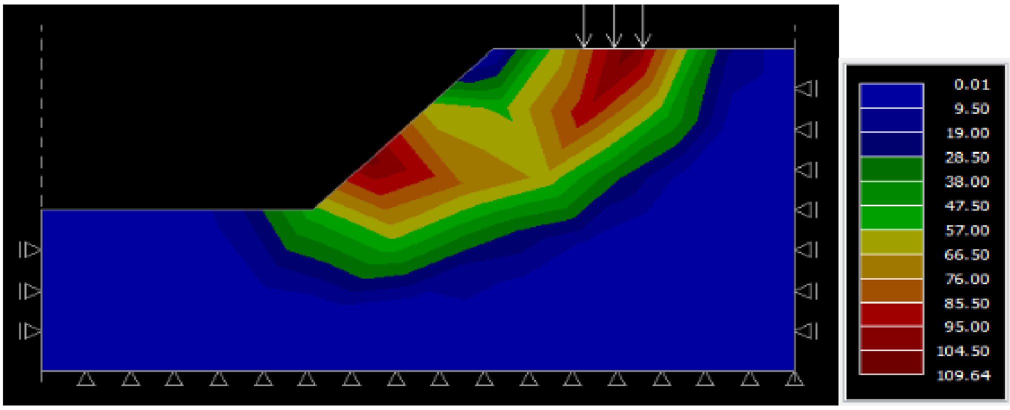

Fig. 6 Contours of the equivalent plastic strain for homogeneous slope by finite element method

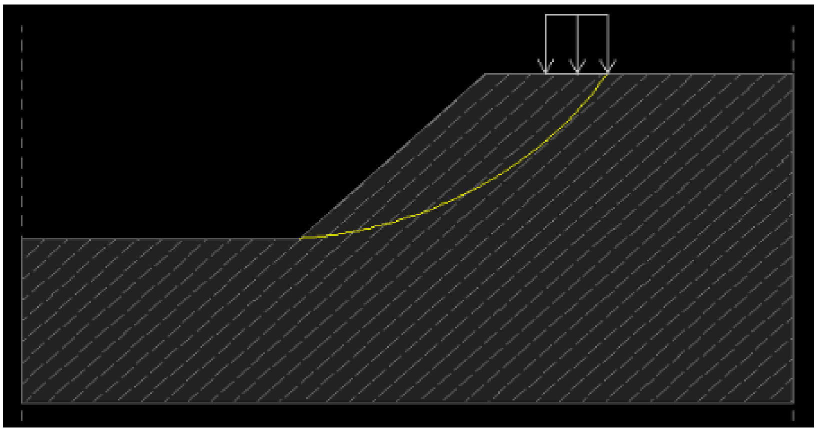

Fig. 7 Failure of a homogeneous slope with surcharge using limit equilibrium method

CONCLUSION

A numerical investigation is carried out to study the effect of surcharge on the stability of a simple homogeneous slope both by the conventional limit equilibrium methods and the finite element method. The major findings of the study are as follows:

I. Fellenius (1936) method depicts less factor of safety than that of Bishop (1955) and Spencer (1967) irrespective of the position of surcharge for a homogeneous soil.

II. Factor of safety is a function of the position of surcharge varying from the crest up to x/W less than 3 for a homogeneous soil.

III. FEM shows greater factor of safety than that of LEM and Mohr-Coulomb model depicts factor of safety less than that of Drucker-Prager model irrespective of the position of surcharge for a homogeneous soil.

IV. FEM depicts deeper localization of slip surface than LEM.

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INTRODUCE TO SOIL PROFILE OF BANGLADESH

U. Saha[1]*, S. K. Das[2], F. Rukshana[1] & S. R. Chowdhury[1]

[1] Geotechnical Research Directorate, River Research Institute, Faridpur, Bangladesh.
[2] River Research Institute, Faridpur, Bangladesh.
**Corresponding Author, <umasaha_65@yahoo.com>*

ABSTRACT

Knowing soil parameter is a conventional means for design and foundation and it is likewise important wherever the structure is to be constructed and what type of structure is to be built on. With a view to make available the information on foundation soils to the design engineer, a study is made to investigate the engineering soil parameters around the country. Accordingly, an effort is made on Bangladesh soils focusing soil profile along with its physical and index properties and to disseminate it to the geotechnical engineers/foundation engineers. The data have been collected from the Soil Mechanics Division of River Research Institute by testing of relevant engineering parameters of soil at different locations of Bangladesh. As River Research Institute has been testing disturbed and undisturbed soil samples collected mainly from the Bangladesh Water Development Board through accomplishment of soil borings by the geologist of Bangladesh Water Development Board throughout the country. The findings of the study are in general that cohesive and non-cohesive soil layers exists almost every region of the country. However, exceptions have also found in different locations and different layers. Soil profiles and geotechnical properties are expected to provide a comprehensive idea for the selection of appropriate measures to the respective zone and if necessary to take proper decision by the design engineers. The findings of this paper might also help the design engineer to get a preliminary concept about the soil of different regions of the country.

Keywords: Bangladesh, soil, profile, engineering parameters, foundation.

INTRODUCTION:

Geologically, Bangladesh is a part of the Bengal Basin, one of the largest geosynclines in the world (Source: Sajjadur, 2008). The Bangladesh landmass has gone through several historical ages to arrive to this present formation. The major divisions of these ages are the PreCambrian, the Paleocene, the Eocene, the Miocene, the Pliestocene and the Holocene ages.Table-1 shows the ages, litho logy and characters of those formations and their accessible locations.

Table 1: The Lithology of Bangladesh

Ages	Formation	Lithilogy & Character	Location
Holocene	Alluvium	Aquifers of Sand Silt & Clay	Flood Plains
Pliestocene	Modhupur	Red Clay, Ferruginous Nodules	Barind
Pliocene	Dihing	Sandstone	Deep
	Dhupi Tila	Aquifer of Sandstone and Clay	Deep
Miocene	Tipam	Aquifer of Sandstone and Clay	Very Deep
Miocene	Bokabil	Gas Producing	Eastern
	Bhuban	Sandstone and Shale	Folds

Ages	Formation	Lithilogy & Character	Location
Oligocene	Barail	Sandstone	Sunamgonj
Eocene	Kopili	Shale, Fossiliferous Sandstone	Sunamgonj
	Sylhet	Exposed Sandstone, Limestone	Sunamgonj
Paleocene	Tura	Exposed Sandstone	Sherpur
Jurassic	Rajmahal	Volcanic rock of Rajmahal Trap	Bogra
Permian	Gondwana	Sandstone, Shale and Coal beds	Dinajpur
PreCambrian	Basement	Igneous & Metamorphic rocks	Dinajpur

(Source : Haque, 2008)

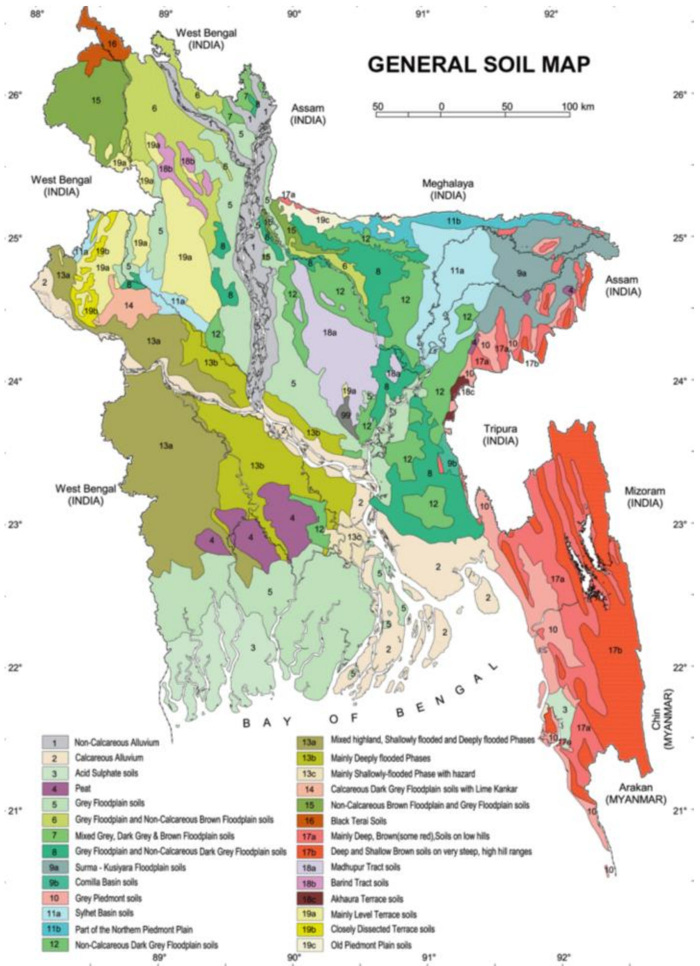

Fig.1 GENERAL SOIL MAP OF BANGLADESH (Source: Internet, 2014)

Bangladesh is a developing country and various structures of constructions are being built-up on it. Usually a typical geotechnical engineering project begins with a review of project needs to define the required material properties. Then follows a site investigation of soil, rock, fault distribution and bedrock properties on and below an area of interest to determine their engineering properties including how they will interact with, on or in a proposed construction. Site investigations are needed to gain an understanding of the area in or on which the engineering will take place. Subsurface exploration usually involves in-situ testing (one common example of in-situ test is the standard penetration test). In addition site investigation will often include subsurface sampling and laboratory testing of the soil samples retrieved (Source: Geotechnical Engineering, Wikipedia, 2014).However, it is a fact that, properties of engineering are parallel to properties of indexes and classification of soils. Thus, an engineer, by knowing how to classify the soil, can know how to solve the problems under building loads at construction (Source: Gokoglu and Gunaydin, 2011)

Under considering the above mentioned, a study is done by testing of relevant engineering parameters of vertical soils of different locations of Bangladesh. As vertical soils are related to depth, particle size, plasticity, moisture content, strength and settlement who are the utmost important parameters among numerous parameters and they provide the comprehensive idea to complete the design work of foundation of structures. So, they are considered for introducing the soil profile of Bangladesh. By this depths and parameters, soil strata will focus which will assist the design engineers. Or it may develop a concept on vertical soils of Bangladesh and the findings may support to decide the direct planning and design of construction if the foundations are not heavy.

MATERIALS AND METHODS

In connection with the soil profile and its physical and index properties, data have been collected from Soil Mechanics Division (SMD) of River Research Institute (RRI).

As a material, whole vertical soil is considered at subsequent depth of different locations of Bangladesh. Field survey data and testing data are collected regarding this connection. Field data are collected from boring logs of Bangladesh Water Development Board (BWDB) as they sent the soil samples to RRI with survey information through the delivered boring logs and soil samples. Geologists of BWDB collected the disturbed soil samples in the polythene bag and undisturbed soil samples in the Shelby tube recording SPT value and ground water level.

Skilled technicians and scientists of RRI tested the samples with great care. Calculations are completed through the conventional equations and plotting the calculative results in the graph from where required parameters are found out.

In this paper, a limited number of parameters are considered for introducing soil profile. Such as, natural moisture content, liquid limit, plasticity index, SPT value, particle size etc. As preliminary idea about soil strength is developed through SPT value and settlement character is determined through the compression index C_c. The compression index of cohesive soil is related to its liquid limit. Terzagi and Peck gave the following empirical relationship,

a) For undisturbed soils, $C_c = 0.009(w_L - 10)$

b) For remoulded soils, $C_c = 0.007(w_L - 10)$

Where $w_L =$ liquid limit (%) (**Source**: Arora,2010)

The soil type, colour and depth are also highlighted in the table. As vertical soil is being indicated soil profile. Relation of Consistency of Clay, Number of Blows N on sampling spoon and unconfined compressive strength q_u in tons/ft^2 and kN/m^2 are given below;

Consistency	Very soft	Soft	Medium	stiff	Very stiff	Hard
N	0-2	2-4	4-8	8-15	15-30	>30
q_u	<0.25/23.94	0.25-0.50/47.88	0.50-1.00/95.76	1.00-2.00/191.52	2.00-4.00/383.04	>4.00

Relative Density of Sands according to Results of Standard Penetration Test

Relative density	**Very loose**	**loose**	**Medium dense**	**dense**	**Very dense**
SPT (N)	0-4	4-10	10-30	30-50	Over 50

(**Source**: Peck, 1967)

RESULTS AND DISCUSSION

To schematize the soil properties of Bangladesh in brief, the area is divided into eight zones such as, Rangpur, Rajshahi, Dhaka, Sylhet, Comilla, Chittagong, Barisal & Khulna zones respectively.

Rangpur zone is consisted of Panchagarh, Thakurgaon, Nilphamari, Lalmonirhat, Kurigram, Dinajpur,Rangpur districts. Rajshahi zone is consisted of Gaibandha, Jaipurhat, Naogaon, Chapai Nawabgonj, Rajshahi, Bogra, Sirajgonj, Natore , Pabna districts.. Dhaka zone is consisted of Sherpur, Jamalpur, Netrokona, Mymensingh, Kishoregonj,Tangail, Gazipur, Narsindi, Dhaka, Narayangonj Manikgonj, Munshigonj, Shariatpur, Madaripur, Gopalgonj, Faridpur, Rajbari districts. Sylhet zone is consised of Sylhet, Sunamgonj, Habigonj,Maulovibazar districts.Comilla zone is consisted of Comilla, Chandpur, Brahmanbaria, Feni, Laxmipur, Noakhali districts. Chittagopng zone is consisted of Chittagong , Rangamati, Khagrachari, Bandarbon, Cox'sbazar districts. Barisal zone is consisted of Barisal , Jhalokathi, Bhola, Pirojpur, Patuakhali and Borguna districts.Khulna zone is consisted of Kushtia, Meherpur, Chuadanga, Jhenaidah,Magura, Narail , Jessore, Khulna, Bagerhat, Sathkhira districts.

Relevant parameters such as depth, soil type, colour, natural, moisture content, plasticity, SPT value, particle size, stiffness and relative density of soils are presented inTable-2 and some comparison graph of SPT, plasticity index, natural moisture content and particle sizes with depth has presented in Fig.2-Fig.7respectively. It has been discussed the results of the zones herewith accordingly.

In Rangpur zone, brown and grey in colour cohesive and non-cohesive soil layers are observed.Cohesive soil layers are found upto the maximum depth of about (0-43'), whose sizes vary about (0.0013-0.074)mm and stiffness varies from very soft to very stiff and then they are non-cohesive soils up to depth (43'-82') which sizes vary from (0.074-6.35)mm and the relative density varies from very loose to very dense. But exception has been found in Gaibandha District whose soil layers (0-72') are non-cohesive soils (Source: SMD,2008, 2009,2010).

In Rajshahi zone, brown and grey in colour cohesive and non-cohesive soil layers are observed.Cohesive soil layers are found upto the maximum depth of about (0-53'), whose sizes vary about (0.0013-0.074)mm and stiffness varies from very soft to very stiff and then they are non-cohesive soils up to the maximum depth of about 102,'whose sizes vary about (0.074-4.76)mm and the relative density varies from very loose to very dense. But exception has been found in Dinajpur District whose soil layers (0-72') are cohesive soils (**Source:** SMD, 2008, 2009, 2010, 2011).

In Dhaka zone, grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-34'), whose sizes vary about (0.001-0.074)mm and stiffness varies from very soft to very stiff and then they are non-cohesive soils upto the maximum depth of about 72,'whose sizes vary about (0.074-4.76)mm and the relative density varies from very loose to very dense. But exception has been found in Narayangonj (0-52'), Bausa Bill, Mirpur, Dhaka &Baganir Khal, Khaliajuri, Netrokona& Gopalgonj which contain organic soils (**Source:** SMD,2007, 2008,2009,2011,2011-12,2012-13).

In Sylhet zone, brown & grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found up to the maximum depth of about (0-72'), whose sizes vary about (0.001-0.074)mm and stiffness varies from very soft to very stiff. However, the occasional layers are found non-cohesive soils up to the depth of about 72,'whose sizes vary about (0.074-4.76)mm and the relative density varies from very loose to dense. But exception has been found in Tengrakhali & Kalaia khal, Zianagar contains organic soils **(Source:** SMD, 2007, 2008, 2009, 2011-12).

In Comilla zone, brown & grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found up to the maximum depth of about (0-64’), whose sizes vary about (0.001-0.074)mm and stiffness varies from very soft to hard and then they are non-cohesive soils up to the maximum depth of about 72,’whose sizes vary about (0.074-2.0)mm and the relative density varies from very loose to dense. But exception has been found in Daulatpur Khal, Fulgazi & Pathangor, Chagolnaya about (0-82') in Feni contains organic soils **(Source:** SMD, 2007, 2009, 2010) .

In Chittagong zone, brown & grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found up to the maximum depth of about (0-47’), whose sizes vary about (0.0013-0.074) mm and stiffness varies from very soft to very stiff and then they are non-cohesive soils up to the maximum depth of about 72,’whose sizes vary about (0.074-25.0) mm and the relative

Table-2: Showing the range of results of soil type, colour, natural moisture content, liquid limit, plasticity index, SPT value, particle sizes and stiffness/relative density.

Name of the Zone	Depth of Boring in feet (')	Soil Type	Colour	N. M.C in (%)*	LL in (%)*	PI in (%)*	SPT* value	Particle Size in mm	Stiff Relative
Rangpur	0-43	Cohesive	Brown & grey	19-58	27-68	5-47	0-32	0.0013-0.074	very sof stiff
	43-82	Non-cohesive	Brown & grey	FINE SAND mixed with varying amount of medium sand and silt and trace mica			2-60	0.074-6.35	very loo very der
Rajshahi	0-53	Cohesive	Brown & grey	15-56	20-73	5-38	0-26	0.0013-0.074	very sof stiff
	53-102	Non-cohesive	Brown & grey	FINE SAND mixed with varying amount of silt and medium sand and trace mica			2-60	0.074-4.76	very loo very der
Dhaka	0-34	Cohesive	Grey	15-210	28-86	6-58	0-19	0.001-0.074	very sof stiff
	34-72	Non-cohesive	Grey	FINE SAND mixed with trace medium sand and varying amount of silt and trace mica			1-90	0.074-4.76	very loo very der
Sylhet	0-72	Cohesive	Brown & grey	12-752	27-129	6-153	0-28	0.001-0.074	very sof stiff
	*Occasional layer	Non-cohesive	Brown & grey	FINE SAND mixed with varying amount of medium sand and silt and trace mica			3-45	0.074-4.76	very loo dense
Comilla	0-64	Cohesive	Brown & Grey	14-603	28-100	5-57	0-45	0.001-0.074	very sof
	64-72	Non-cohesive	Brown & Grey	FINE SAND mixed with varying amount of medium sand and silt and trace mica			3-49	0.074-2.0	very loo dense
Chittagong	0-47	Cohesive	Brown & Grey	17-180	27-118	4-60	2-17	0.0013-0.074	very sof stiff
	47-72	Non-cohesive	Brown & Grey	FINE SAND mixed with trace medium sand and varying amount of silt and trace mica			7-99	0.074-25.0	loose to dense
Barishal	0-72	Cohesive	Brown & Grey	18-72	30-58	9-45	1-18	0.0013-0.074	very sof stiff
	*Occasional layer	Non-cohesive	Brown & Grey	FINE SAND mixed with varying amount of silt and trace mica			3-49	0.074-2.0	very loo dense
Khulna	0-102	Cohesive	Brown & grey & black	18-485	27-133	5-79	0-57	0.001-0.074	very sof
	*Occasional layer	Non-cohesive	Brown & grey & black	FINE SAND mixed with varying amount of coarse sand, trace medium sand and varying amount of silt and trace			2-93	0.074-4.76	very l very

*N.M.C=Natural Moisture Content, LL=Liquid Limit, PI= Plasticity Index, SPT= Standard Penetration Resistance for Test

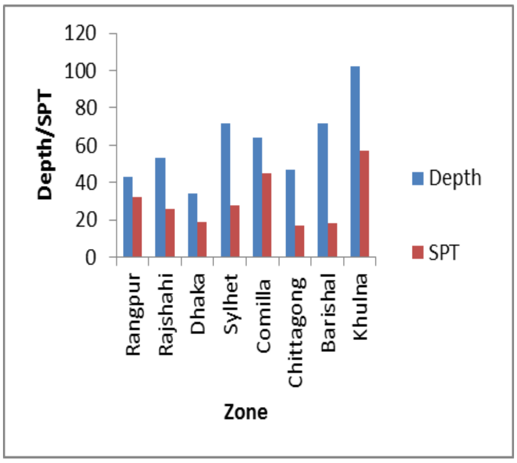

Fig.2: Comparison of Depth & SPT of Cohesive soils Zone wise

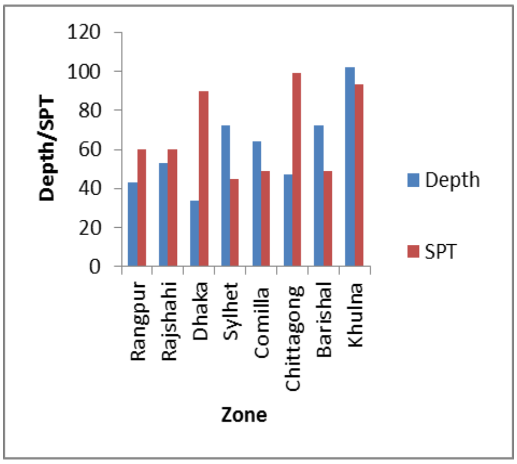

Fig.3: Comparison of Depth& SPT of Cohesive soils Zone wise

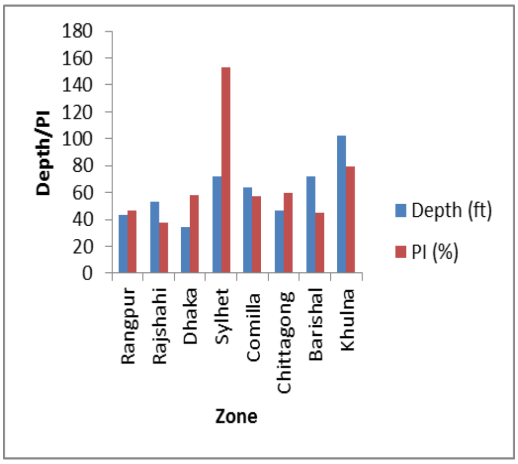

Fig.4: Comparison of Depth & PI of Cohesive soils Zone wise

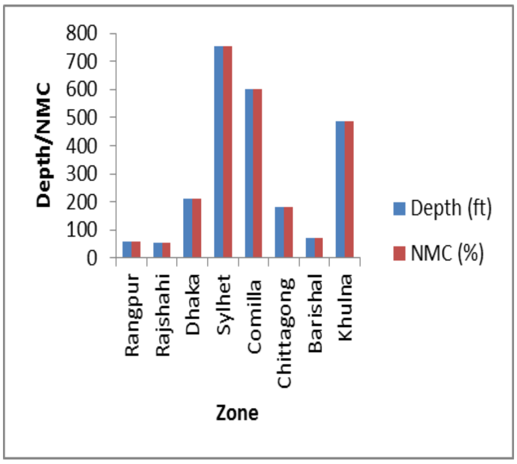

Fig.5: Comparison of Depth & NMC of Cohesive soils Zone wise

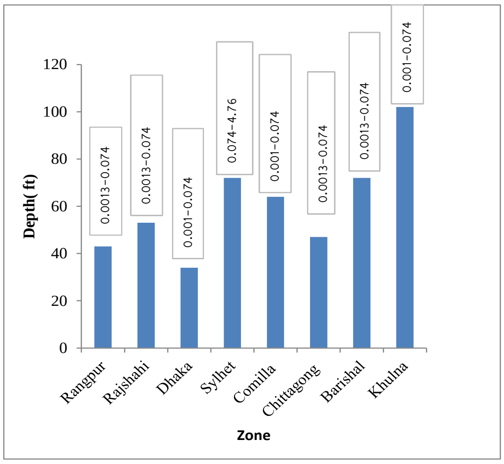

Fig.6: Zone wise Particle sizes of Cohesive soils according to depth

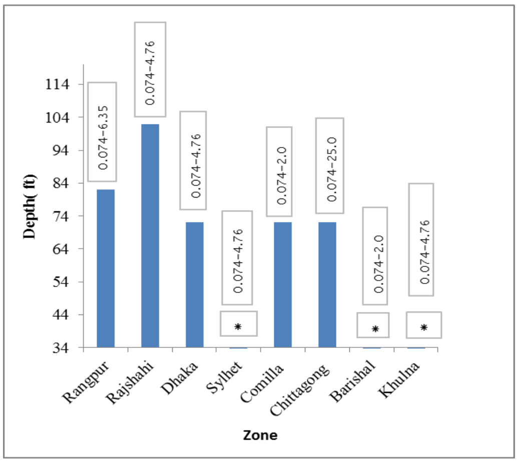

Fig.7: Zone wise Particle sizes of non-Cohesive soils according to depth

density varies from loose to very dense. But exception has been found in Bandarban District (0-12') contains SILT to COARSE gravel (**Source:** SMD,2008, 009,2010,2011,2012-13).

In Barisal zone, brown & grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found up to the maximum depth of about (0-72’), whose sizes vary about (0.0013-.074) mm and stiffness varies from very soft to very stiff. However, the occasional layers are found non-cohesive soils up to the depth of about 72,’ whose sizes vary about (0.074-2.0) mm and the relative density varies from very loose to dense. But exception has been found in Tengrakhali & Kalaia khal, Zianagar contains organic soils (**Source:** SMD, 2007, 2008, 2010,2011-12).

In Khulna zone, brown, grey & black in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found up to the maximum depth of about (0-102’), whose sizes vary about (0.001-0.074) mm and stiffness varies from very soft to hard. However, the occasional layers are found non-cohesive soils up to the depth 102,’ and the sizes vary about (0.074-4.76) mm and the relative density varies from very loose to very dense. But exception has been found in Bagerhat & Narail (38'-72'), which contains non-cohesive layers. (**Source:** SMD, 2007, 2009, 2010, 2011)

CONCLUSIONS

In this study, a large number of soil testing data are analyzed location wise for the entire Bangladesh. Finally, a range of results are selected zone wise for presentation. So, the similar results have been united as much as possible and exceptions are pointed out. However, a perspective of soil profile of Bangladesh has been focused which are introducing soil profile of Bangladesh. From these results, a design engineer will get chance to acquire knowledge about Bangladesh soil and its character. The authors suggest using these results of this study for assistance of design of construction and simultaneously they also recommend conducting the soil tests about particular locations of as and when necessary.

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ID: GE 006

IDENTIFICATION OF SOME BLACK SPOTS IN CHITTAGONG CITY AND ASSESSMENT OF THEIR GEOMETRIC FAULTS

H. F. Kanon[1*] & M. O. Imam[1]

[1] *Bangladesh Water Development Board, Dhaka, 1000, Bangladesh.*
[2] *Department of Civil Engineering, CUET, Chittagong, 4349, Bangladesh.*
**Corresponding Author, <h.f.kanon@gmail.com>*

ABSTRACT

Road transport network is one of the most important means of the communication system in Bangladesh as elsewhere in the world. An efficient transport network contributes to the economic and industrial growth, social and cultural development of any country. In recent years Bangladesh is experiencing an increasing trend of road accidents. A study conducted by the Accident Research Centre (ARC) of BUET indicates that road accidents are the cause of death of 12,000 lives annually and 35,000 injuries. This paper gives an insight towards detection of four "Black Spots" and the geometric faults in those black spots in Chittagong city which is the second largest city and main sea port of Bangladesh. Black spots are the road locations with geometric faults and have a record of large numbers of crashes. Selecting four most accident prone locations (Nimtoly, Karnaphuli Bridge, Custom Bridge, Saltoga crossing) in Chittagong city, the accident data of those locations are collected from BRTA, Chittagong City Corporation and corresponding police stations. Geometry of those locations are also measured. Using the obtained accident data and formula of "Flemish Speaking Community of Belgium", those locations are proved as black spots. From the data it is observed that accident rate of these locations are very high. Theoretically from the analysis, those four locations can also be identified as Black spots. From the geometry analysis of those locations some serious faults are identified. Geometric fault of road or junctions is the main reason to make a location "Black spot" or hazardous road section and causes fatal accidents in a regular interval. Black Spot Management has a long tradition all over the developed world and should be followed in Bangladesh very seriously to reduce the number of road accidents.

Keywords: Black Spot, Geometric Fault, Road Accident, Road Network

INTRODUCTION

Road transport network is the important means of the communication system in Bangladesh as in elsewhere in world. Transportation contributes to the economic, industrials, social and cultural development of any country. Road network is vital for the economic development of any region since every commodity produced whether it is food clothing industrials products or medicine need transport at all stages from to distribution.

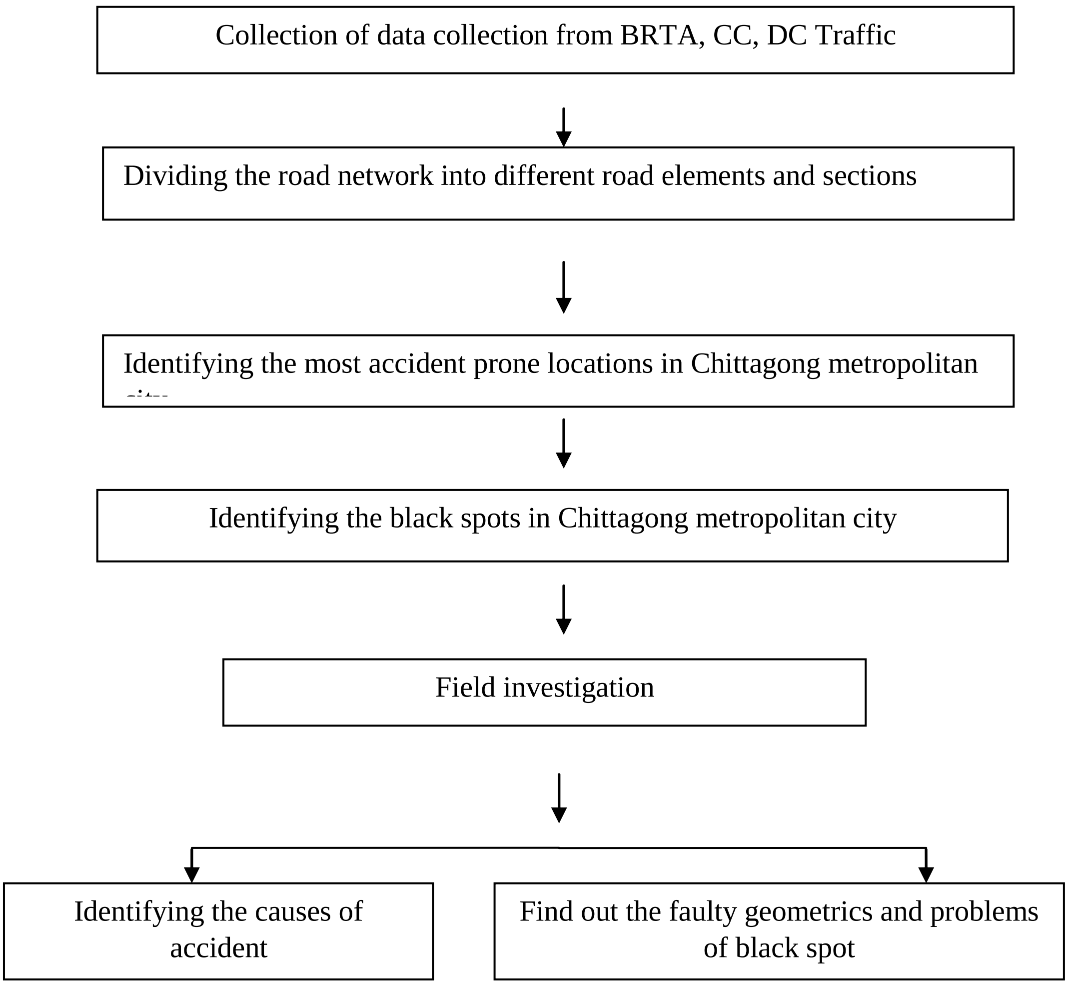
According to the latest RHD database report, there is the total length of about 22,378 Km road of different categories under the control and management of Roads and Highways Department, of which about 16274 Km is paved, while the remaining 6104 Km is either partly paved or unpaved. Of this total road network under the department, 3723 Km is national Highways, 4832 Km Regional Highway and remaining 13823 Km is the district road. Besides this, RHD has under its control about 4617 number of bridges with total length of 128.15 Km and 5320 number of culverts with a total length of about 35 Km . The geometrics of highway should be designed to provide optimum efficiency in traffic operation with reasonable cost.

The designer may be exposed to meet the requirement of the existing and the anticipated traffic. Geometric design of highway involve some element such as Cross section element, Sight distance considerations, Horizontal alignment, Vertical alignment, Intersection elements.

The problem of accident is very acute in highway transportation due to complex flow patterns of vehicular traffic presence of mixed traffic and pedestrians. Traffic accidents may involve property damages, personal injuries or even pedestrians. One of the main objectives of traffic engineering is to provide safe traffic movement. Road accident cannot be totally prevented, but by suitable traffic engineering and management measures, the accident rate can be decreased considerably. Road accident is not the results of a single cause but the combination of several contributory factor which are associated with the road users, the vehicles and the road environment. It becomes easier to take corrective measure only when identification of locations of accident hazards could be made on the basis of "accident type" and "accident location" studies. The hazardous accident location is known as "BLACK SPOTS".

METHODOLOGY

The specific objective include: identifying black spots in Chittagong metropolitan city, finding out the existing geometric faults and problems of black spot and suggesting remedial measures of the problem. Firstly, Information and accident data of Chittagong city are collected from BRTA, CC, DC Traffic.



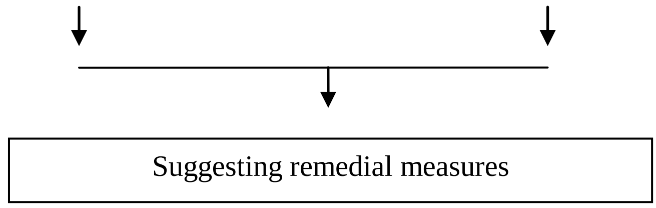


Fig 1: Methodology

Secondly initial analysis of the accident data highlights the hazardous sites that will be subject future to detailed study with a view to introducing low-cost engineering treatment. High accident site in Chittagong city are Nimtali, Karnaphuli bridge, Custom bridge Saltgola Crossing. In Chittagong city these four accident locations are considered as ‘dangerous’. These ‘dangerous’ accident sites or so-called black spots are selected by means of their historic accident data for the period of one year 2010. Thirdly formula of “Flemish speaking community of Belgium” is used to mathematically prove whether those locations are black spots or not. According to “Flemish speaking community of Belgium” A location is considered to be dangerous in terms of road accident and causalities when its priority value (P), calculated using the following formula, equals 15 or more . Where,

$P = X + 3*Y + 5*Z$, where X = total number of damage accidents

Y = total number of injury accidents

Z = total number of fatal accidents

And weighting values: For, Damage accidents = 1, Injury accidents = 3,Fatal accidents = 5. [Ref.4]. Fourthly field investigation is done to find out the faulty geometrics and causes behind the high accident rate. Finally some remedial measures are suggested to reduce the accident rate. The methodology can be summarised as below:

Most Hazardous Location in Chittagong City

The transportation network system in Chittagong city are very complex, most of the roads are built without following their geometrics design. Accidents occur due to poor road design are more frequent in Chittagong city. Accidents by location in Chittagong city which are consists of 47.1% of mid-block accidents, 51.9% of intersection accidents and 1.0% of unknown. The most hazardous mid-block locations and most hazardous intersection locations in Chittagong city with accidents frequency are following:

Table 1. Most hazardous mid-block location (In Order To Decreasing Frequency) (Source : Police Station)

Police Station	Mid-block Location	Total Accidents
Bandar	1. Barik building to port connecting Junction	12
Kotowali	2. New Market to Kadamtali	9
Panchlaish	3. Sholashahar to G.E.C.	8

Panchlaish	4. Sholashahar to Bayejid	7
Chandgaon	5. Bahaddarhat to Amanat Shah Bridge	7
Double Mooring	6. Dewanhat to Mansurabad	6
Bandar	7. Port connecting Junction to Barapool	6

Table 2. Most hazardous intersection locations with accident frequency (Source:Police Station)

Police Station	Intersection Location	Total Accidents
Chandgaon	Bahaddarhat (CDA Avenue and Badurtala Road)	6
DoubleMooring	Dewanhat (Dhaka Trunk Road and SK. Muzib Road)	10
Panchlaish	Sholashahar (CDA Avenue and Bayejid Bostami Road)	11
Kotowali	Tiger pass (CDA Avenue and Pologround Road)	12
Bandar	Port connecting Road Barik building Road Junction	9
Bandar	Custom (Near Custom House)	9
Panchlaish	G.E.C. (CDA Avenue and O.R. Nizam Road)	8
Bandar	EPZ (Near EPZ gate)	6
Kotowali	WASA (CDA Avenue and High level Road)	6

Among those locations four locations are selected on the basis of high frequency of accident. Accident record of those locations is mentioned below:

Table 3. Accident record at the selected four locations

Location	Total Fatal Accident	Total Injury Accident	Total Damage Accident	Year
Nimtali	13	22	11	2010
Karnaphuli bridge	17	35	41	2010-11
Saltgola Crossing	4	3	2	2010
Custom bridge	3	5	7	2010

Results and Discussions

Black Spot Identification

In Chittagong city the four accident locations are considered as ‘dangerous’. These ‘dangerous’ accident sites or so-called black spots are selected by means of their historic accident data for the period of one year 2010. According to “Flemish speaking community of Belgium” A location is considered to be dangerous when its priority value (P), calculated using the following formula, equals 15 or more.

Table 4. Accident Locations Identified As Black spots

Location	Priority value (P)	Comment
Nimtali	P= 11+22*3+13*5=142 > 15	Black spot
Karnaphuli bridge	P= 41+35*3+17*5=231 > 15	Black spot
Custom bridge	P= 3+5*3+7*5=53 > 15	Black spot
Saltgola Crossing	P= 4+3*3+2*5=142> 15	Black spot

Geometric fault or problem at Nimtali

Field investigations were carried out to assess the roadways geometric faults and other roadside hazards. From the field investigation geometric faults were assessed and enlisted in table 5.

Fig 1: Black Spot Location at Nimtali

Fig 2: Sharp Turning at Nimtali

Table 5. Comparison between the geometric fault or problem with geometric standard Nimtali

Location	Geometric fault / problem	Geometric standard
Nimtali	1. Existing lane width 2.8m	For multilane-pavements width of carriageway 3.5m per lane.
	2. No parking facility	Parking lane should have 3.0m width for parallel parking
	3. No lane marking on road	At least 10cm thick marking provided
	4.No crossing facility for pedestrian in the direction of Nimtali to Alankar	The width of pedestrian crossing should be between 2.0m and 4.0m.
	5. Turning is sharp.	Smooth curve turning provided.

Suggesting remedial mesure at the location of Nimtali

- Overbridge should be provided for pedestrians.
- Wider footway for pedestrians.
- Provide automatic signal system.
- Provide parking provision for parked vehicles.
- Provide parking controls for parked vehicles.
- Increase lane width and follow standard width of 3.5m per lane

Geometric fault and problem at Karnaphuli Bridge

The major problems are mentioned in table 6.

Table 6.Compariosn between the geometric fault or problem with geometric standard Nimtali

Location	Geometric fault / problems	Geometric standard
Karnaphuli Bridge	1.Vehicle moves at a speed of 56.52 K.P.H.	Design rotaries in urban areas to a speed of 30K.P.H.
	2. Existing width of carriageway at entry and exit is 8m.	For 14m (4 lanes) width of carriageway at entry and exist is 10m.
	3. Existing width of weaving section is 5m	Width of the weaving section is in the range of (6m-18m)
	4. Vehicle parked at the entry width.	There are no vehicle standing on the approach of rotary.

Fig 3. Less Weaving length of rotary in front of Karnaphuli Bridge

Fig 4. Contraction of entry width due to parking of vehicle on the road.

Suggesting remedial measure at the location of Karnaphuli Bridge

- Providing speed limits.
- Providing parking controls for the parked vehicles.
- Providing parking provision for the parked vehicles.
- Increase entry and exist width of the rotary.
- Increase the weaving length.

Geometric fault or problem at Custom Bridge

The major problems are mentioned in table 7.

Table 7. Comparison between the geometric fault or problem with geometric standard at Custom Bridge

Location	Geometric fault / problems	Geometric standard
Custom Bridge	Three no. of speed breaker gathered each of height is 15.24 cm(6 inch)	Width 25.4cm(10inch) x Height 6.35cm (2.5 inch)

Fig 5. Very high and wide speed breaker in front of custom bridge.

Fig 6. Very high single speed breaker at custom bridge.

Suggesting remedial measure at the location of Custom Bridge

- ➢ Provide smooth speed breaker following standard.
- ➢ Provide proper lighting.
- ➢ Provide reflecting signing/marking.

Fault or problem at Saltgola Crossing

The major problems are mentioned in table 8.

Table 8. Comparison between the geometric fault or problem with geometric standard at Saltgola Crossing

Location	Traffic Management problem	Traffic Management standard
Saltgola Crossing	At the time of train crossing no barricade exist.	For train crossing on the road barricade applied.
	Manual operated signal	Automatic signal

Fig 7 : No Automatic signal

Fig 8: At the time of train crossing no existence of barricade

Suggesting remedial measure at the location of Saltgola Crossing

- ➢ Proper traffic signal provided with barricade.

CONCLUSION

A zero level of accidents in ideally desirable but practically impossible to attain. Accident reduction targets must therefore be based on what is socially acceptable, and practically and economically achievable. It can be seen that the greatest potential for accident savings lies influencing human behavior. For very many road safety problems, there are alternative remedies, and the measure finally decided upon in any given instance will usually depend upon its ease of application and economic considerations. The following steps may take for lowering accident rate or remedy in black spots:

1. Construct the road in proper engineering method and ascertain the safety issues during construction stage.
2. Proper placement of signs, signals and marking and Improvement of existing black spots.
3. Placement of proper safety barrier in bridge approaches and in curve section of road.
4. Within the area of highway engineering alone, there are alternatives of geometric design or control, surfacing, sings, or marking, which individually or in combinative may provide a satisfactory solution to intersection conflicts.

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COMPARISON BETWEEN NUMERICAL AND ANALYTICAL SOLUTION OF SEEPAGE FLOW THROUGH EARTH DAM

M. M. Sazzad[1*], M. Roy[1] & M. S. Rahman[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi, Bangladesh.*
**Corresponding Author, <mmsruet.research@gmail.com>*

ABSTRACT

Measurement of seepage through hydraulic structures is very important from their safety point of view. In case of a dam made of earth, seepage generally occurs through the dam. Due to excessive seepage, scouring and piping may occur which may lead to ultimate failure of the dam. Consequently, seepage investigation is very important in designing any hydraulic structures. In this research, a comparison between numerical and analytical solutions has been conducted. Different geometric models have been investigated numerically with and without internal clay core. Discharge rates are computed for different numerical and analytical models. The discharge rates obtained from the numerical models have been compared with those of analytical solutions. The numerical results are consistent to that calculated using the analytical methods.

Keywords: Seepage, Earth dam, Clay core, Numerical analysis.

INTRODUCTION

Earth dam is an economical hydro-engineering structure. They are constructed for various purposes, such as the protection of living beings, properties, etc. from flood and storing water for irrigation, water supply and energy generation. It is sometimes used as recreational purposes. Therefore, it is very important to investigate the disastrous effect of water on earth dam. According to Foster and Fell (1999), 25% of dam failure has been due to wash out of fine granules of body of dam or dam foundation. There are various methods for calculating discharge rate (i.e., seepage) through the dam body. Using analytical solutions, the seepage through an earth dam can be estimated. Many researchers have used analytical methods to calculate the seepage through the dam body. Among the analytical methods, Dupit (1983), Schaffernak (1917), Casagrande (1937), Stello (1987), Rezk and Senoon (2010), Fakhari and Ghanbari (2013) are worthy of noting. The disadvantage of the analytical solution is that, it requires many assumptions and only simple and straightforward seepage problem can be solved.

Apart from this analytical approach, numerical approaches such as finite element method, finite difference method and finite volume method are also used to determine the seepage through the earth dam. Among these numerical methods, finite element method is the most popular and widely used method. An important advantage of using finite element method (FEM) in seepage analysis is that the solution of seepage problem is faster and complex seepage problems can be solved using FEM. Several authors have used FEM to solve the seepage problem of earth dam (Papagianakis and Fredlund, 1984; Lam et al, 1988; Potts and Zdravkovic, 1999). However, it is important to examine the consistency of the analytical and FEM based solutions to warrant the applicability of FEM based solutions of seepage problem of earth dam.

Consequently, the objective of this paper is to compare the seepage calculated by using the numerical method (FEM) and analytical solutions for earth dams of different geometric models. Analytical methods such as Schaffernak (1917), Casagrande (1937) and Fakhari and Ghanbari (2013) are used to compute the discharge rate whereas FEM based software SEEP/W (2012) is used to solve the discharge rate for numerical analysis. The results are compared and their consistency is discussed.

ANALYTICAL METHODS

Schaffernak (1917) presented an equation for calculating the seepage from the body of a homogenous dam placed on an impervious foundation as follows:

$$q = kl \sin\beta \tan\beta \qquad (1)$$

where,

$$l = \frac{d}{\cos\beta} - \sqrt{\frac{d^2}{\cos^2\beta} - \frac{H^2}{\sin^2\beta}} \qquad (2)$$

Here, β, d and H are the angle of downstream slope, length of drainage path and height of upstream water, respectively (Fig. 1).

Casagrande (1937) proposed equations for evaluating the amount of discharge rate passing through the body of a dam by assuming that the hydraulic slope dz/dx equals to dz/ds as follows:

$$q = kl \sin^2\alpha \qquad (3)$$

where

$$l = s - \sqrt{s^2 - \frac{h^2}{\sin^2\alpha}} \qquad (4)$$

$$s = \sqrt{d^2 + h^2} \qquad (5)$$

$$d = b - 0.7\Delta \qquad (6)$$

Here, α, h and b are downstream angle, downstream water height and width of core floor, respectively (Fig. 2).

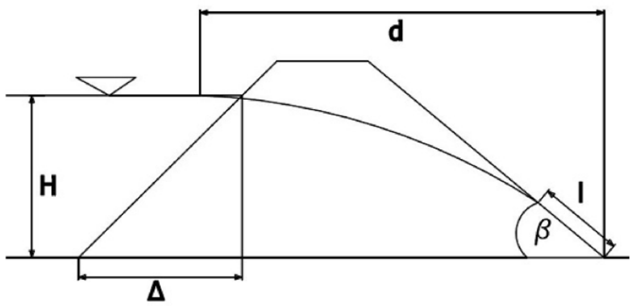

Fig. 1: Schematic diagram of dam for Schaffernak's (1917) solution.

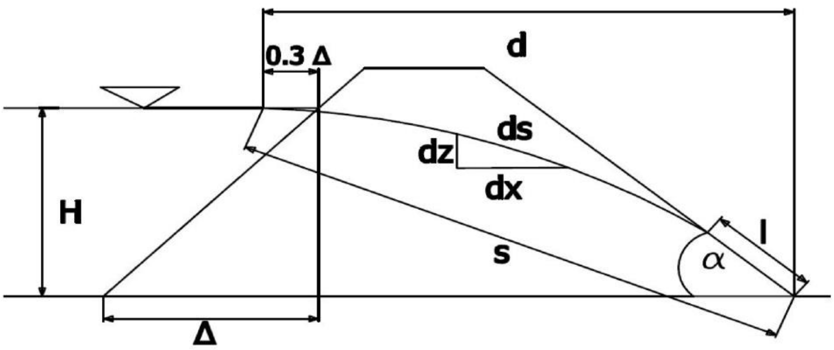

Fig. 2: Schematic diagram for Casagrande's (1937) solution

Fakhari and Ghanbari (2013) presented equations for estimating discharge rate through vertical and inclined clay core as follows:

$$q = fkh \qquad (7)$$

$$f = (2.27 - 0.006w - 0.004h - 0.38\tan\alpha) \times H^{(-0.361)} \times \left(\frac{c}{h}\right)^{(0.3947\tan\alpha + 0.0015h - 1.3591)} \qquad (8)$$

$$c = b - 0.7\Delta \qquad (9)$$

where, b and f are the base width and seepage factor, respectively.

NUMERICAL ANALYSIS

In this study, numerical models of earth dams have been analyzed using SEEP/W (2012) along with the various analytical methods. Steady state seepage analysis has been conducted. Base of the earth dam is considered impervious. Models with and without clay core have been analyzed. Earth dam is analyzed with reservoir water height varying from 40 m to 46 m and height of dam is kept constant (50 m). Width of dam crest is also kept constant (8 m). At first, the numerical model dam has been analyzed with the same upstream and downstream angle of 45°, 40° and 35°. Later, it has been analyzed for varying downstream angles. Hydraulic conductivity (k) for dam material is considered 1×10^{-6} m/sec and for the internal clay core, it is set to 1×10^{-9} m/sec (Das, 2001). Fig. 3 shows the typical numerical models used in the present study.

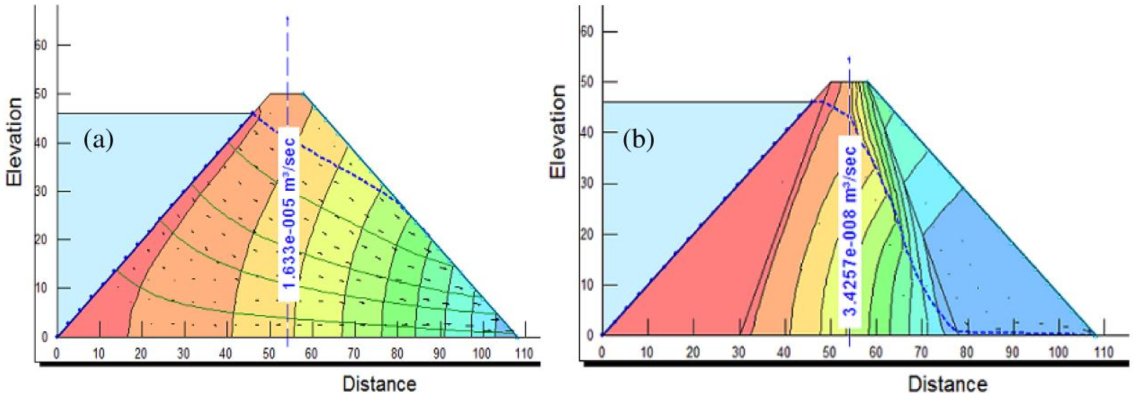

Fig. 3: Numerical model of earth dam: (a) without clay core and (b) with clay core

RESULTS AND DISCUSSION

The results obtained from the numerical and analytical solutions have been presented in the tabular form. From Table 1, it is obvious that the discharge rate obtained from the numerical analysis is quite similar to that of the analytical solution by Fakhari and Ghanbari (2013) compared to Schaffernak (1917) and Casagrande (1937) when same upstream and downstream angles are considered. Table 2 shows that the numerical results are also closer to the analytical solution by Fakhari and Ghanbari (2013) when different upstream and downstream angles are considered. Note that discharge rate decreases as the water height of the reservoir decreases.

Numerical results for earth dam with internal core for the same upstream and downstream angles are depicted in Table 3 and Table 4, respectively. From Table 3 and Table 4, it is observed that discharge rate is almost same for both the same and different upstream and downstream angles i.e. when core is provided in the earth dam, the upstream and downstream slope angles do not affect the discharge rate.

Note also that discharge rate reduces for the incorporation of clay core compared to that for homogeneous soil.

Fig. 4 shows the effect of clay core on seepage. In case of homogeneous earth dam, the discharge rate decreases with the decrease of both the height of the water head and the upstream or downstream angle; however, for earth dam with clay core, the change of upstream or downstream angle has no influence on the discharge rate.

Table 1 Comparison between numerical and analytical results for homogeneous dam (Same upstream and downstream angle)

α or β	Crest width W (m)	Dam Height H (m)	Water Height h (m)	k (m/sec)	Seepage (m^3/sec)			
					q Schaffernak	q Casagrande	q Fakhari and Ghanbari	q SEEP/W
45	8	50	46	1.00E-06	2.53E-05	1.42E-05	1.62E-05	1.63E-05
	8	50	43	1.00E-06	2.35E-05	1.20E-05	1.45E-05	1.36E-05
	8	50	40	1.00E-06	2.19E-05	1.01E-05	1.27E-05	1.13E-05
40	8	50	46	1.00E-06	2.20E-05	1.25E-05	1.54E-05	1.43E-05
	8	50	43	1.00E-06	2.03E-05	1.05E-05	1.36E-05	1.19E-05
	8	50	40	1.00E-06	1.89E-05	8.75E-06	1.18E-05	9.84E-06
35	8	50	46	1.00E-06	1.90E-05	1.08E-05	1.44E-05	1.25E-05
	8	50	43	1.00E-06	1.75E-05	9.03E-06	1.25E-05	1.03E-05
	8	50	40	1.00E-06	1.62E-05	7.52E-06	1.07E-05	8.50E-06

Table 2 Comparison between numerical and analytical results for homogeneous dam (Different upstream and downstream angle)

α	β	Crest width W (m)	Dam Height H (m)	Water Height h (m)	k (m/sec)	Seepage (m^3/sec)			
						q Schaffernak	q Casagrande	q Fakhari and Ghanbari	q SEEP/W
45	40	8	50	46	1.00E-06	2.25E-05	1.30E-05	1.57E-05	1.46E-05
		8	50	43	1.00E-06	2.09E-05	1.10E-05	1.39E-05	1.23E-05
		8	50	40	1.00E-06	1.96E-05	9.20E-06	1.22E-05	1.02E-05
40	35	8	50	46	1.00E-06	1.94E-05	1.13E-05	1.48E-05	1.28E-05
		8	50	43	1.00E-06	1.80E-05	9.50E-06	1.30E-05	1.07E-05
		8	50	40	1.00E-06	1.69E-05	7.93E-06	1.12E-05	8.83E-06
35	30	8	50	46	1.00E-06	1.66E-05	9.69E-06	1.36E-05	1.10E-05
		8	50	43	1.00E-06	1.53E-05	8.10E-06	1.18E-05	9.09E-06

		8	50	40	1.00E-06	1.43E-05	6.74E-06	1.01E-05	7.50E-06

Table 3 Numerical results for earth dam with internal clay core
(Same upstream and downstream angle)

α or β	Crest width W (m)	Dam Height H (m)	Water Height h (m)	*k* (m/sec)	*k(core)* (m/sec)	q Seepage (m^3/sec) SEEP/W
45	8	50	46	1.0E-06	1.0E-09	3.43E-08
	8	50	43	1.0E-06	1.0E-09	2.92E-08
	8	50	40	1.0E-06	1.0E-09	2.47E-08
40	8	50	46	1.0E-06	1.0E-09	3.43E-08
	8	50	43	1.0E-06	1.0E-09	2.93E-08
	8	50	40	1.0E-06	1.0E-09	2.47E-08
35	8	50	46	1.0E-06	1.0E-09	3.41E-08
	8	50	43	1.0E-06	1.0E-09	2.91E-08
	8	50	40	1.0E-06	1.0E-09	2.46E-08

Table 4 Numerical results for earth dam with internal clay core
(Different upstream and downstream angle)

α	β	Crest width W (m)	Dam Height H (m)	Water Height h (m)	*k* (m/sec)	*k(core)* (m/sec)	q Seepage (m^3/sec) SEEP/W
45	40	8	50	46	1.0E-06	1.0E-09	3.42E-08
		8	50	43	1.0E-06	1.0E-09	2.91E-08
		8	50	40	1.0E-06	1.0E-09	2.46E-08
40	35	8	50	46	1.0E-06	1.0E-09	3.42E-08
		8	50	43	1.0E-06	1.0E-09	2.92E-08
		8	50	40	1.0E-06	1.0E-09	2.47E-08
35	30	8	50	46	1.0E-06	1.0E-09	3.42E-08
		8	50	43	1.0E-06	1.0E-09	2.91E-08
		8	50	40	1.0E-06	1.0E-09	2.46E-08

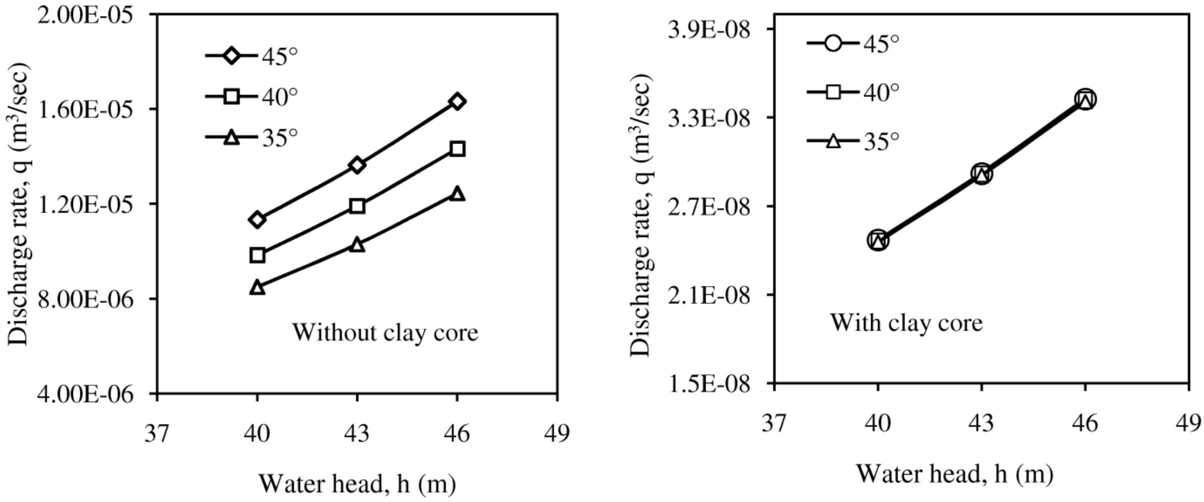

Fig. 4: Variation of discharge rate for earth dam with and without clay core when upstream and downstream angles are same using SEEP/W (2012)

CONCLUSION:

In this study, a comparison between the numerical and analytical solution is presented. Sixty numerical models of earth dams have been analyzed without and with clay core. From the study, it is noted that the discharge rate obtained from the numerical analysis is close to that from the analytical solutions by Fakhari and Ghanbari (2013) whether the upstream and downstream angles remain same or different. Noticeable reduction of the discharge rate is observed with the addition of clay core in the model. It should be noted that the upstream and downstream angle has no effect on discharge rate when internal clay core is used.

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EFFECT OF PARTICLE SIZE AND RELATIVE DENSITY ON DYNAMIC RESISTANCE OF CLEAN SAND

Md. Assaduzzaman[1*], Md. Rajedul Islam[1] & Md. Jahangir Alam[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka-1000, Bangladesh.*
**Corresponding Author, <asadbuet14@gmail.com>*

ABSTRACT

Sand fill are required for many purposes, for example, backfill of earth retaining structures, backfill in foundation trenches, reclamation of low lands etc. In all these situations good compaction of fill should be ensured to avoid future subsidence, failure of foundation and moreover liquefaction. The air pluviation method was calibrated to know the height of fall and holes diameter of discharging bowls for a desired relative density of specific sand in the first stage. In the second stage using this relation between relative density and height of fall, sand deposits of different relative densities were prepared in calibration chamber. On the prepared sand deposit DPL and DCP tests were performed. Penetration of cone was recorded for every blow of hammer. N_{10} and P_{index} value of DCP and DPL tests were determined. N_{10} is the number of blows per 10 cm of penetration of dynamic cone and P_{index} is the penetration rate of cone in mm/blow. To get a generalized correlation for various sizes of sand, P_{index} values were normalized by multiplying it with $D_{50}^{0.75}$ value. Then a generalized correlation between relative density and $P_{index}D_{50}^{0.75}$ were found in DPL and DCP for clean sand of any particle size. The density of granular soils varies with the shape and size of grains, the gradation and the manner in which the mass is compacted. Resistance of sand increases exponentially with relative density. The larger the particle size greater the resistance to penetration for a certain relative density of sand. Denser sand gives more resistance for a specific type of sand.

Keywords: Relative density, Sand cone method, Penetration index, Air pluviation, Dynamic cone and Sand deposit

INTRODUCTION

In our country, quality control of sand fill is done by determining field density near the top surface of fill using Sand Cone Method (ASTM D 1556-90, 2006). It has limitations because it is a direct method of determining field density and Relative Density of sand fill. This method is very difficult to perform at deeper locations. Sand Cone Method has to be applied to control the quality of sand fill after compaction/densification of each layer of fill. For this reason it is time consuming and expensive to use Sand Cone Method. Sand Cone Method cannot be applied in saturated sand or where water table is high. To determine Relative Density of sand fill easily, it is necessary to develop an indirect method which can be performed in all seasons and in any location. It is proved that dredge fill sand is liquefiable from several case studies of earthquakes. In earthquake when seismic liquefaction occurs, even pile foundations could not save the structure from damage in many cases. To make the sand fill non liquefiable it should be well compacted. Mitigation measures become very expensive if a structure is constructed on liquefiable soil and it would be damaged during earthquake. So, it is very important to control the quality of sand fill. The density of granular soils varies with the shape and size of grains, the gradation and the manner in which the mass is compacted. The term used to indicate the strength characteristics in a qualitative manner is Relative Density (D_r) (Murthy, 1993) which describes the state condition in cohesionless soils. It is commonly used to identify liquefaction potential under earthquake or other shock-type loading (Seed and Idris, 1971). So Relative Density is a very important index for a sandy soil. Relative Density is 0% for loosest condition of sand and 100% for densest condition of sand. If maximum index density and minimum index density of sand is determined in laboratory as per ASTM D4253 and D4254, and field dry density is determined by any one of the methods such as Sand Cone Method (ASTM D1556), Sleeve Method (ASTM D4564), Rubber Balloon Method (ASTM D2167), and Drive-Cylinder Method (ASTM D2937), Relative Density can be calculated using the following formula.

$$D_r(\%) = \left(\frac{\gamma_d - \gamma_{\min}}{\gamma_{\max} - \gamma_{\min}}\right) \cdot \left(\frac{\gamma_{\max}}{\gamma_d}\right) 100 \quad (1)$$

Where, γ_d = field dry density of sand deposit, $\gamma_{\max}$ = maximum index density and $\gamma_{\min}$ = minimum index density. Sand fill are required for many purposes, for example, backfill of earth retaining structures, backfill in foundation trenches, reclamation of low lands etc. In all these situations good compaction of fill should be ensured to avoid future subsidence, failure of foundation and moreover liquefaction. Relative Density is the most appropriate index to control the compaction of sand fill. Depending on the importance of structure, minimum Relative Density generally be specified as 70% to 95%.

Dynamic Cone Penetration (DCP)

The dynamic cone penetration test (DCPT) was developed in Australia by Scala (1956). The current model was developed by the Transvaal Roads Department in South Africa (Luo, 1998). The mechanics of the DCPT shows features of both the CPT and SPT. The DCPT is performed by dropping a hammer from a certain fall height measuring penetration depth per blow for a certain depth. For the rapid in situ measurement of the structural properties of existing road pavements with unbound granular materials the Transportation Research Laboratory Dynamic Cone Penetration (TRL-DCP) test apparatus is designed. Continuous measurements can be made to a depth of 800 mm or to 1200 mm when an extension rod is fitted. The instrument is held vertical; the weight is carefully raised up to the handle; then the weight is dropped on anvil freely. A reading should be taken at increments of penetration of about 10 mm is recommended. However it is usually easier to take readings after a set numbers blows. It is therefore necessary to change the number of blows between readings according to the strength of the layer being penetrated. It is important to note that DCP and DPL has similar features except differences in cone size, weight of anvil, weight of drop hammer and height of fall. Table 1 shows the differences between DCP and DPL.

Table 1: Basic differences between DCP and DPL

Parameters	DCP	DPL
Hammer (kg)	8	10
Height of fall (m)	0.66	0.50
Mass of anvil and guide rod (kg)	--	6
Cone diameter (mm)	22.5	35.7
Apex angle of cone (degree)	60	90

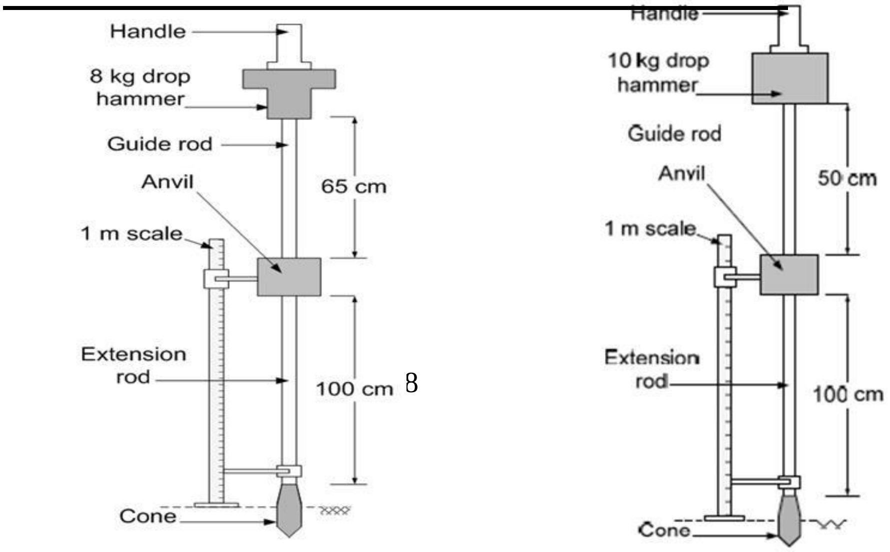


Fig. 1: Shametric diagram of DCP (Left) and DPL (Right)

Application of DCP and DPL

DCP and DPL testing can be applied to the characterization of subgrade and base material properties in many ways. Perhaps the greatest, strength of the DCP device lies in its ability to provide a continuous record of relative soil strength with depth. By plotting a graph of penetration index (P_{index}) versus depth below the testing surface, a user can observe a profile showing layer depths, thicknesses, and strength conditions. This can be particularly helpful in cases where the original as-built plans for a project were lost, never created, or found to be inaccurate.

Procedure of DCP and DPL Uses

All DCP and DPL tests were performed by two operators. One person operated the hammer, while the other person reads and records the penetrations. Before each test, the tip of the ruler used to measure the penetrations was placed to a marked reference point on the surface. The person who took the readings was responsible to ensure that the ruler was kept parallel to the penetrating rod while taking measurements. Friction between the rod and the tested material has negative effects on the results. In order to minimize the friction of the rod with surrounding soil, the DCP and DPL must be kept vertical during penetration. If the DCP and DPL deviates from vertical position and operator continues to test, the device might be damaged and the results obtained for that test will not be reliable.

Determination of Pentration Index from DCP and DPL Test

The lower shaft containing the cone moves independently from the reading rod sitting on the testing surface throughout the test reading is counted as initial penetration corresponding to blow 0. The initial reading is not usually equal to 0 due to the disturbed loose state of the ground surface and the self-weight of the testing equipment. The value of the initial from the first drop of the hammer. Hammer blows are repeated and the penetration depth is measured for each hammer drop. This process is continued until a desired penetration depth is reached. Since the recorded blow counts are cumulative values, results of DCP and DPL test in general are given as incremental values defined as follows,

$$P_{index} = \frac{\Delta D_{P2}}{\Delta BC} \quad (2)$$

Where, P_{index} = DCP penetration index in units of length divided by blow count, ΔD_p = incremental penetration depth and ΔBC = blow counts corresponding to incremental penetration depth ΔD_p.

The main objectives of the study areas follow:

i) To identify the effect of particle size on penetration index at different relative density.

ii) To calibrate the DCP and DPL in a calibration chamber so that a correlation can be made between Penetration Index (P_{index}) and Relative Density (D_r %) for sand by using of different mean diameter sand having D_{50}=0.70 mm and 0.35 mm respectively.

MATERIALS AND METHODS

Describing the overall program including sample preparation, dynamic cone penetrometers (DCP) and dynamic probabing light (DPL) test,

Sample Preparation

In the first stage sample was prepared for required mean diameter (D50). Sylhet sand was collected from local market. Mean diameter (D50) value of test sample is 0.70 mm that was achieved by sieving thoroughly No. 4 passing and No. 200 retained sand collecting. Similarly same procedure was followed to achieve mean diameter value 0.35 mm of test sample by sieving thoroughly No. 30 passing and No. 200 retained sand collecting. Grain Size distributions are shown in Fig. 2. Index properties of two types of sand are shown in Table 2.

Fig. 2: Grain size distribution curve of sand having mean diameter (D_{50}) is 0.70 mm and 0.35 mm that used in the study

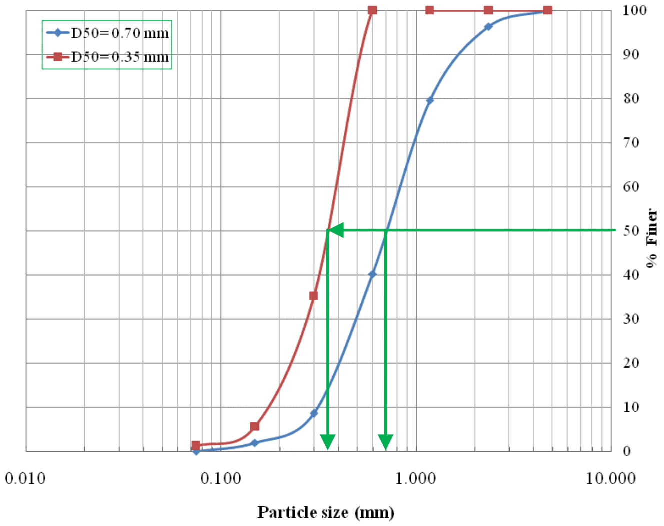


Table 2: Properties of two types of sand

Properties	Type 1	Type 2
F.M	3.74	2.58
D_{30} (mm)	0.50	0.28
D_{50} (mm)	0.70	0.35
D_{60} (mm)	0.85	0.40
C_u	2.66	2.22
C_c	0.92	1.08
e_{max}	0.67	0.95
e_{min}	0.48	0.57
γ max (kN/m^3)	17.50	16.52
γ min (kN/m^3)	14.54	13.30
Fines (%)	0	0
Types (USCS)	SP (Clean Sand)	SP Clean Sand)

**Note: D_{10} (mm) = particle size corresponding to 10% finer*

, D_{50} (mm) = mean diameter of sand.

different diameters (3.5 mm, 4.0 mm, 5.0 mm and 6.0 mm) are punched into the plastic bowl. Hole to hole triangle distance was 35 mm. A CBR mold was filled up by discharging sand from these holed bowls (here after called discharging bowl) maintaining fixed height of fall. Then density of sand was determined by weighing sand in CBR mold. This procedure was repeated for different height fall to get different densities for a specific type of sand. Two types of sand were calibrated by this procedure having mean diameter D_{50}=0.70 mm and D_{50}=0.35 mm.

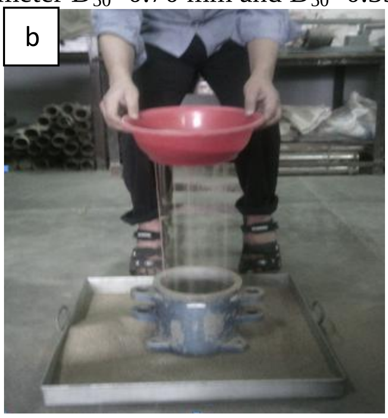


Fig. 3: a) Sand discharge bowl with 3.5 mm diameter holes; b) Air pluviation method

Preparation of Sand Deposit

The calibration chamber was placed on a level ground. Sands were air dried by spreading them on dry floor. Then sand deposit of desired density was prepared by air pluviation method. Height of fall was maintained by suspending a small weight from the discharging bowl through a fixed length of rope. Dry deposition of sand is shown in Fig. 4. Sand deposit of various relative densities was prepared using this method.

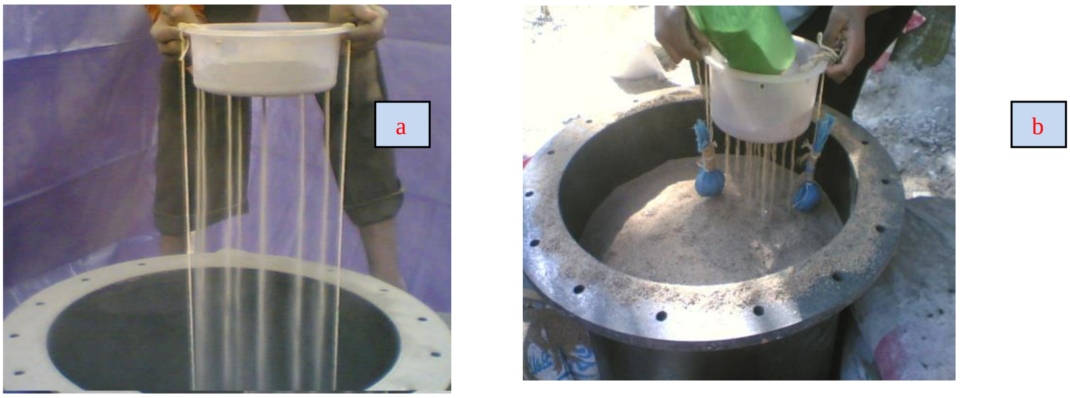


Fig. 4: a) Dry d eposition into calibration chamber from discharging bowl maintaining a constant height of fall, b) Filling of calibration chamber in progress

Then DCP and DPL test was performed on the prepared sand deposit. The total weight of sand was measured to determine the density of sand deposit. To perform the DCP and DPL test a steel cylinder of diameter 0.5 m and height 1.0 m was used as a calibration chamber. The thickness of calibration chamber wall was 13 mm.

DCP and DPL Tests

The calibration chamber was filled up by discharging sand from the discharging bowls maintaining fixed height of fall. After filling the calibration chamber, every time DPL and DCP tests were performed and for each blow the penetration of cone was recorded. Sand deposits of different relative densities were prepared in calibration chamber where DCP and DPL tests were performed. To obtain data for analysis, many laboratory tests were performed. It contained physical and index properties for strength properties determination. All the tests have standard procedure. After completion of tests, sands were taken out from the calibration chamber and weighed by digital balance to check the density and Relative Density of sand deposit. Correlation between Penetration Index (P_{index}) and relative density was made from the test result in calibration chamber.

RESULTS & DISCUSSIONS

Penetration of cone was recorded for every blow of hammer. N_{10} and P_{index} value of DCP and DPL tests were determined. N_{10} is the number of blows per 10 cm of penetration of dynamic cone and P_{index} is the penetration rate of cone in mm/blow. Figure 5 and 6 shows DPL and DCP test result. To get a generalized correlation for various sizes of sand, P_{index} values were normalized by multiplying it with $D_{50}^{0.75}$ that shown in Fig. 9 and 10. Then a generalized correlation between Relative Density and $P_{index}D_{50}^{0.75}$ were found in DPL and DCP for clean sand of any particle size.

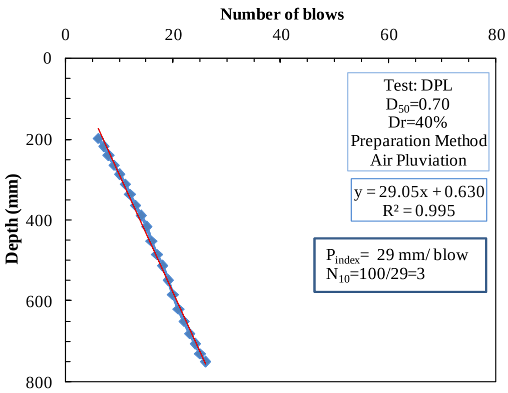


Fig. 5: Typical plot of number of blows vs. depth for sand having D_{50}= 0.35 mm at D_r= 78.9% in calibration chamber using DPL

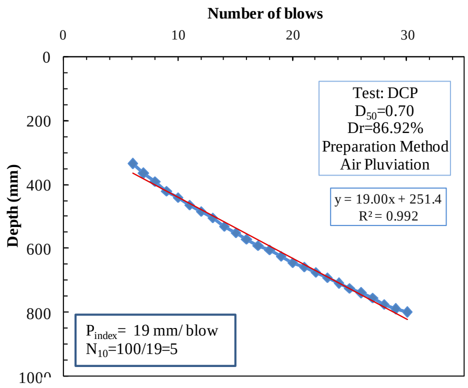


Fig. 6: Typical plot of number of blows vs. depth for sand having D_{50}= 0.35 mm at D_r= 78.9% in calibration chamber using DCP

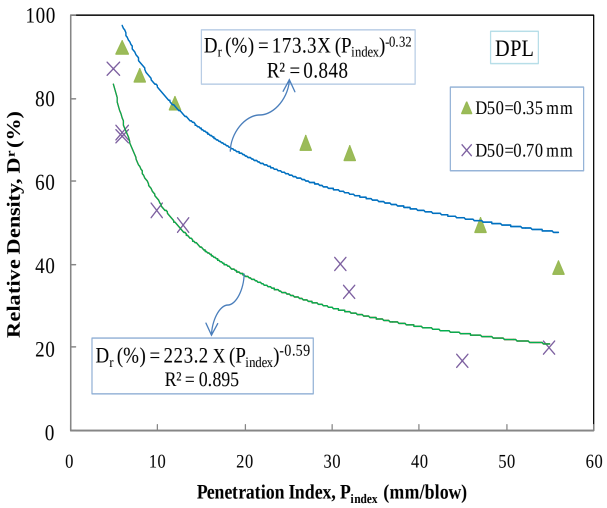
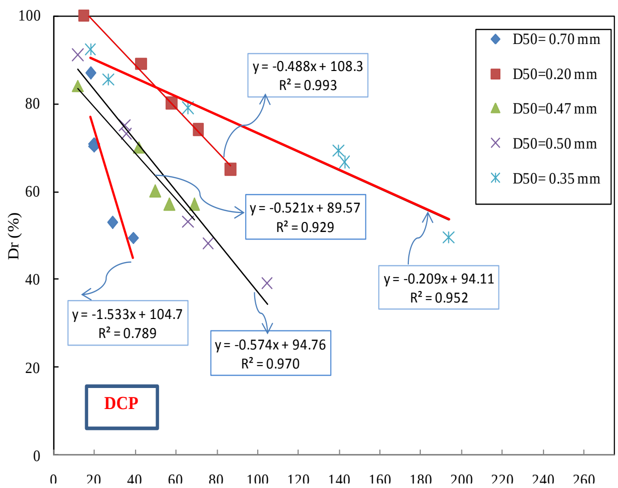


Fig. 7: Correlation between Relative Density and P_{index} in DPL for sand (Non linear scale)



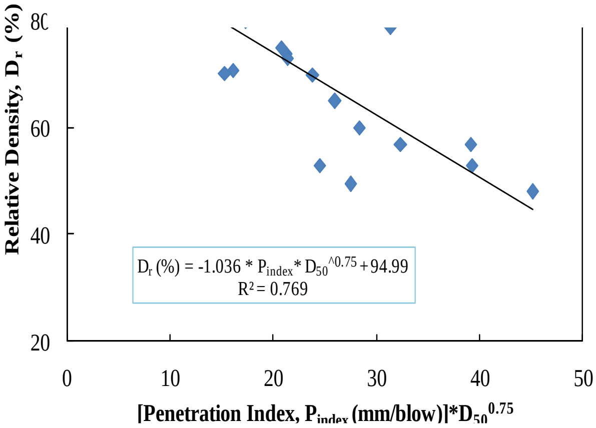


Fig. 9: Correlation between Relative Density and $P_{index}D_{50}^{0.75}$ in DCP for sand (Linear scale)

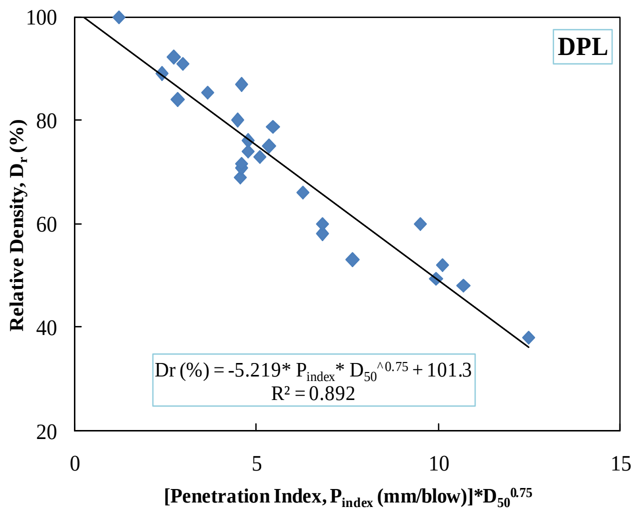


Fig. 10: Correlation between Relative Density and $P_{index}D_{50}^{0.75}$ in DPL for sand (Linear scale)

Generalized equation for DCP can expressed as

$$\mathrm{D_r}\,(\%) = -1.036 P_{index} D_{50}^{\;0.75} + 94.99 \qquad (3)$$

Developed equation for DPL can be expressed as

$$\mathrm{D_r}\,(\%) = -5.219 P_{index} D_{50}^{\;0.75} + 101.3 \qquad (4)$$

CONCLUSIONS

The conclusions are drawn with respect to this experimental study

A generalized correlation between Relative Density and P_{index} were found which is applicable to clean sand of any particle size. Resistance of sand increases exponentially with Relative Density. The larger the particle size greater the resistance to penetration for a certain Relative Density of sand. Denser sand gives more resistance for a specific type of sand. The proposed method can be used as an indirect method to determine in situ Relative Density of sand deposit.

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EFFECT OF SURCHARGE ON THE STABILITY OF SLOPE IN A HOMOGENEOUS SOIL BY FEM

M. M. Sazzad[1*] & M. F. Haque[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi, Bangladesh,*
**Corresponding Author, <mmsruet@gmail.com>*

ABSTRACT

Engineering structures such as bridge abutments, tower footings for electrical transmission lines, column load from high-rise buildings etc. are often required to construct near the slope. So, it is important to study the stability of slope which is subjected to the surcharge load from such engineering structures. Limit equilibrium methods such as Fellenius method, Bishop method, Janbu method and Morgenstern-Price method are usually used. However, these methods suffer from many assumptions. With the advances in the computer technology, finite element method (FEM) is widely used in recent years to study the stability of slopes. In this paper, the effect of surcharge on the stability of slope in a simple homogeneous soil is studied under the variation of the different factors such as the slope height, slope angle, etc. using FEM. The slope stability analysis is carried out by adopting the strength reduction method. Mohr-Coulomb failure criterion is used as the material model. A typical model slope is analyzed using FEM to study the effect of surcharge on the stability of slope with different conditions and related results are reported.

Keywords: Surcharge, Slope stability, FEM

INTRODUCTION

Usually a slope fails by the change in location of surcharge, magnitude of surcharge, variation of sloping angle, etc. Some natural disasters are involved in the failure of slope which causes loss of lives and assets. So, the analysis of the stability of slope is important in geotechnical engineering. Traditional methods such as the limit equilibrium method, limit analysis method, slip line fields method, etc are usually used in the stability analysis of slopes. However, these traditional methods require many assumptions to be considered beforehand. Alternatively, a numerical approach can be used. Finite element shear strength reduction method can easily calculate the failure plane of slope with allowable safety factor. Calculation efficiency of finite element method (FEM) is very high. Griffiths and Lane (1999) used the finite element shear strength reduction method to comprehensively analysis the stability of slope. Manzari and Nour (2000) studied the impact of different factors on the stability of slope using the finite element strength reduction method. He and Zhang (2012) indicated that the equivalent area circle Drucker-Prager yield criterion is suitable for the stability analysis of slope for a homogeneous soil. These studies by FEM depicts in general that FEM can be successfully used in the stability analysis of slopes. In the present study, the effect of surcharge on the stability analysis of slopes is studied by finite element shear strength reduction method. Variation of slope angle and slope height with different values of surcharge are also studied using the FEM based computer program GEO5 (2013) and related results are reported.

CONCEPT OF STRENGTH REDUCTION METHOD

Finite element shear strength reduction method is successfully used in the calculation of the factor of safety of slope. In this method, a strength reduction factor (SRF) is assigned. The factor of safety is usually calculated as follows:

$$FS = \tau / \tau_f \qquad (1)$$

where, τ is the shear strength of the materials used in the slope and calculated as follows:

$$\tau = c + \sigma_n \tan\phi \qquad (2)$$

Here, c and ϕ are the shear strength parameters of the materials used in the slope. The shear strength of the sliding surface is denoted by τ_f and given as follows:

$$\tau_f = c_f + \sigma_n \tan\phi_f \qquad (3)$$

where, c_f and ϕ_f are the factored shear strength parameters and they can be given as follows:

$$c_f = c/SRF \qquad (4)$$

$$\phi_f = \tan^{-1}(\tan\phi/SRF) \qquad (5)$$

To mark out the correct value of SRF, the values of the factor of safety is to be determined that will just cause the slope to fail.

FINITE ELEMENT MODEL

Fig. 1shows the typical model of a slope for the analysis by finite element shear strength reduction method. The symbols and their corresponding values are presented in Table 1.

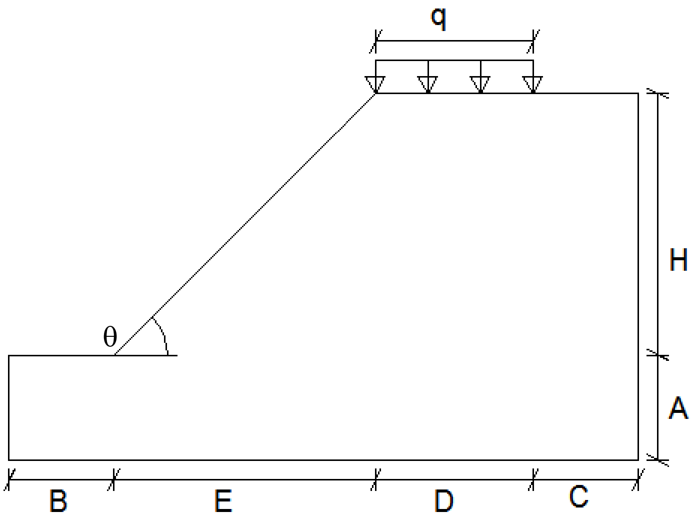


Fig. 1 A typical model of slope with surcharge used in the present study.

Table 1 Values of different symbols used in Fig. 1

Symbols	**Values**	**Unit**
Surcharge, q	10, 50, 100	kN/m^2
Slope angle, θ	20, 30, 40, 50, 60, 70, 80, 90.	degree
A	2	m
B	2	m
C	2	m

D	3	m
H	1, 2,3,4,5	m

MATERIALS AND METHODS

In this study, the model slope contains homogeneous clayey soil, the property of which is shown in Table 2. The geometric model of the slope is incorporated in the software GEO5 (2013) following Fig.1. The properties of soil are assigned. Mohr-Coulomb model is selected as material model. Meshes are generated and a strip of surcharge is added on top of the slope as shown in Fig.2. Maximum number of iterations for one calculation step is set to 100. Changing phenomena of stiffness matrix is followed after each iteration. Displacement error is taken as 0.0100. Imbalanced force error is considered as 0.01. Analysis is completed by the reduction of shear strength parameters.

Table 2 Value of soil parameters used in this study

Soil Parameters	**Values**	**Unit**
Bulk unit weight, γ	21	kN/m^3
Elastic modulus, E	3	MPa
Poisson's ratio, υ	0.40	---
Saturated unit weight, γ_{sat}	23	kN/m^3
Internal friction angle, φ (eff.)	19	Degree
Cohesion, c (eff.)	12	kN/m^3

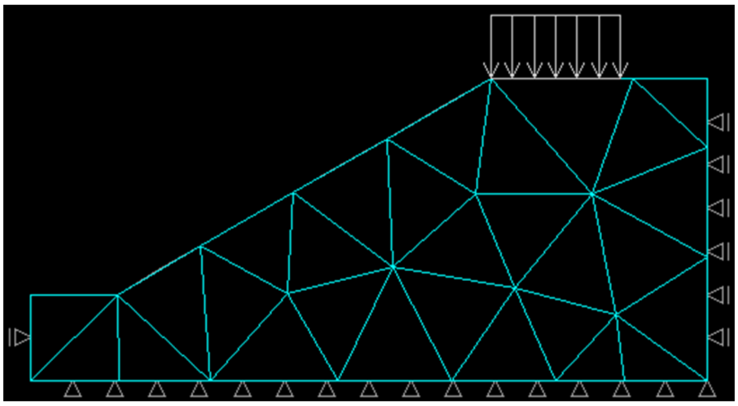
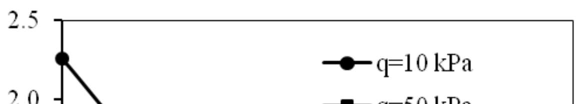


Fig. 2 A typical FEM mesh used in the present study.

RESULTS AND DISCUSSION

The effect of slope angle on the factor of safety for different values of surcharge is depicted in Fig.3. Factor of safety gradually decreases as the slope angle increases. The pattern of the reduction of factor of safety due to the variation of surcharge is almost same except that the values of the factor of safety are higher when the surcharge is smaller. It indicates that the increased values of surcharge have negligible influence on the pattern of the reduction of factor of safety. The effect of slope height H (Fig.1) on the factor safety of slope for particular slope angle and surcharge is shown in Fig. 4. Note that the factor of safety decreases as the height of slope increases. The contours of equivalent plastic strain for homogeneous slope by finite element method for the slope angle of 30˚ and a



surcharge of 50 kN/m^2 are shown in Fig. 5, while the same for the slope angle of 60˚ is shown in Fig. 6. Maximum plastic strain in observed near the crest of the slope where the surcharge is located.

Fig. 3 Effect of slope angle on the factor of safety for different values of surcharge

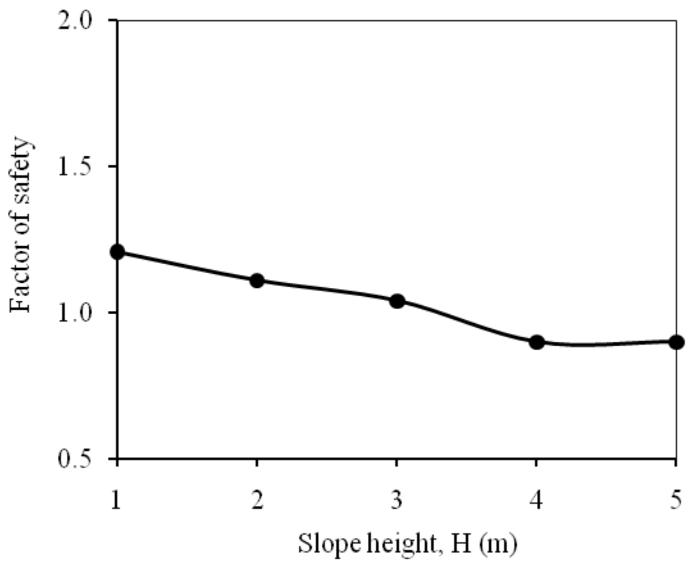


Fig. 4 Effect of slope height on the factor of safety for a particular slope angle and surcharge

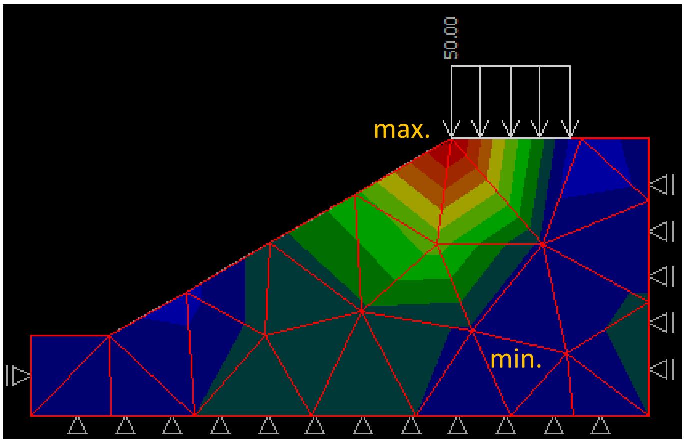


Fig. 5 Contour of equivalent plastic strain in a homogeneous slope by finite element method for slope angle, $\theta = 30^\circ$ and surcharge, $q = 50$ kN/m^2.

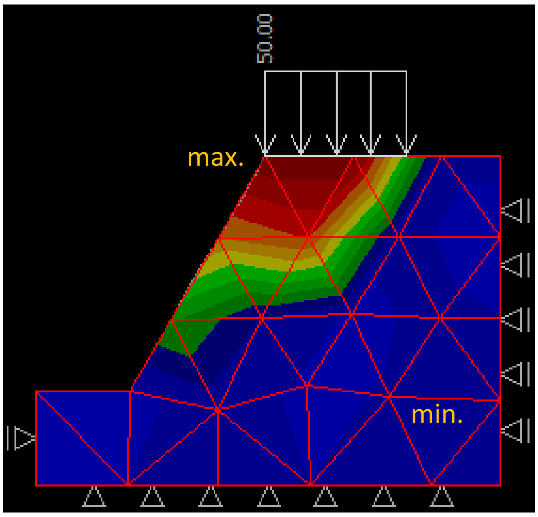


Fig. 6 Contour of equivalent plastic strain in a homogeneous slope by finite element method for slope angle, $\theta = 60^\circ$ and surcharge, $q = 50$ kN/m^2.

CONCUSION

In this study, the effect of surcharge on the stability of slope is studied using the FEM based software GEO5 (2013). The effect of slope angle and slope height is also studied. The evolution of the equivalent plastic potential for different surcharge and slope angle is presented as well. It is observed that the factor of safety decreases as the slope angle increases and the decrease pattern is almost similar regardless of the values of the surcharge. The factor of safety also decreases with the increases of slope height up to a certain limit and above that limit; increase in the slope height has no influence on the factor of safety for a particular slope angle and surcharge.

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MODELING THE UNDRAINED BEHAVIOR OF GRANULAR MATERIALS FROM DRAINED TEST USING DEM

M. M. Sazzad[1*], M. A. Habib[1] & R. Karim[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi, Bangladesh,*
**Corresponding Author, <mmsruet@gmail.com>*

ABSTRACT

This paper aims at modeling the undrained macro- and micro-mechanical behaviors of granular materials from the strain controlled constant volume drained test by two-dimensional (2D) discrete element method (DEM). To model the undrained behavior from the strain controlled constant volume drained test, three isotropically compressed rectangular samples of different void ratios were prepared using the periodic boundaries. Each sample contains 8450 ovals, widths of which range from 1 to 2 mm. The isotropically prepared numerical samples were subjected to the strain controlled constant volume drained test using the periodic boundaries. The numerical data were recorded at regular interval during the simulation. The simulated data from the strain controlled constant volume drained test depicts the similar stress-strain behavior as usually observed in the laboratory based experiments. The evolution of fabric, a microscopic parameter, is also presented. The fabric is quantified using the fabric tensor and its evolution is discussed for samples of different void ratios for the simulation of undrained test modeled by the strain controlled constant volume drained test.

Keywords: Undrained, Drained, Modeling, Fabric, DEM

INTRODUCTION

Modeling the undrained behavior for granular materials is very complex with respect to the interaction between the solid particles and fluid in the pore space. In coupled discrete–continuum system, discrete-continuum structure is disintegrated into solid phase and fluid phase systems. Hydrodynamics of fluid phase is to be incorporated into the model. The equations of motion of solid particles and fluid phase have to be solved simultaneously and individually in a Fluid-coupled DEM. Solid particles move due to the forces from the sample boundaries and/or due to the pressure from the fluid phase. This particle movements cause pore volume changes resulting in the increase or the decrease of the pore pressure. Buoyancy and drag forces are exerted on the particles due to the pressure gradient and fluid flow. The measurement of pore pressure due to the movement of particle is a very complex phenomenon and it requires very tough computer coding. Besides, it is computationally very expensive. Several researchers used the fluid-couple method to model the undrained behaviors (e.g., Zeghal and Shamy, 2004; Fakhimi, 2009). Apart from this complex method, several researchers used an alternative method of simulations of undrained behavior where they avoid the consideration of pore fluid. Instead, they used the strain controlled constant volume drained test to model the undrained behavior on the principal that the volume of sample in an undrained test on fully saturated geomaterials remains constant as a result of low particle and fluid compressibility (e.g., Ng and Dobry, 1994; Shafipour and Soroush, 2008). In the present study, the strain controlled constant volume drained test is used to model the undrained behavior because it is computationally less expensive. So, the objectives of the present study are: (i) to simulated the undrained behaviour of granular materials such as sand using the strain controlled constant volume drained test by DEM and (ii) to explore the evolution of fabric during the undrained test modeled by the strain controlled constant volume drained test. For this purpose, three 2D rectangular shaped samples of different void ratios were prepared using 8450 ovals in each sample by setting the coefficient of interparticle friction to 0.2, 0.3 and 0.4, respectively. The simulated data were compared to validate the applicability of the strain controlled constant volume drained test for undrained simulation and the micro-level data were used to explore the evolution fabric during the undrained tests.

BASIC CONCEPTS OF DEM

DEM is a numerical method used to model the discrete behavior of granular materials. In DEM, each particle is modeled as an element. Either force or velocity is applied on the boundary of the model sample which causes the discrete particles of the model sample to move. The resulting force on a particle is computed and Newton law of motion is used to compute the accelerations. The accelerations are integrated twice with respect to time to yield the velocities and displacements of the particles, respectively. The displacements are used in the force-displacement law to compute the relative increase of forces on a particle. This incremental force is added to the earlier force and the previous process continues for the next time steps. For details of DEM, readers are referred to Cundall and Strack (1979).

SAMPLE PREPARATION AND NUMERICAL SIMULATION

Three numerical samples consisting of 8450 ovals were prepared using the periodic boundaries. In the sample generation phase, ovals were placed on the equally placed grids of rectangular shaped numerical samples without any overlap. The sizes (widths) of ovals (height to width ratio=0.60) varied from 1-2 mm. These samples were subjected to isotropic compression to 100 kPa using the periodic boundaries by varying the interparticle friction coefficient 0.2, 0.3 and 0.4, respectively. The void ratios (e) of the isotropically compressed samples were 0.193, 0.212, and 0.226, respectively. An isotropically compressed numerical sample (void ratio=0.193) with reference axes is shown in Fig. 1 as an example. Computer code OVAL (Kuhn, 2003) was applied both to prepare the sample and to simulate the undrained behavior. A simple contact model, consisting of a spring in the vertical direction and the other spring in the lateral direction, was incorporated. Slipping between particles would occur when the given interparticle friction coefficient was attained. The DEM parameters used in both the sample preparation and the numerical simulation are presented in Table 1. Simulation of undrained behavior modeled by the strain controlled constant volume drained test was carried out by applying equal and opposite strain increment to the vertical and lateral boundaries to keep the volume constant and to account for the fluid incompressibility. A very small strain increment of 2×10^{-4}% was applied during the strain controlled constant volume drained test.

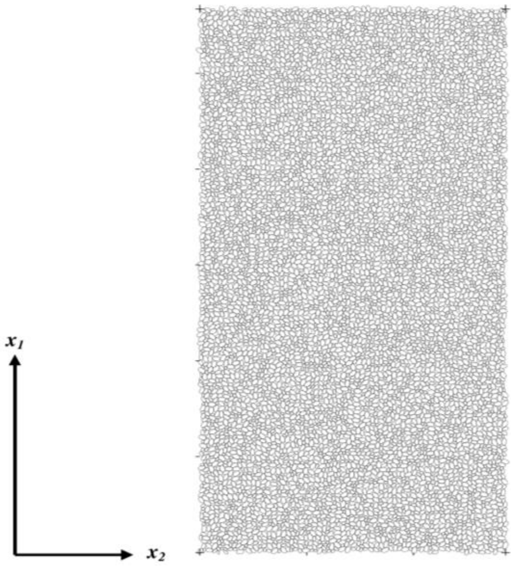

Fig.1 Isotropically compressed sample with reference axes

Table 1 DEM parameters used in the study

DEM Parameters Used	Values
Normal contact stiffness (N/m)	1×10^{8}
Shear contact stiffness (N/m)	1×10^{8}
Particle density (kg/m^3)	2650
Interparticle friction coefficients during isotropic compression	0.2-0.4
Interparticle friction coefficient during shear	0.5
Increment of time step (s)	1×10^{-6}
Damping coefficients	0.05

MACRO-MECHANICAL BEHAVIOR

The simulated undrained stress-strain behavior modeled by the strain controlled constant volume drained test for samples of different void ratios are depicted in Fig. 2 (a) while the evolution of effective stress path is depicted in Fig. 2 (b). The shear stress is defined as $q=(\sigma'_1-\sigma'_2)/2$ and the mean effective stress is defined as $p'=(\sigma'_1+\sigma'_2)/2$, where σ'_1 and σ'_2 are the effective stresses in $x_1-$and x_2-directions, respectively. Dilation is observed in dense sample ($e=0.193$) resulting in the increate in the effective stress while contraction is observed in loose sample ($e=0.226$) resulting in the decrease in the effective stress as observed in Fig. 2 (a). The effective stress path for the loose sample ($e=0.226$) moves towards the origin indicating the liquefaction as noticed in Fig. 2 (b). The simulated macroscopic behaviors depicted here for different void ratios are pretty similar to that observed in experimental studies (e.g., Yoshimine et al., 1998).

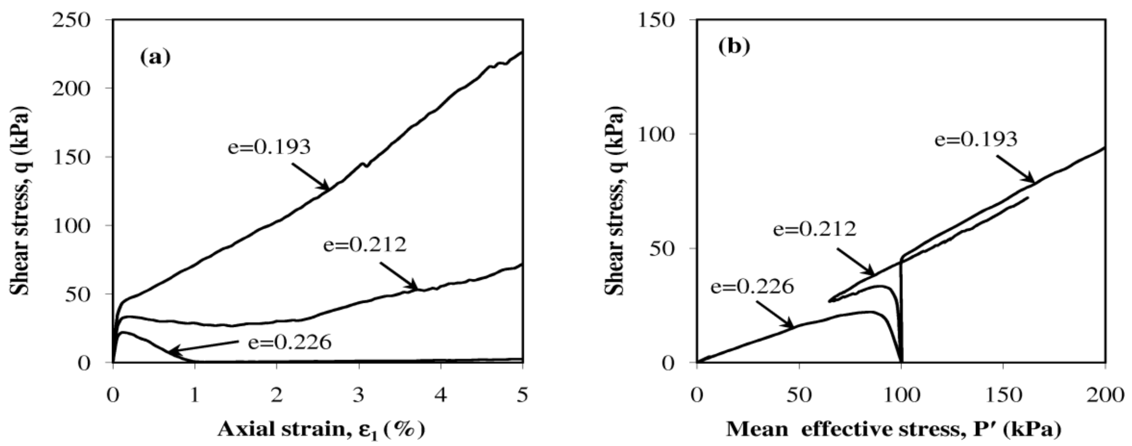

Fig.2 (a) Relationship between (a) the shear stress and axial strain ; (b) shear stress and mean effective stress for different void ratios during undrained shear

EVOLUTION OF FABRIC

The fabric of a granular system is often represented by the distribution of contact normals (e.g., Oda et al., 1985). Such representation of fabric is referred to as contact fabric. A contact normal is defined as a vector perpendicular to the contact plane between two contacting particles. In the present study, contact fabric is quantified using a fabric tensor based on the contact normal vectors as follows (Satake, 1982):

$$H_{ij} = \frac{1}{N_c}\sum_{i=1}^{N_c} n_i^c n_j^c \qquad (1)$$

where N_c is the total number of contacts in the system and n_i^c is $i-$th component of the contact normal vector at the $c-$th contact. The contact fabric can be further classified on the basis of strong contacts. A contact is said to be strong if it carries a contact force greater than the average contact force. Based on the strong contacts, another fabric tensor can be defined as follows (Sazzad and Suzuki, 2013):

$$H_{ij}^s = \frac{1}{N_c}\sum_{s=1}^{N_s} n_i^s n_j^s \qquad (2)$$

where N_s is the total number of strong contacts in a set of total contacts N_c and n_i^s is the $i-$th component of the contact normal vector at the $s-$th strong contact. The evolution of fabric measure considering all contacts defined as $H_q = (H_{11} - H_{22})/2$ with axial strain during undrained shear for samples of different void ratios is depicted in Fig. 3 while the evolution of fabric measure considering strong contacts defined as $H_q^s = (H_{11}^s - H_{22}^s)/2$ with axial strain during undrained shear for samples of different void ratios is depicted in Fig. 4. It should be noted that both H_q and H_q^s increases with the increase of strain irrespective of the void ratio of the sample. It should pointed out that both H_q and H_q^s increases with axial strain even though the effective shear stress decreases with axial strain for loose sample ($e = 0.226$). Note also that, evolution tendency of neither H_q versus ε_1 curve nor H_q^s versus ε_1 curve has any similarity with the q versus ε_1 curve. However, similarity between H_q^s versus ε_1 curve and q versus ε_1 curve is usually noticed for drained tests (e.g., Sazzad et al., 2013). This study suggests that simulation of undrained test using the strain controlled constant volume drained test cannot replicate the stress ratio or deviatoric stress versus axial strain relationships considering only the contact fabric.

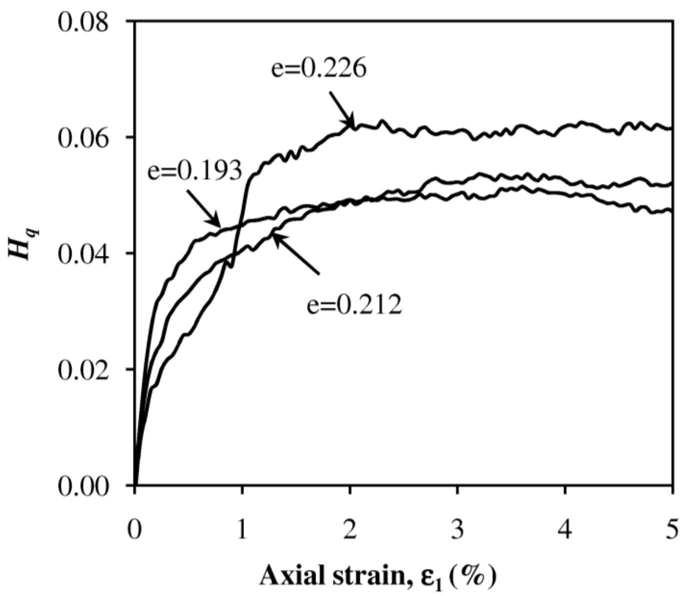

Fig.3 Evolution of fabric measure H_q with axial strain during undrained shear for samples of different void ratios considering all contact

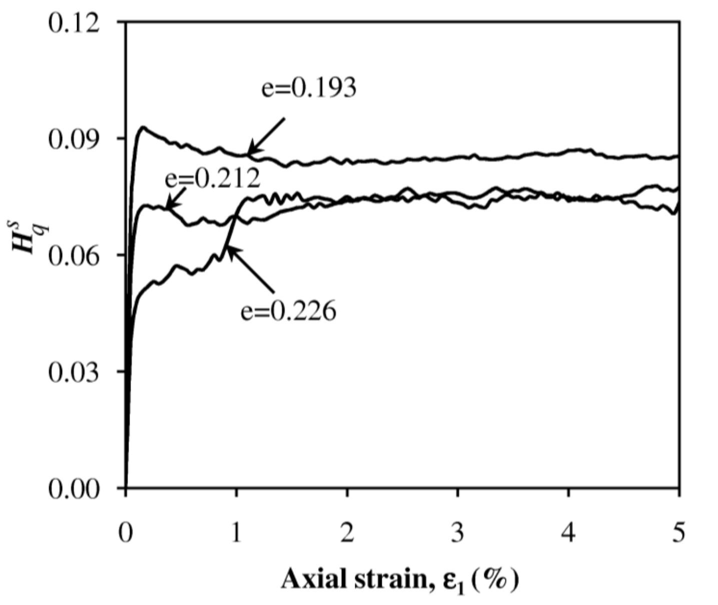

Fig.4 Evolution of fabric measure H_q^s with axial strain during undrained shear for samples of different void ratios considering strong contacts

CONCLUSION

Undrained behavior is modeled using the strain controlled constant volume drained test considering samples of different void ratios by DEM. The study depicts that the simulated macroscopic behavior is fairly similar qualitatively to that observed in the laboratory based experiments. This reveals that the strain controlled constant volume drained test can replicate the macroscopic behaviors, at least in a qualitative sense. The evolution of fabric is also examined. The fabric is quantified by two fabric tensors based only on the contact normal vectors for all and strong contacts. Fabric measures continue to evolve with strain as usually observed in drained test considering all and strong contacts. However, no similarity in the evolution tendency is noticed between the fabric measure versus axial strain curve with the effective shear stress versus axial strain curve even when strong contacts are considered.

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SIMULATION OF UNDRAINED BEHAVIOR OF GRANULAR MATERIALS BY CONSTANT VOLUME METHOD

M. M. Sazzad[1], M. A. Habib[1*] & R. Karim[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi, Bangladesh.*
**Corresponding Author, <ahsan090063@gmail.com >*

ABSTRACT

The aim of present paper is to simulate the undrained behavior of granular materials such as sand by two dimensional (2D) discrete element method (DEM) using the constant volume method. A numerical sample consisting of 8450 ovals was generated in a rectangular frame without any overlap. The generated sample at this stage was very sparse. Three different samples of different initial void ratios prior to shear were prepared by varying the interparticle friction coefficient during the isotropic compression stage using the initially generated sparse sample. The isotropically prepared numerical samples were then subjected to undrained shear using the constant volume method by 2D DEM. The digital data were recorded at regular interval and the post-analysis was carried out. It is noted that the simulated macroscopic results are in good agreement to that usually observed in laboratory based experiments. Different micro-parameters such as the coordination number and slip coordination number were monitored and their connections with the macro-data were reported.

Keywords: Undrained, Granular materials, Simulation, DEM

INTRODUCTION

Soil is a multi-phase particulate system consisting of solid particles and water or air or both in pore spaces. When a particulate system with fully saturated condition undergoes loading, it may change from a looser state to a denser state which is referred to contraction or it may change from a denser state to a looser state which is referred to as dilation. During contraction as obvious in a loose sample, the pore water pressure continues to accumulate. This increase in pore water pressure forces the particles to move apart and consequently, the contact loss occurs which resulting in the decrease in the effective stress. The cumulative build up of pore water pressure may even lead the effective stress path to move to the origin, i.e., the effective stress may even reduce to zero value. The soil at this state is said to be in initial liquefied state (Seed and Lee, 1966). During dilation as obvious in a dense sample, the pore water pressure decreases causing a regain of strength during loading. Thus, it is obvious that soil under undrained condition exhibits a very highly complicated nonlinear response. The global soil mass response at the boundary under undrained condition can be evaluated and measured using the laboratory based experimental devices. However, the particle-scale behavior during undrained condition is hardly possible to extract using the conventional experimental devices. Indeed, advanced instrumental facilities and methods such as photo-imaging analysis (Oda and Konishi, 1974), X-ray tomography (Lee et al., 1992), electrical resistance or wave velocity measurement (Arulmoli and Arulanadan, 1994; Santamarina and Cascante, 1996) and magnetic resonance imaging (Ng and Wang, 2001) can be used. However, they are expensive, sophisticated and all required particle-scale behaviors can not be extracted using these methods and advanced devices. Alternatively, numerical methods such as DEM (Cundall and Strack, 1979) can be used. The major advantage of DEM is that one can monitor and extract necessary particle-scale data at any desired strain for the post-process and analysis.

The objectives of the present study are: (i) to simulated the undrained behaviour of granular materials such as sand using the DEM and (ii) to extract and discuss the evolution of particle-scale variables during the undrained shear by the constant volume method. Constant volume method is used because it reduces the computational time of simulation; otherwise, huge time is necessary in fluid-couple DEM. A 2D rectangular shaped sample was first generated with 8450 ovals and then, using this initial sample, three different samples of different void ratios were prepared by setting the coefficient of interparticle friction to 0.2, 0.3 and 0.4, respectively. The simulated data were compared with the experimental data qualitatively and the evolution of particle-scale variables such as the coordination number and slip coordination number were presented.

FUNDAMENTAL IDEA OF DEM

DEM is a research tool used to model the discrete behavior of granular materials and understand their micro-mechanics. In DEM, the dynamic motions are solved adopting an explicit time-stepping scheme. Either force or velocity is applied on the boundary of the model sample. The external force or velocity causes the discrete particles of the model to move. If force is applied at the boundary, the resulting force on a particle is computed. Newton law of motion is then used to compute the accelerations. The accelerations are integrated twice with respect to time, which yields the velocities and displacements of the particles, respectively. The incremental displacements are computed and used in the force-displacement law to compute the relative increase of forces on a particle. Thus, the cycle continues for the next time steps. Although the material of each particle used in DEM is elastic, the overall behavior is inelastic due to the dissipation of energy through the relative motions (sliding, rolling) between particles.

PREPARATION OF NUMERICAL SAMPLES

The numerical sample was prepared in two steps namely, sample generation and isotropic compression. In the sample generation phase, the oval shaped particles were placed on the equally placed grids of rectangular shaped numerical samples without any overlap. The sizes (widths) of ovals varied from 1-2 mm while the height to width ratio of ovals was kept constant (0.60). The sample at this stage was very loose. The numerically generated initial sample was subjected to isotropic compression to 100 kPa using the periodic boundaries. Three different samples were prepared form the initially generated sample by varying the interparticle friction coefficient μ (0.2, 0.3 and 0.4). μ is defined as $\mu = \tan\phi_\mu$, where ϕ_μ is the interparticle friction angle. The void ratios (e) of the three samples after the end of isotropic compression to 100 kPa were 0.193, 0.212, and 0.226, respectively. An isotropically compressed sample with reference axes is shown in Fig. 1. Computer code OVAL (Kuhn, 2003) was used to prepare the sample. In OVAL, a simple contact model consisting of two springs, one in vertical direction and the other in lateral direction, was incorporated. Slipping between particle would occur as soon as the given interparticle friction coefficient was attained. The parameters used in the numerical simulation are given in Table 1.

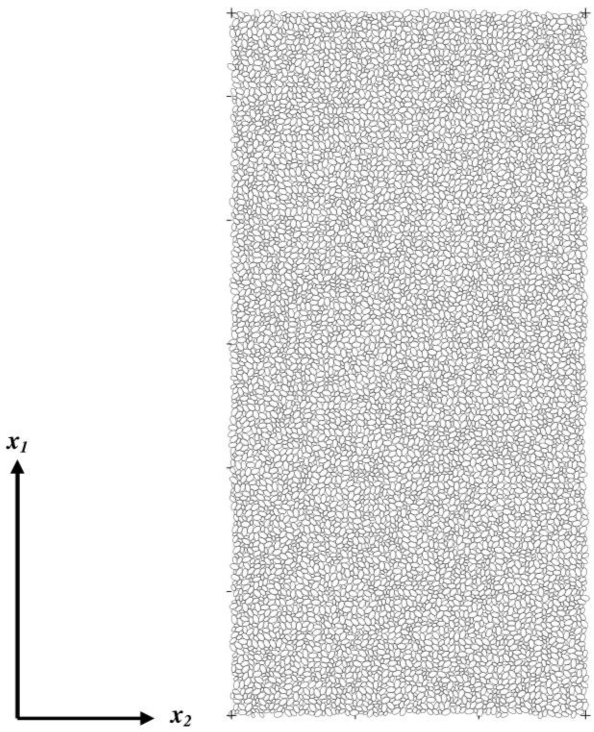

Fig.1 Isotropically compressed sample with reference axes (void ratio=0.193)

Table 1 DEM parameters used in the simulation

DEM Parameters	**Value**
Normal contact stiffness (N/m)	1×10^{8}
Shear contact stiffness (N/m)	1×10^{8}
Particle density (kg/m^3)	2650
Interparticle friction coefficients during isotropic compression	0.2-0.4
Interparticle friction coefficient during shear	0.5
Increment of time step (s)	1×10^{-6}
Damping coefficients	0.05

SIMULATION BY CONSTANT VOLUME METHOD

Undrained test on fully saturated geomaterials is generally known as constant volume test as a result of low particle and fluid compressibility (Shafipour and Soroush, 2008). Considering this principle, many researchers presumed that the drained strain controlled constant volume test is equivalent to an undrained test (e.g., Ng and Dobry, 1994). The major advantage of this method is that it reduces the computational time of the simulation. In the present study, the constant volume method is used to simulate the undrained behavior of granular materials using the DEM. Simulation of undrained behavior was carried out by maintaining the volume constant to account for the fluid incompressibility. Constant volume of the drained test was achieved by applying equal and opposite strain rate to the lateral boundary to that of the vertical boundary. A very small strain increment of 2×10^{-4}% was applied to maintain the quasi–static condition during the simulation.

MACROSCOPIC RESPONSES

Fig. 2 (a) depicts the simulated stress-strain behavior of undrained test by constant volume method for samples of different void ratios while Fig. 2 (b) depicts the evolution of stress path. The shear stress is defined as $q=(\sigma'_1-\sigma'_2)/2$ and the mean stress is defined as $p'=(\sigma'_1+\sigma'_2)/2$. Here, σ'_1 and σ'_2 are the effective stresses in $x_1-$and x_2-directions, respectively. The dense sample ($e=0.193$) exhibits dilation resulting in the increate in the effective stress while loose sample ($e=0.226$) exhibits contraction resulting in the decrease in the effective stress as observed in Fig. 2 (a). The effective stress path of the loose sample ($e=0.226$) moves towards the origin indicating the liquefied state as noticed in Fig. 2 (b). The simulated behavior reported in this study for undrained test by constant volume method for different void ratios are quite similar and consistent to that observed in experimental studies (e.g., Yoshimine et al., 1998). This demonstrates the versatility and capability of DEM to capture the undrained behaviors qualitatively.

Fig. 3 depicts the evolution of pore water pressure from the simulated undrained test by constant volume method for different void ratios. The pore water pressure is calculated indirectly as follow (Shafipour and Soroush, 2008): pore water pressure, $u=\sigma'_{2(ini)}-\sigma'_2$, where $\sigma'_{2(ini)}$ = effective stress in lateral direction (x_2-direction) at the initial stage prior to shear and σ'_2 is the effective stress in lateral director at any given time step. The pore water pressure in case of dense sample ($e=0.193$) decreases and even becomes negative due to dilation during undrained shear. By contrast, loose sample ($e=0.226$) depicts an increase in the pore pressure due to contraction during undrained shear. The increase in pore water pressure results in the decrease in the effective stress as obvious in Fig. 2 for loose sample.

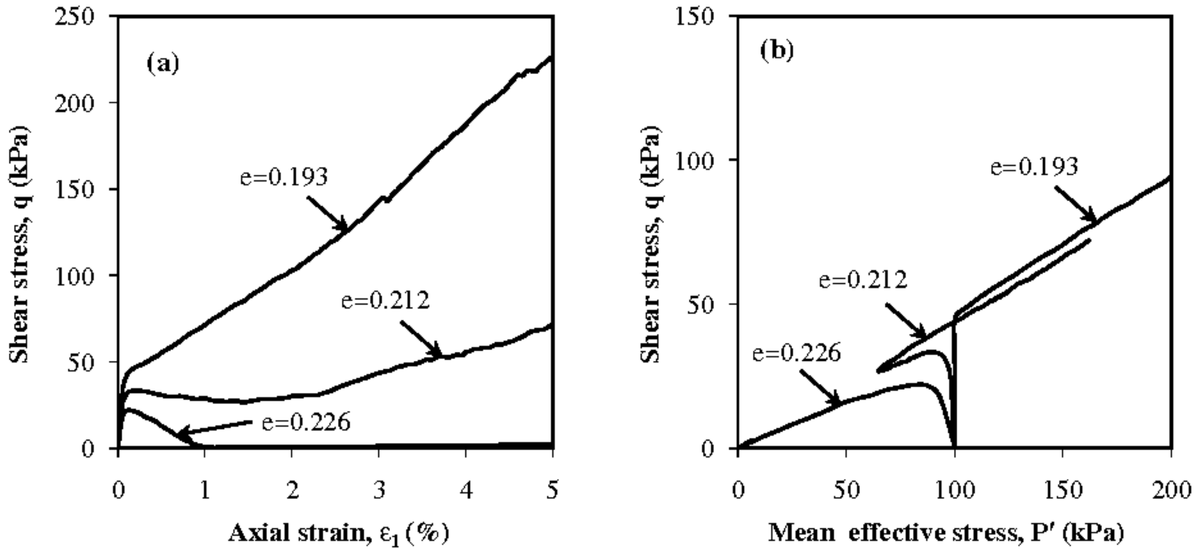

Fig.2 (a) Relationship between the shear stress and axial strain for different void ratios during undrained shear; (b) Relationship between the shear stress and mean effective stress for different void ratios during undrained shear

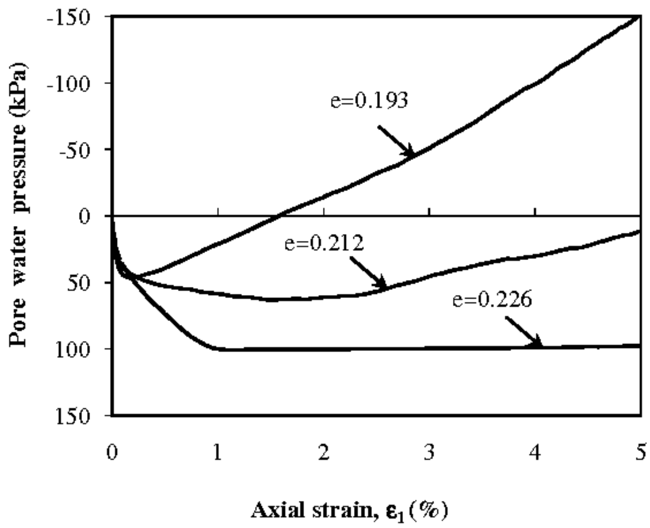

Fig.3 Pore water pressure builds up during undrained shearing

PARTICLE-SCALE RESPONSES

The evolution of different particle-scale parameters such as the coordination number and the slip coordination number is depicted in Fig. 4. The coordination number is defined as $Z_m = 2N_c/N_p$ and the slip coordination number is defined as $Z_s = 2N_s/N_p$, where N_c, N_s and N_p are the total number of inter-particle contact, slip contact and particles, respectively. Note that, void ratio significantly affects these behaviors. Z_m decreases as the void ratio increases and interestingly, the evolution tendency is similar to that usually observed in drained test for dense sample. Note also that Z_m is above 3 regardless of the void ratio of the sample, even when the sample is at the verge of liquefaction. Z_m at $\varepsilon_1 = 1\%$ (at liquefaction) for sample having $e = 0.226$ is 3.31 (minimum one). On the other hand, Z_s suddenly peaks up at $\varepsilon_1 = 1\%$ for sample having $e = 0.226$ (i.e., at the state of liquefaction). This indicates that the maximum number of slipping occurs when the sample is at the state of liquefaction.

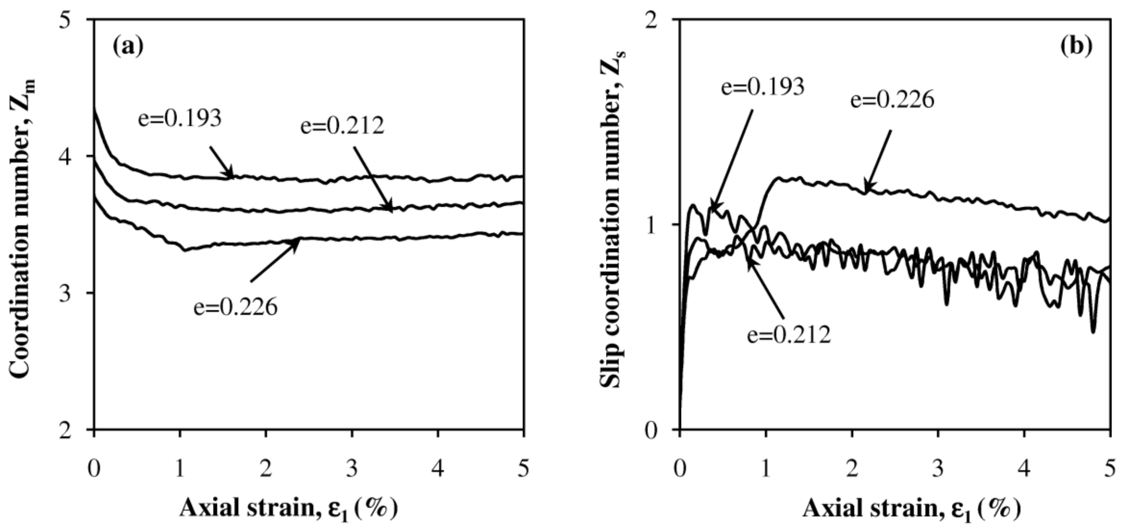

Fig.4 Evolution of (a) Coordination number and (b) Slip coordination number during undrained shear for samples of different void ratios

Note also that the behavior of Z_s for sample having $e = 0.226$ is quite different than that for sample having $e = 0.212$ and $e = 0.193$. Once the sample is liquefied or at the verge of liquefaction, the slipping contacts dramatically increases and the behavior continues, indicating an unstable micro-structural evolution. The minimum Z_m and the maximum Z_s occurs almost at the same state of strain (i.e, at the liquefied state). This indicates a strong connection between the particle-scale parameters to the macro-scale behaviors.

CONCLUSION

Simulation of undrained behavior was carried out for three different samples under the constant volume method. The simulated macro behaviors show excellent qualitative agreement with the laboratory based experimental results. The evolution of different particle-scale variables have also been reported and discussed. It is noted that the evolution tendency of coordination number and slip coordination number is different. Slip coordination number for liquefied sample evolves in a different manner than that for other samples. Note also that minimum coordination number and the maximum slip coordination occurs simultaneously at the liquefied state. The coordination number remains above 3 even when the sample is at liquefiable state. The study clearly illustrates that the macro behaviors at undrained condition is significantly influenced by the particle-scale parameters.

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EFFECTIVENESS OF SAND COLUMN AS A GROUND IMPROVEMENT TECHNIQUE

Md. Ikramul Hoque[1]* & Mir Abdul Kuddus[2]

[1] *Khulna University of Engineering & Technology, Bangladesh.*
[2] *Department of Civil Engineering, Khulna University of Engineering & Technology, Bangladesh.*
Corresponding Author, <ikramul3300@gmail.com>

ABSTRACT

This paper is concerned with the performance of granular pile as ground improved technique at a selected site of South-West region of Bangladesh. Ground improvement by granular piles is considered as one of the versatile and cost effective ground improvement method. Installation of granular piles transforms the soils into a stiffer composite mass with intervening native soil proving lower overall compressibility and higher shear strength. In the recent years this technique has also been adopted in Bangladesh in various projects to improve the marginal sites. The installation technique has the big influence on the performance of this ground improvement method. There is a record of successful application of rammed-displacement method in the installation of granular piles in the soft ground of Bangladesh. The sub-soil consists of fine-grained soil of very soft to soft consistency up to the great depth. The more one sand pile installation in this study, sylhet sand. Granular piles of 300 mm diameter and 8.25 m length were installed in single. Load tests were conducted on 0.30m diameter plate resting at a depth of 0.91m from the existing ground surface on both the natural and improved ground. The plate load test was conducted on the natural ground at the foundation depth and on the top of sand piles after one month and one year of installation of granular piles. Standard penetration tests were also conducted to observe the change in soil stiffness due to the installation of sand pile. The test results reveal that the bearing capacity of the normal ground was increased by the installation of granular piles. Comparing to the natural ground, the bearing capacity of improved ground was increased by 200% to 250%. Field investigation reveals that the granular pile made-up of locally available granular materials and employed installation technique can be used successfully as a suitable ground improvement method to improve the bearing capacity of such soft ground.

Keywords: Bearing capacity, Granular materials, Rammed displacement, Soft ground.

1. INTRODUCTION

Construction of granular piles is considered as one of the most versatile and cost effective ground improvement technique compared to the other methods such as preloading, dredging and replacement, dynamic compaction etc. Granular piles, a fairly recent ground improvement technique, have been used successfully in several projects throughout the world. Granular piles are generally cylindrical in shape and composed of compacted gravel, crushed stone or sand. Various techniques of installation have been conceived depending on technical ability, efficiency and local condition. In Europe and USA, the vibro-floatation technique is widely used while in Japan, the vibro-compozer method is used. In India, granular piles are constructed by simple bored piling equipment. In this study field investigation is carried out on the construction of granular piles by rammed-displacement and its various aspects. Granular piles with different types of granular materials were constructed in a typical soft ground of south western region of Bangladesh. Load tests were done over the constructed granular piles and the results show that the bearing capacity of the improved ground increased by 2.00 to 2.50 times than that of the natural ground.

One of the oldest historical examples of the use of granular piles is found in the 1830's where French Military Engineers used it to support heavy foundations of iron works at the artillery arsenal in Bayoune. The modern origin of the granular piles actually began 60 years ago in the 1930's in Germany by their Russian Émigré. After the beginning of the modern phase of the use of granular piles, the theoretical background, analysis and design aspects, and installation techniques have been developed by various researchers and the practicing engineers. As a result in the recent years, this method of ground improvement has been proved to be a most popular versatile and cost effective technique. The vibro-compaction method is used to improve the density of cohesion less soil using a vibro-flot which sinks in the ground under its own weight and with the assistance of water and vibration (Baumann and Bauer 1974). The vibro-replacement method is used to improve cohesive soils with more than 18% passing no. 200 U.S standard sieve. The vibro-compozer method is popularized in Japan and is used for stabilizing soft clays in the presence of high ground water level (Aboshi et al 1979).

A large number of laboratory and field tests have been conducted in order to quantity the applicability of this ground improvement technique to improve the behavior of soft ground. In Bangladesh sand piles have been used successfully in some ground improvement projects. Alamgir and Zaher (1999a and 1999b) reported that a large number of sand piles were installed in soft cohesive soils in south-western region of Bangladesh where a six-vent regulator was constructed. Some research works were conducted in the department of civil engineering, Khulna University of Engineering & Technoloogy to see the response of soft ground improved by columnar inclusion as a part of post graduate research (Zaher 2000, Sobhan 2001, Hossain 2009).

1. Construction of Granular Pile

The sub-soil condition of Khulna University of Engineering and Technology (KUET) campus at Khulna, Bangladesh in which granular piles were installed and the installation methods are described in the following sections. Locally available granular materials and the manual labor intensive installation technique were employed. To obtain a vivid picture on the effectiveness, granular piles were considered.

1.1 Sub Soil Condition

From most of the past records, it is found that the sub-soil of KUET campus consists of soft soil layers containing organic. Sub-soil investigation was done and index properties of soil were determined at different layers. For determining the sub-soil properties Standard Penetration Test (SPT) was performed at selected site which is situated at the place of KUET guest house cum club building. The sub-soil stratification and the soil properties were investigated by means of Standard Penetration Test. The Standard Penetration Test (SPT) developed in 1927, is currently the most popular and economical means to obtain the sub-surface information. It is estimated that 85% to 90% of conventional foundation design in North and South America is made using the sub-soil condition determined by SPT (Bowles 1988). In sensitive silty clay and clayey silts, it is often difficult to determine the true soil properties by conventional field investigation methods. In such cases in-situ methods such as CPT sounding, dilatometer or pressure meter tests often give the most reliable results. However, SPT is employed here due to non availability of appropriate method for soft soil investigation such as CPT test, dilatometer test and field vane shear test etc. It can be noted here that the SPT test still the most popular field test in Bangladesh to determine the sub-soil profile.

1.2 Installation of Granular Pile

The granular piles were installed by rammed-displacement method in dry process. The dry method is frequently used to construct columnar inclusions through weak soils in developed areas because of the problems associated with the acquisition, retention and disposal of significant amount of water. The rammed-displacement type construction method employed in this project using simple technique was found suitable for this site consists of soft fine grained soil deposits having high water table. Sylhet sand are usually used as granular materials to construct sand piles of an average diameter 300mm of 8.5m length. Compaction is done by a dropping hammer of weight 1000kg.

The granular piles were installed here by rammed-displacement method. The construction sequences are described in the following statements.

Step-1. A two end open casing pipe, 300mm in diameter and 7m long was placed vertically at the designed point on the natural ground surface for sand pile construction.

Step-2. The casing pipe was then inserted vertically into the ground about 300mm to 450mm depth at its own weight just by applying some pressure manually. At first a plug is made by the designated sand up to 750mm of casing pipe at bottom level.

Step-3. The rammed-hammer 250mm in diameter, 3.0m long and weight 1000kg was placed inside the casing pipe. The rammed-hammer displaced the soil from beneath the casing pipe and the casing pipe was driven by its own weight till reached the designated position (depth) into the ground. Here one casing pipe of 7m long was driven inside the ground.

Step-4. After reaching the designated depth, the sand plug is broken by providing excess energy then the rammed-hammer is withdrawn from the casing pipe.

Step-5. Casing pipe was then lifted up by about 1m from its original bottom position. The designated granular materials were poured into the hole about 1m layer thickness measured from the bottom. The poured granular materials was then densified by rammed-hammer till the required compactness achieved.

Step-6. Casing pipe was then withdrawn from inside the ground that left the bottom portion of the hole unsupported and the top portion supported by the casing pipe. It was observed that the bottom portion of the hole standing safely without any lateral support.

Step-7. Then hole was poured by the selected granular materials in layers and hence 10 to 15 drops compacted each layer was densified by rammed-hammer till the designated compactness was reached. In general, the thickness of each layer was about 1.0m and 0.65m to 0.75m before and after densification respectively and the free fall height of the rammed-hammer was 0.75m to 1.0m.

Step-8. After the top of granular piles were reached about 1.0m to 1.5m below the ground surface the casing pipe was withdrawn and left the remaining hole unsupported.

Step-9. Then step five (v) was continued until the granular piles were constructed up to the ground level.

In this investigation, to depict the effectiveness of granular pile, granular piles of same properties in terms of diameter, length and stiffness were installed in group pattern. Same spacing was used for the group granular pile as 900mm center to center.

3. Load Test

The measurement systems are very similar to that of follow in standard pile load and plate load tests. In natural ground, the load intensity was increased from 67.17 kN/m^2 to 671.10 kN/m^2 at an equal interval of 67.17 kN/m^2. In each load, the settlement were measured till the rate of deformation less than 1.5 mm/hr. From this figure, it can be seen that maximum settlement observed as 18.00 mm at a load intensity of 671.10 kN/m^2. Similarly, load intensities on the footing resting one month after sand pile installation and one year after sand pile installation of treated ground. From this figure 4, it can be seen, that maximum settlement observed as 8.20mm and 7.00mm at a load intensity of 671.10 kN/m^2. The total time required for this observation was 650 minutes.

Figure2. Load arrangement for footing load test

Figure 3. Load test arrangement

4. Result and Discussion

The test results obtained in the field by plate load test were compared in this section. The improvement of load carrying capacity due to the installation of granular piles in different conditions were compared with that of natural ground. Comparison is also made among the different improvement conditions.

4.1 One month after sand pile installation

The plate load test on the sand pile was performed immediately i.e one month after sand pile installation. The improvement of natural ground due to the installation of single granular pile is shown in Figure 4 in a load intensity versus settlement diagram. This figure shows that the different settlements at same carrying load (671.70kN/m^2), The load carrying capacity for the natural ground is 671.70kN/m^2 and settlement is 18.00mm, while it is 671.70kN/m^2 for single sand pile and settlement is 8.20mm. The result shows that the installation of granular piles increased the bearing capacity of natural ground significantly irrespective of the materials. The plate load test after one month of installation sand pile, the load carrying capacity is 2.0 times higher for sand piles comparing the natural ground for the allowable settlement.

4.2 One year after sand pile installation

The plate load test on the sand pile was performed one year after sand pile installation. The improvement of natural ground due to the installation of single granular pile is shown in Figure 4 in a load intensity versus settlement diagram. This figure shows that the different settlement at same carrying load (671.70kN/m2). The load carrying capacity for the natural ground is 671.70kN/m^2 and settlement is 18.00mm, while it is 671.70kN/m^2 for single sand pile and settlement is 7.00mm. The result shows that the installation of granular piles increased the bearing capacity of natural ground significantly irrespective of the materials. The plate load test after one year of compaction sand pile, the load carrying capacity is 2.50 times higher for sand piles comparing the natural ground for the allowable settlement.

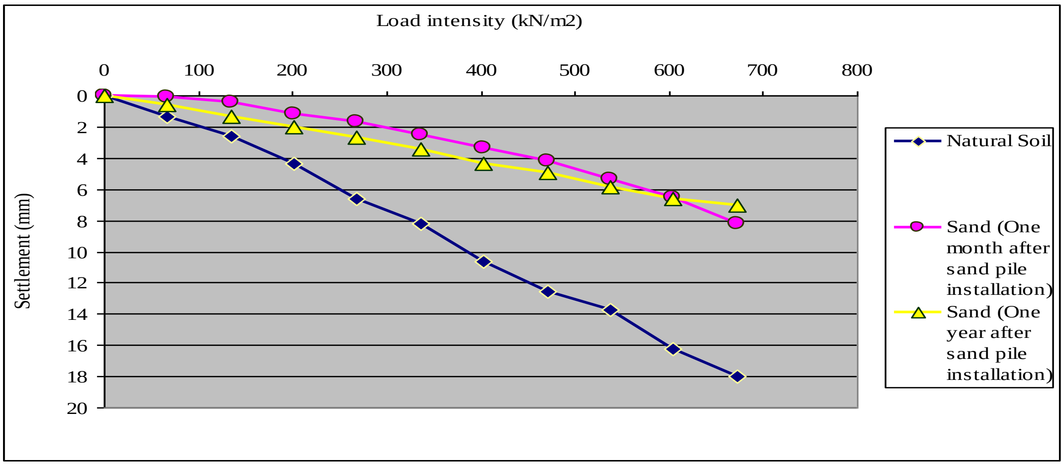

Figure 4. Load-settlement response of footing resting on both the natural and improved ground

4.3 Summary of the plate load test result

The plate load test reveals that the bearing capacity of the improved ground increased significantly due to the installation of granular piles. The arrangement of granular piles, installation pattern and the ratio of bearing capacity of treated (q_t) and natural ground (q_n) are shown in Table 1. In this table q_t and q_n represent the load intensity that the treated and natural ground can carry, respectively, during the plate load test for the different settlement at same carrying load. The measure results also show that the installation sand pile and load test immediately after sand pile installation increases the bearing capacity by 2.00 times than that of natural ground corresponding to 7mm settlement. The results also show that the installation of sand pile and load test one year after sand pile installation increases the bearing capacity by 2.50 times than that of natural ground corresponding to 7mm settlement.

Table 1 Measured load carrying capacity of sand pile yields the largest increment of load carrying capacity.

No. of Test	Description of granular piles	Load test after sand pile installation	Location of plate	q_t/q_n (Corresponding to 7.00 mm settlement)
1	Sand piles	After one month	Top of pile	2.00
2	Sand piles	After one year	Top of pile	2.50

5. Comparison of Result Obtained from SPT

Standard Penetration Tests were performed on the improved ground for different conditions. These are (i) The boring immediately after sand pile installation and (ii) The boring one year after sand pile installation. The Standard Penetration Test results are given in the following sections for different improved conditions.

5.1 One month after sand pile installation

Standard Penetration Test was performed in two bore holes on improved ground to depict the improvement of soft ground along the depth due to the installation of granular piles. The penetration resistance i.e. N-values obtained in two boreholes are compared with those of natural ground before granular piles installation. The comparison is shown in Figure 5 This Figure shows that N-values ranges 3 to 9. for the natural ground, while the values increases to 4 to 15 and 5 to 24 for the bore holes one and two respectively, in case of improved ground. The N-values of improved ground of depth 12 m to 15 m decreased slightly from the N-values of natural soil which is an unusual phenomenon of this study. During the installation of sand pile, the soil surrounding the pile is disturbed due to continuous hammering and then compressible soil is lost their capacity. The inadequate pouring of sand, lack of compaction as well as dispersion of sand in the surrounding soil during hammering may be the causes of producing lower N-values on the sand pile of improved ground when compared to the untreated one.

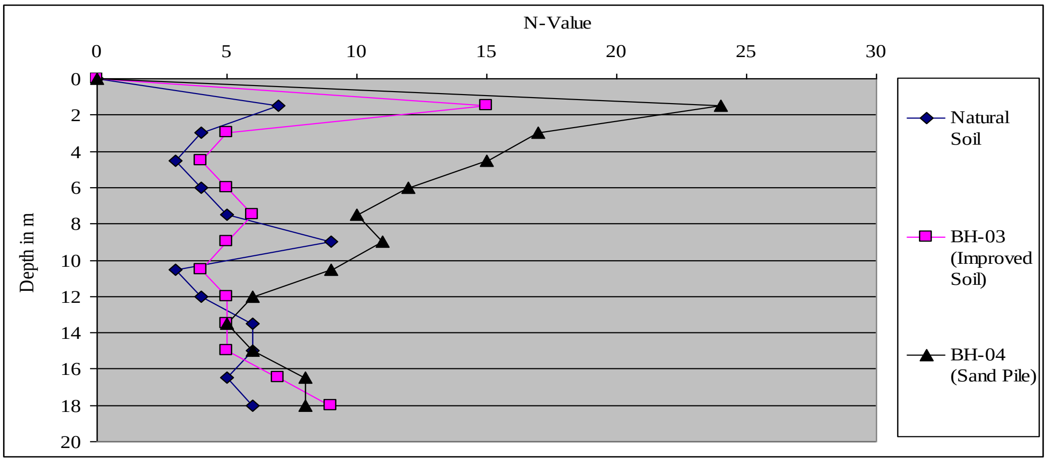

Figure 5. Comparison of SPT result treated and natural ground (One month after sand pile installation)

5.2 One year after sand pile installation

Standard Penetration Test was performed in two bore holes on improved ground to depict the improvement of soft ground along the depth due to the installation of granular piles. The penetration resistance i.e. N-values obtained in two boreholes are compared with those of natural ground before granular piles installation. The comparison is shown in Figure 6. This figure shows that N-values ranges 3 to 9. for the natural ground, while the values increases to 4 to 12 and 5 to 19 for the bore holes three and four respectively, in case of improved ground. The N-values of improved ground of depth 12 m to 15 m decreased slightly from the N-values of natural soil which is an unusual phenomenon of this study. During the installation of sand pile, the soil surrounding the pile is disturbed due to continuous hammering and then compressible soil is lost their capacity. The inadequate pouring of sand, lack of compaction as well as dispersion of sand in the surrounding soil during hammering may be the causes of producing lower N-values on the sand pile of improved ground when compared to the untreated one.

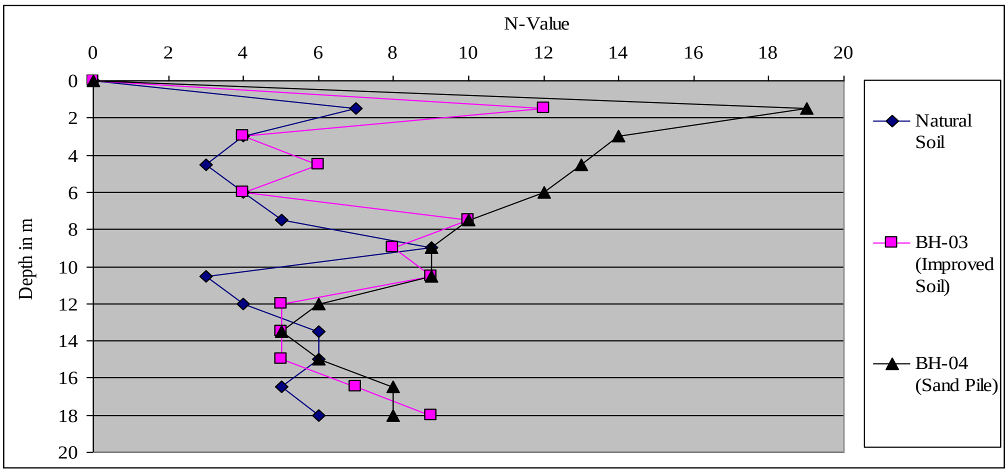

Figure 6. Comparison of SPT result treated and natural ground (One year after sand pile installation).

6. Conclusions

Based on this study the following conclusions can be made:

1. The soft ground improvement using granular piles technique is revealed as fast, economical and an efficient method to improve weak soil.
2. Comparing to the natural ground, the bearing capacity of improved ground was increased by 200% to
250%.
3. The field investigation on the improved ground by plate-load test revealed that the granular piles improved substantially the bearing capacity of the natural soil.
4. The field experience during the installation of granular pile depicts that such installation technique required very close monitoring and precaution in case of group pile arrangement.
5. Proper FM of sand and pouring of granular materials are required to ensure the stiffness of granular pile. The layer thickness of granular material, dropping height, number and placement of hammer are to be maintained properly through close field monitoring to achieve the designed stiffness of the granular pile.
6. The better increments of bearing capacity by granular pile are observed than that of natural soil.
7. The practicing engineer can get help from this study and experience to improve the soft ground by the installation of granular piles

Acknowledgments

This research was supported by the author admits the worthy recommendations and advise to Dr. Quazi Sazzad Hossain, Professor, Department of Civil Engineering, KUET and Mir Abdul Kuddus, Assistant Professor, Department of Civil Engineering, KUET for their unremitting encouragement and co-operation at every stage of his dissertation works. Special thanks to laboratory technician, Geotechnical Engineering Laboratory, KUET and also all laboratory technicians, Geo-environmental Engineering Laboratory, KUET for their assistance during working in the laboratory.

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DESIGN OPTIMIZATION OF FOUNDATIONS USING GAS

Islam, M.S.[1] & Rokonuzzaman, M.[1]

[1] *Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh*
Corresponding Author, <sumon2k8@yahoo.com>

ABSTRACT

The design of foundation is done in two different stages: geotechnical design and then structural design. The geotechnical design of foundations focuses on three issues: bearing failure, settlement limit and economy. Most of the design concentrates on bearing and settlement criteria ignoring the economy most of time. This paper introduces an optimized design process of footing foundation using genetic algorithms (GAs) in which the objective function is construction cost with the design parameters and design requirements as the optimization variables and constraints, respectively. The design process is illustrated using examples of shallow isolated column footing in sands. Sensitivity analysis on the soil parameters and design requirements is evaluated to find out the key parameter (s) to control the construction cost.

Keywords: Foundation, Design optimization, Cost, GAs

INTRODUCTION

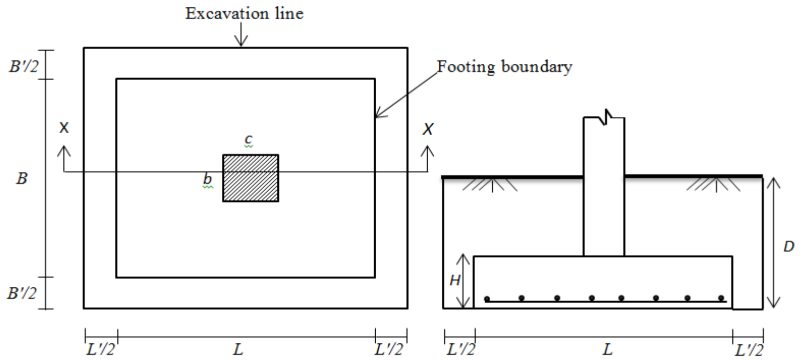
A geotechnical foundation design should address at least three basic requirements: bearing capacity, settlement limit and economics. Three primary limit states of soil supporting an isolated foundation are bearing failure, serviceability failure and total settlement. The designed foundation must be safe structurally against shears and flexure failure also. The significance of economics in geotechnical engineering designs is well recognized and is discussed in various textbooks (e.g., Lambe and Whitman, 1969; Coduto, 1994). Economic considerations usually follow the technical and carry a lesser weight. Generally speaking, multiple designs could satisfy both bearing and settlement requirements and, ideally, the final design should be the one with minimum construction cost.

Civil engineers frequently work with optimization problems, such as structural design, resource allocation, transportation routing, and so forth. Traditionally, most optimization problems have been solved using operations research (OR) techniques, such as mathematical programming that a lot of gradient information is required. In recent years, genetic-based evolution algorithms have become a popular method for solving optimization problems. In recent years, the geotechnical engineers are also using GA for the computerization of their many problems. For example, Simpson and Priest (1993) demonstrated the application of a GA for identifying the maximum discontinuity frequency in a complex rock structure for three different problem sizes. Goh (1999) incorporated a GA to search for the critical slip surface in multi-wedge stability analysis. Javadi *et al* (1999) used a GA to identify material parameters in a constitutive relationship describing the time dependency of air permeability of shotcrete tunnel lining. Deng and Lee (1989) applied a GA for displacement back analysis of a steep slope at the Three Gorges Project site.

In this paper, a new approach using genetic-based evolution algorithm without gradient requirement is framed as an optimization process, in which the optimization variables are the footing dimensions and depth, the optimization objective is to minimize the construction costs, and the bearing and settlement requirements which are treated as design constraints.

EXAMPLE PROBLEM STATEMENT

Fig. 1 illustrates a spread footing in dry sand with a unit weight (γ)= 18.5 KN/m^3 and an effective friction angle (Ø′)=35°. The sand has a Young's modulus *(E)* =50 MPa and a Poisson's ratio (υ)=0.3. The groundwater is at considerable depth, so the spread footing is not affected by groundwater. The footing is designed to support a column (b=c=0.46 m) that transfers dead load= 1400 KN and live load = 900 KN to the footing. Four key design parameters for the spread footing are the depth *(D)*, width *(B)*, and length *(L)* and thickness (*H*) as shown in Fig. 1.



(a) Plan of spread footing (b) X-X section of spread footing

Fig. 1 Spread footing design example (B′ and L′=over excavation distances along width and length direction, respectively, which are assumed 0.3 m).

GEOTECHNICAL DESIGN

A geotechnical foundation design should address at least two basic requirements: the required factor of safety, FS_r, against bearing capacity failure and the allowable settlement, δ, respectively. In this example, $FS_r \geq 3$ and $\delta_r \leq 25$ mm are used as the design criteria. In addition, the foundation depth, D, should be greater than a minimum depth, $D=0.5m$ (e.g. to prevent frost damage) and should be limited to a maximum depth, $D_u=2.0m$ (e.g. to minimize disturbance to adjacent structures).

The ultimate bearing capacity (q_{ult}) of a footing on a cohesionless soil can be calculated from Eq. (1), for general shear and a concentrically applied vertical load (Vesic, 1975)

$$q_{ult} = 0.5B\gamma' N_\gamma \zeta_{\gamma s} \zeta_{\gamma d} + q' N_q \zeta_{qs} \zeta_{qd} \quad (1)$$

Where q'=effective overburden stress at foundation level; γ'=effective unit weight of soil below the foundation level; N_q and N_γ =bearing capacity factors and $\zeta_{\gamma s}, \zeta_{\gamma d}, \zeta_{qs}, \zeta_{qd}$ =shape and depth factors. Allowable bearing pressure, q_a= [total working load (P)/footing area (BL)]

Then, the factor of safety (FS) can be calculated as

$$FS = \frac{q_{ult}}{q_a} \quad (2)$$

For this example, the settlement **δ** is calculated using the following elastic solution (Poulos and Davis, 1974):

$$\delta = \frac{P(1-\upsilon^2)}{\beta_z E \sqrt{BL}} \quad (3)$$

Where β_z is shape factor (Whitman and Richart 1967) approximated by the following second-order polynomial function:

$$\beta_z = -0.0017\left(L/B\right)^2 + 0.0597\left(L/B\right) + 0.9843$$
(4)

COST ESTIMATION

A key component in the optimization is calculating cost. Cost estimation of spread footing can be done using unit prices of each individual item. The quantities of five items are function of design parameters. The unit prices of each individual item are extracted from Schedule of rates for civil works, 14th Edition, Public Works Department (PWD) as summarizes in Table 1.

Table 1: Unit prices of each individual item

Items	Unit	Unit price (including labor cost) (TK.)	Equivalent United States dollars (US$)*
Excavation	m^3	67.00	0.893
Formwork	m^2	354.00	4.72
Reinforcement	kg	61.50	0.831
Concrete	m^3	7319.00	97.59
Compacted backfill	m^3	114.00	1.52

*Note : 1US$ = 75.00 TK.

Then the total construction cost (Z) for spread footing can be estimated as follow

$$Z = C_c Q_c + C_r Q_r + C_e Q_e + C_f Q_f + C_b Q_b \qquad (5)$$

Where Q_c=quantity of concrete (m^3), Q_r =quantity of reinforcement (kg), Q_e =quantity of excavation (m^3), Q_f =formwork (m^2), Q_b =quantity of compacted backfill (m^3), C_c, C_r, C_e, C_f, C_b = unit prices of concrete ($/$m^3$), reinforcements ($/kg), excavation ($/$m^3$), formwork ($/m^2) and backfill ($/$m^3$).

GENETIC ALGORITHM

The GA used in the present study codes and decodes four input parameters (L, B, H and D) represented by four strings (chromosomes) of a mean of five binary bits (genes): 0 and 1 to process an individual (a point in the search space). Finally, an individual contains 20 bits, yielding a task search space of 2^{20} possible combinations. Therefore, an appropriate population size (*Pop*) should be on the order of 80 (Goldberg et al, 1992). As such, population sizes of 100, 200, and 400 are used for conventional GA runs. However, the initial sets of parameters (populations) are created randomly. In every generation, the following common operations are carried out:

a) Fitness function evaluation

The fitness function (Z) is the cost of material such as concrete and reinforcement, cost of excavation and backfilling and cost of formworks as a function of design variables (B, L, H and D). The objective function can be expressed as follow:

$$\text{Minimize } Z=f(B,L,D,H,C_c,C_r,C_e,C_f,C_b)$$

Subjected to design requirements: $FS_r \geq 3$, $\delta_r \leq 25$ mm, B and $L>0$, $H \geq 150$ mm, and $0.5 \leq D \leq 2.0$m. The real fitness value for an individual is the difference between the fitness of the fit individual and the fitness value of the individual in question in a generation.

a) Reproduction

The reproduction process uses a tournament selection scheme (Goldberg and Deb, 1991) in order to eliminate the shortcomings of the fitness-proportional selection. The number of randomly selected individuals (parents) for tournament competition is referred to as the tournament size, which, in the present study, is two.

b) Crossover

The parents are then crossed at one point to produce a child (**Fig. 2**). The probability of single-point crossover is 0.5 (Carroll, 1996).

Crossing site

Parent 1 0 0 1 1 1

Parent 2 0 1 1 0 1

Child 1 0 0 1 0 1

Child 0 1 1 0 1

Child (after mutation) 0 0 1 0 1

Fig. 2 Schematic of genetic operators: Crossover

Fig. 3 Schematic of genetic operators: Jump mutation

c) **Jump and creep mutations**

After crossing, the jump mutation takes one of the bits inside the chromosome of a child as a simple mutation and determines the mutation of a selected bit based on the mutation probability (**Fig. 3**).The jump and creep mutation probabilities are 1/*Pop* and 4/*Pop*, respectively (Carroll, 1996).

d) **Elitism**

The propagation of the best, or elite, individual with lowest unfitness value is ensured in the next generation causing the GA to progress faster.

e) **Convergence check**

Steps (a) through (e) are repeated until the number of generations exceeds the maximum limit.

f) Constraint handling

In engineering optimization problems, it is vital to satisfy performance constraints. As GAs are unconstrained optimisation techniques, it is necessary to transform the constrained optimisation problem to an unconstrained one. Michalewicz, (1995) and Coello (2002) proposed several methods for handling constraints by Gas. Among them method based on penalty approach can be identified. In a penalty method, a constrained optimization problem is converted to an unconstrained problem by adding a penalty for each constraint violation to the objective function, Z′(x), as follows

$$Z'(x) = Z(x) + r\sum_{i=1}^{n} \emptyset_i(x) \tag{6}$$

Where Z'(x) is the penalized objective function, r is the penalty multiplier, n is the number of constraints and $\emptyset_i$ (x) is the i^{th} penalty function which can be expressed in a general form as follows:

$$\emptyset_i(x) = [\max(G_i(x), 0)]^m \tag{7}$$

Where m is the power of penalty function and G_i(x) is the value of the i^{th} constraint.

OPTIMUM DESIGN

The optimization model is coded in FORTRAN. The performance of the GA is given in Fig. 4. It is observed that after only 20 generations, the algorithm has found the optimum design. The optimized design was found with total cost of 228.19 US$ which occurs when L=1.716 , B=1.553, H=0.574, and D=1.780 m. Table 2 given below summarizes the optimized design and comparison with other two designs which also satisfies the all design requirements.

Table 2: Comparison of three designs

Design option	Width, B (m)	Length, L (m)	Thickness, H (m)	Depth, D (m)	Cost (US$)	Difference (%)
Optimum	1.553	1.716	0.574	1.780	228.19	--
Example 1	2.040	2.040	0.639	1.308	384.13	68.34
Example 2	2.040	1.878	0.639	1.308	354.46	55.34

PARAMETRIC STUDY

Design requirements are paramount, and their variation can result in different designs and consequently different construction costs. Because an economically optimized design incorporates construction cost estimates, it is possible to explore the effect of design requirements on the construction costs.

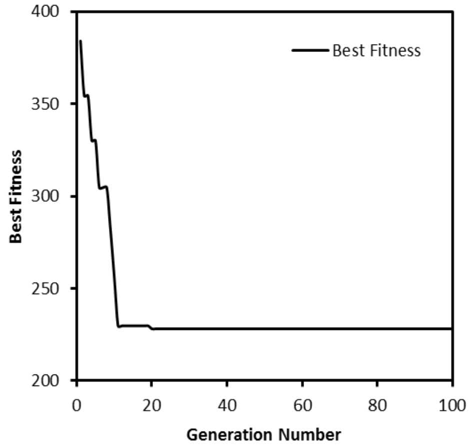


Fig. 4 Variation of best fitness with generation

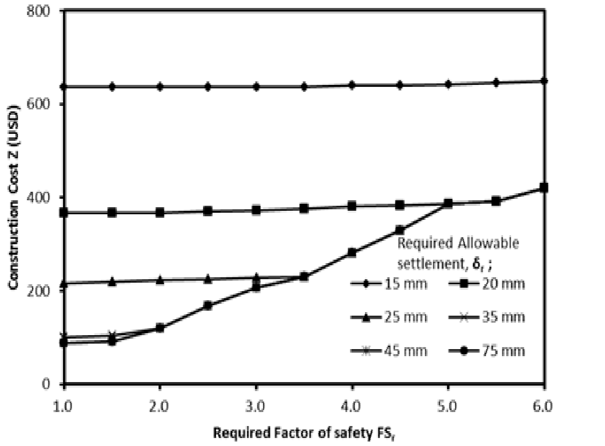


Fig. 5 Effects of design requirements FS_r and δ_r

Fig. 5 shows the variation of construction cost as functions of required FS_r and δ_r. For a given δ_r, the construction cost increases as FS_r increases. But, when δ_r is relatively small, the effect of δ_r is very significant on construction cost and the effect of the variation of FS_r on final design diminishes. For δ_r=25mm, there is no change in construction cost of the optimum design below the FS_r=3.5, but if the required factor of safety is greater than 3.5, the foundation cost increases.

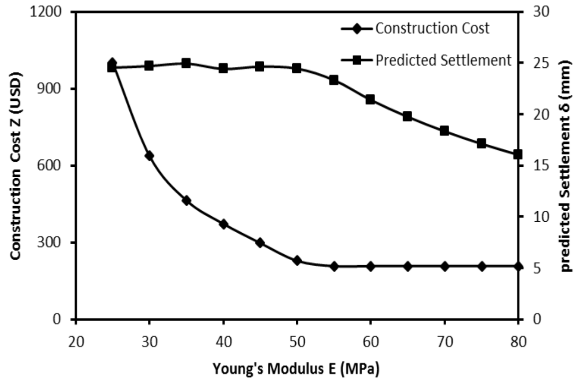


Fig. 6 Effect of Young's modulus

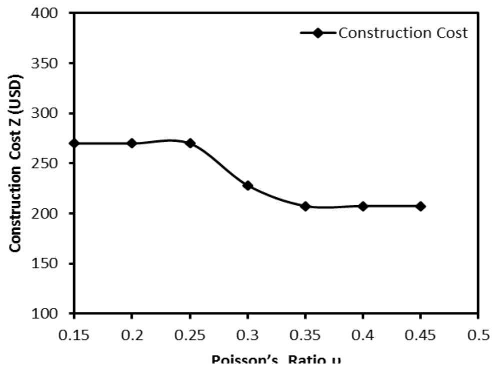


Fig. 7 Effect of Poisson's Ratio

Fig. 6 shows the variation of construction cost and predicted settlement of the optimum design with Young's modulus of the foundation material. The predicted settlement decreases with increasing the elastic modulus. Specially, the cost decreases significantly with the increase of Young's modulus up to the value of 60MPa: when it increases by 50%, the construction cost decreases by more than 300%. Thus, additional efforts might be warranted for improving the characterization of elastic modulus in sub-soil investigation. Fig. 7 shows the variation of construction cost as a function of Poisson's ratio **υ.** It shows that construction cost decreases maximum 30% with the increase of Poisson's ratio upto 0.45.

CONCLUSIONS

The design procedure of foundations for given loading, soil properties and strengths of structural material is framed in an optimization process using genetic algorithm, in which the optimization variables are the footing dimensions and depth and the objective function is the total construction cost, treating the design requirements (e.g. FS against bearing failure and maximum settlement) as design constraints.

The summarized findings of this paper are:

- In spread footing design, a relatively large FS_r generally fulfil δ_r.
- Additional efforts might be warranted for improving the characterization of elastic modulus in sub-soil investigation.
- Total construction cost decreases to a small amount (below 30%) with the increase of Poisson's ratio **υ.**

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SHAPE EFFECT ON THE BEARING CAPACITY OF FOOTING RESTING ON CLAY

M. Rokonuzzaman[1], M. K. Ahsan[1*], M. N. Haque[1] and T. Sakai[1]

[1] *Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna, Bangladesh.*
**Corresponding Author, < kamrul0701047@gmail.com >*

ABSTRACT

Traditional bearing capacity theory is based on two dimensional assumptions. To find out the bearing capacity for rectangular foundation empirical shape factor is applied. However shape effect of foundation resting on clay is still required to be investigated. In this paper, the behavior of square, circular and rectangular footings resting on undrained clay has been investigated by experiment and numerical analysis under vertical loading. PLAXIS3D software is used to create models, generate mesh and analyze the load displacement behavior. The FE model is validated against findings from experimental model test. Terzaghi used shape factor 1.3 for both circular and square footing for cohesion term. But in this present study, the shape factor for square footing was found 1.36 and circular footing it was found 1.26. For rectangular footing, the load bearing capacity decreases with the increase of length/width ratio of the footing. The shape effect is analyzed and compared with existing empirical relationships and that obtained by FE analysis.

Keywords: Bearing Capacity, Undrained soil, Shape Factor, Finite Element Analysis

INTRODUCTION

Ultimate bearing capacity for footing under vertical loading is conventionally achieved by applying modification factors to the classical plane-strain solution for vertical bearing capacity (Terzaghi, 1943). For undrained condition the ultimate vertical bearing capacity of a surface footing can be expressed by equation (1),

$$q_{ult} = N_c s_u s_c \tag{1}$$

where, N_c is the plane-strain bearing capacity factor ($2 + \pi$), s_u is the undrained shear strength of the soil, and s_c are modification factors to account for footing shape. For a rectangular footing of effective breadth, *B*, and length, *L*, a widely adopted shape factor is that proposed by Skempton (1951),

$$S_c = 1 + 0.2\, B/L \tag{2}$$

Meyerhof (1963): $s_c = 1 + 0.2 K_p\, B/L$ (3)

Hansen (1970): $s_c = 1 + \dfrac{N_q}{N_c} . \dfrac{B'}{L'}$ $(\phi \neq 0^0)$, $s_c = 0.2 \dfrac{B'}{L'}$ $(\phi = 0^0)$, (4)

Vesic (1793): $s_c = 1 + \dfrac{N_q}{N_c} . \dfrac{B}{L}$ (5)

$s_c = 1.0$, for strip footing (applicable for both Hansen and Vesic)

Terzagi's (1943) proposed shape factors are, s_c =1.0 (strip), 1.3 (round), 1.3 (square).

Previous studies of undrained bearing capacity have addressed plane-strain conditions and circular footings (e.g. Ukritchon et al., 1998; Taiebat & Carter, 2003; and Gourvenec & Randolph, 2003). These previous studies have shown that the mechanisms governing failure, differ for plane-strain and axisymmetric geometry. Undrained failure of rectangular footings, has not previously been considered. Mechanisms of failure under axial loading of rectangular footings would be expected to be similar for varying aspect ratios, and similar to failure in plane strain (more so than a circular footing). Exact solutions for the uniaxial vertical bearing capacity of square or rectangular footings have yet to be found. As indicated in equation (2), a semi-empirical shape factor is conventionally applied to the vertical bearing capacity solution for plane-strain conditions to account for aspect ratio.

The most commonly adopted expression, proposed by Skempton (1951), suggests a linear relationship between the ultimate vertical load and aspect ratio, with a maximum increase of 20% for a square footing compared with the plane-strain case. A detailed review of the available literature of analytical and numerical studies addressing bearing capacity factors for square and rectangular footings is reported by Gourvenec et al. (2006). Based on results from finite element analyses, Gourvenec et al. (2006) propose shape factor for rough rectangular footings, which can be described by a quadratic function,

$$s_c = 1 + 0.214\frac{B}{L} - 0.067\left(\frac{B}{L}\right)^2 \tag{6}$$

In this study, a series of computations were carried out to determine the influence of various footing shape on the bearing capacity under vertical loading on homogeneous un-drained clay soil. The FE model is validated against experimental model test results. The shape effect is analyzed, compared with existing empirical relationships, and a new proposal is presented.

FINITE ELEMENT MODELING

The finite element analyses were performed using the software PLAXIS 3D 1.1 professional version (2004). The analyses used unstructured meshes consisting of 15-noded wedge elements. Unstructured meshing allows efficient element arrangement and refinement of the elements in the vicinity of the corners of the footing, which is crucial for the accurate prediction of the collapse load (Potts and Zdravkovic, 1999). The 15-noded elements converge more rapidly and perform better numerically than 6-noded elements. The final models were created using very fine mesh for greater accuracy (Figure 2). Loading of the footing is accomplished through prescription of uniform incremental vertical displacements at the nodes located at the base of the footing (displacement control), while the horizontal degrees of freedom are fixed. This way, the footing is considered to be perfectly rigid and rough. At the bottom boundary of the finite element mesh, both the horizontal and vertical degrees of freedom are fixed. At the lateral boundaries, only the horizontal degree of freedom is fixed. The notations L, B, W and H are shown in Figure 1. There are no interface elements placed between the soil and the footing, so any slippage between footing and soil occurs within the soil. This is realistic because concrete footings poured against the ground form a very rough interface. For all the analysis, the minimum distance of edge of soil element was 4.5 times of width (B) or diameter (D) of footing and the distance of the bottom of the soil element from footing was 5 times of width (B) or diameter (D) of footing based on boundary effect analysis.

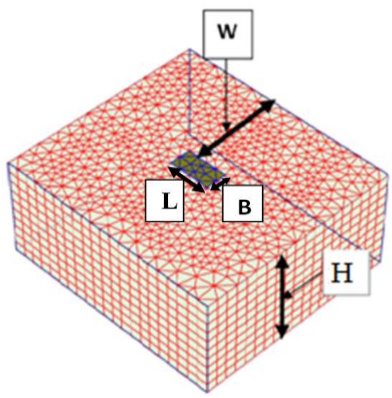


Fig. 1 Footing dimension

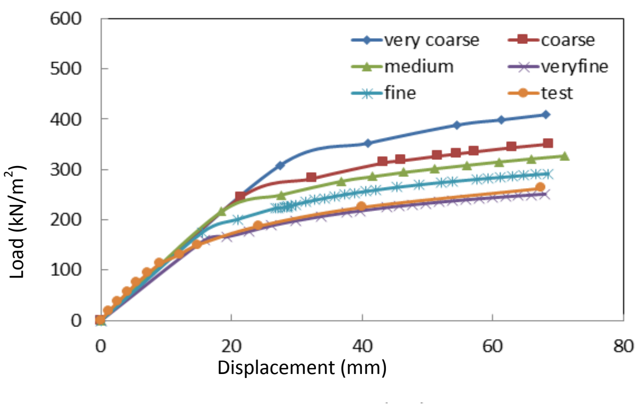


Fig. 2 Effect of mesh size

RESULTS AND DISCUSSION

FE Model Validation

For the validation purpose, two model footing tests data are used in this study. The model tests were conducted using steel plate of square (length= 0.4 m) and circular (diameter=0.42 m) on the surface in JAPAN. Load cell and displacement transducer were used for data acquisition. For characterization of the foundation material, undisturbed soil samples were collected from the field and undrained shear strength parameters were obtained from CU test. In the FE model, the soil is modelled as elastic–perfectly plastic material with the Mohr–Coulomb failure criterion.

The properties of soil element are from laboratory test results:

Unit weight of soil, γ= 16 kN/m^3

Undrained shear strength, S_u=37.5 kN/m^2

Modulus of elasticity, E = 3000 kN/m^2

Poisson's ratio = 0.3

From load displacement curves in Figure 3 it is seen that finite element models closely predicts the experimental results. It is also clear that square footing has about 8.6% greater load bearing capacity than that of circular footing.

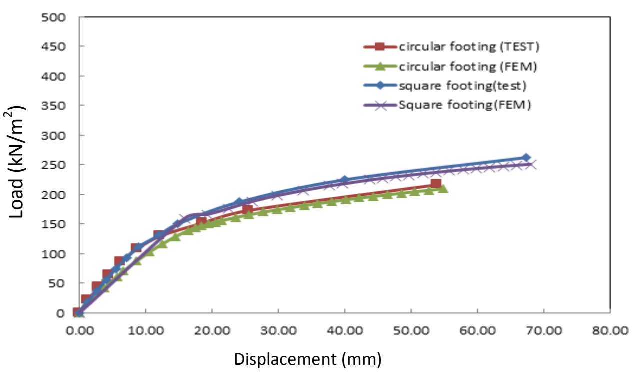


Fig. 3 Load-displacement relationship.

Fig. 4 shows failure mechanism of the square and circular footing in terms of deformed ground around the footings and numerical shear strain distribution. There is a difference of failure mechanism between circular and square footings. For circular footing, the failure surface development is all around uniform but for Square footing it is non uniform: at corners the development is faster than that at the sides.

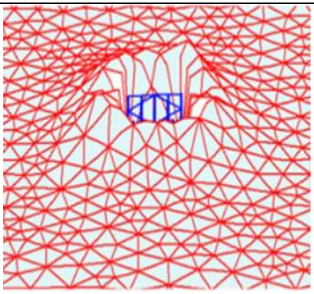
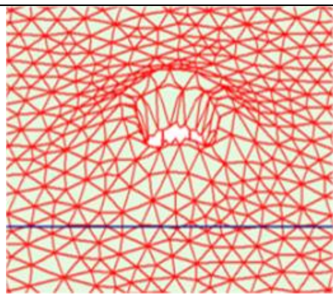
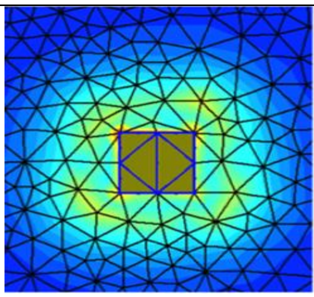
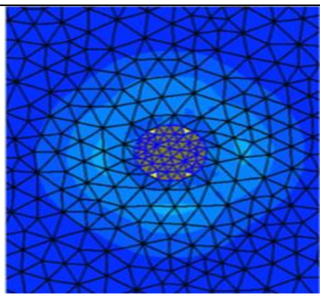
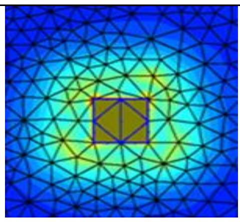
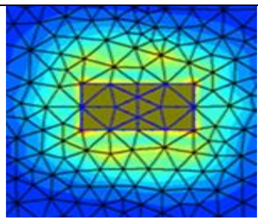
Square footing	Circular footing
	
(a) Deformed mesh	
	
(b) Shear strain distribution	

Fig. 4 Failure mechanism

PARAMETRIC STUDY

Effect of L/B

Several models were analyzed with various L/B ratio using the validated FE model only varying the geometry of the footing. The comparative results of total shear strain of soil around footing at failure are shown in Figure 5.

L/B=1	L/B=2
	
L/B=4	L/B=6

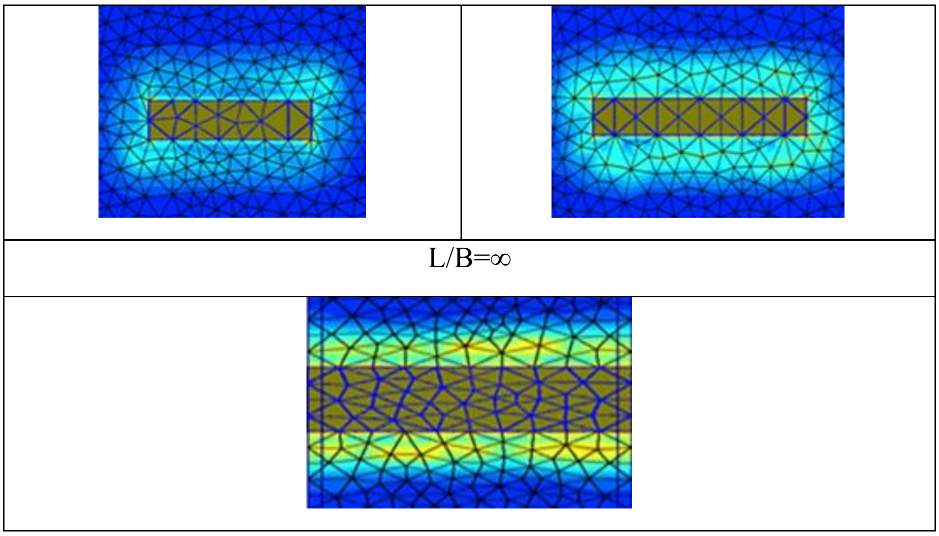


Fig. 5 Effect of L/B

From the Figure 6 it is seen that the square footing has greater load bearing capacity than circular footing for given undrained clay soil and it is about 8.6%. As the L/B ratio of rectangular increases the load bearing capacity of the footing decreases.

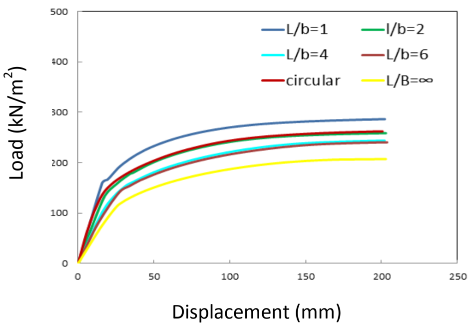


Fig. 6 Effect of Shape

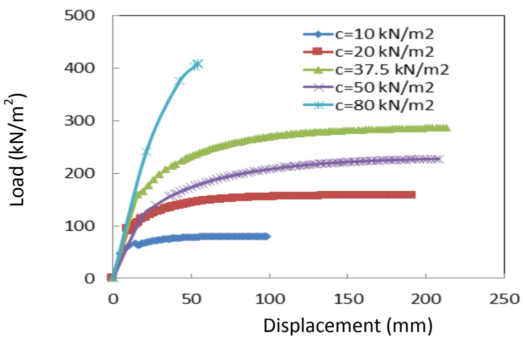


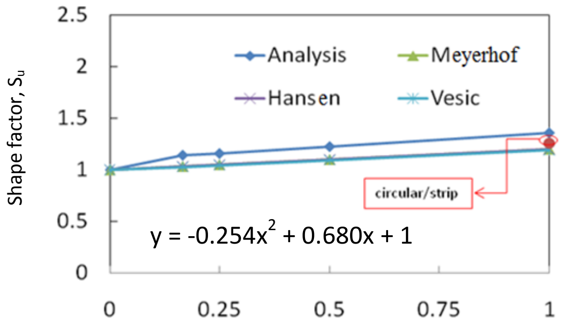
Fig. 7 Effect of cohesion

Effect of cohesion

From Figure 7 it is seen that the bearing capacity of square footing increases with the increase of cohesion of soil.

PROPOSED SHAPE FACTOR

At first the shape factors was calculated using Terzaghi's bearing capacity equation, for footing of different shapes. From the plot of shape factor (S_u) vs. B/L (Figure 8) an equation for shape factors was developed. The shape factors was also calculated from Meyerhof (1963), Hansen (1970) and Vesic (1973) equations.



B/L

Fig. 8 Comparison between Obtained Shape Factors and Theoretical Shape Factors

From Figure 8 it is seen that the shape factors from various equations are close to obtained shape factors from FE analysis. From the plot of shape factor (S_u) vs. B/L, an empirical quadratic equation for shape factors was developed-

$$S_u = -0.254\left(\frac{B}{L}\right)^2 + 0.680\left(\frac{B}{L}\right) + 1$$

Terzaghi assumed that the shape factor for both circular and rectangular footing =1.3, but in our project for given undrained clay soil it is found slight different for circular and square footing as shown in Table 1.

Table 1: Terzaghi's shape factor

Shape factors	Present Study	Terzaghi
Square/Strip	1.36	1.3
Circular/Strip	1.26	1.3
Square/Circular	1.08	1

CONCLUSION

The aim of the work presented in this paper was to assess shape effects on the capacity of footings under general loading. In the present work load –displacement behavior of the different types of footing resting on un-drained clay soil was investigated and equation for shape factor was also developed. From which shape factor for any rectangular footing resting on undrained clay soil can be obtained.

The summarized findings of this paper are:

- FE model created in PLAXIS 3D gives almost exact result as experimental test result.
- Square footing has around 8.6% greater load bearing capacity than circular footing for given un-drained clay soil.
- As the L/B ratio increases the load bearing capacity decreases.
- As the value of cohesion c increases the bearing capacity of the footing also increases for this particular soil.
- The shape factors obtained from this work for undrained clay soil is close to shape factors from Meyerhof (1963), Hansen (1970) and Vesic (1973) equations.
- The finite element results from this study suggest the relationship between shape factor and footing aspect ratio for rectangular footings can be accurately predicted by a quadratic polynomial , $S_u = -0.254(B/L)^2 + 0.680(B/L) + 1$

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ID: SEE 001

A STUDY OF RCC FRAME BUILDINGS WITH VERTICAL DISCONTINUITY

M. J. Islam[1] & N. B. Masud[2]*

[1] *Islamic University of Technology, Gazipur - 1704, Bangladesh.*
[2] *Stamford University Bangladesh, Dhaka - 1217, Bangladesh.*
**Corresponding Author, <snafis05@gmail.com>*

ABSTRACT

With growing population and limited land area it becomes essential to optimize the use of available land area, especially for the urban areas. To achieve sufficient parking facilities the designer may need to displace only the ground floor column in such a way that the structure becomes irregular. Without proper attention effect of this irregularity in reinforced cement concrete (RCC) frame buildings can be devastating. It can cause sudden failure of structure because of the discontinuous loading path from the super structure to the foundation. This work presents an investigation on vertical irregularity at the ground floor of high rise RCC frame building by displacing the column. Five widely varying cases have been considered and analysed by using a finite element software. Size of transfer girders, change in stiffness for various floors and factor of safety against overturning are compared for all five cases. Based on the results best structurally safe and economical solution has been recommended. The result shows that if proper analysis is conducted vertical discontinuity will not cause structural instability rather it will provide economic solution and stability for the structure.

Keywords: Irregularity, column, stiffness, stability

INTRODUCTION

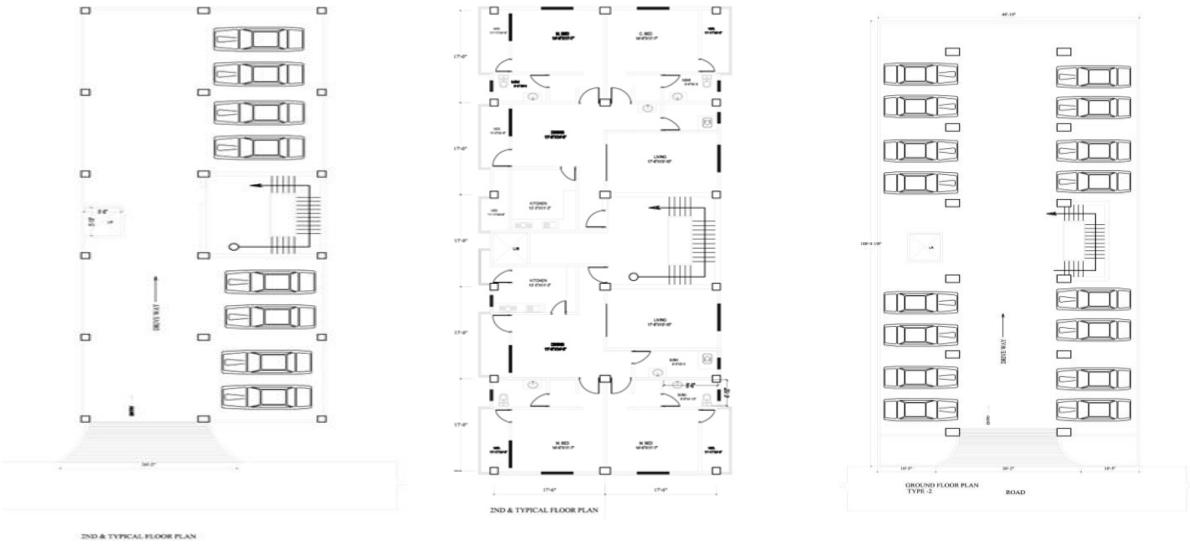
In the recent times, the demand of apartments, office spaces and other facilities are increasing at an alarming rate, although the available land is not increasing in that ratio. Now a day, soft story buildings are much more preferred than irregular buildings. But if the space of the building is not sufficient, it is very difficult to design it as a soft story structure. For this reason, a vertical discontinuity in the plan occurs to provide sufficient parking facilities. But the design of this kind of irregular structure needs lots of precautions like resisting the lateral loads, making the structure stiff, reducing the undesirable torsional effects. An essential characteristic of any lateral load resisting system is that it must provide a continuous load path to the foundation. Inertial loads that develop due to acceleration of individual elements must be transferred from the individual reactive elements to floor diaphragms to vertical elements in lateral load system.

Many buildings with an open ground story intended for parking collapsed or were severely damaged in Gujarat during the 2001 Bhuj earthquake. Buildings with columns that hang or float on beams at an intermediate story and do not go all the way to the foundation have discontinuities in the load transfer path. During the January 26, 2001 earthquake, numerous mid- to high-rise residential buildings collapsed in the city of Ahmedabad leading to several hundred causalities and significant financial loss. The typical residential construction in Ahmedabad consists of reinforced concrete moment resisting frame system. The frame at the ground floor is open while frames at the upper floors are filled with un-reinforced brick panels. The columns at the ground floor may not align with the columns at the upper floors giving rise to vertical discontinuities in the lateral load resisting system. The above-described lateral load resisting system occurs because of two factors. First, the open ground floor is needed to provide car parking; the buildings are usually built on very small land lots with little room for open parking .The analysis was not properly done.

Chintanapakdee and Chopra (2004) studied the effects of stiffness and strength irregularities on story drift demand and floor displacement responses. They studied the influence of vertical irregularities in the stiffness and strength distributions, separately and in combination, on the seismic demands of strong-column-weak-beam frames .The main purpose of this study is to study the effect of irregularities in the building structures and to provide an economical and safe solution for the additional parking facilities for residential or commercial building structures.

METHODS

Five alternative cases has analyzed where the height of structure is 90 feet (G+8), 35'x85' plan view but columns in the ground floor are to be displaced in order to increase parking facility. Both analytical analysis and numerical analysis are done. For numerical analysis, "ETABS" software is used, where the effects of earthquake load and wind load are given automatically by according to the UBC-94 which is similar to BNBC code. In analytical analysis girder design, stiffness calculation and factor of safety against overturning have been analyzed.



(a) Regular structure (b) Floor plan (c) Irregular structure

Figure 1

Table 1 Variations of the cases at ground floor

Description	Case 1	Case 2	Case 3	Case 4	Case 5
Connection with ground floor to upper floors	Used girder at ground floor. No contact with upper floor	Used girder at ground floor. No contact with upper floor	Used girder at ground floor. No contact with upper floor	Used girder at ground floor. Extending corner column connects the upper floors	Used girder at ground floor. Extending corner column connects the upper floors

				with GF	with GF
Column spacing at GF	17.5’	26’	35’	17.5’	26’

NUMERICAL ANALYSIS

For the numerical analysis, “ETABS” is used where strength of concrete f’c=3 ksi and strength of reinforcement is fy = 60 ksi . As the structure is very stiff in Y-direction, it does not show any significant displacement along the Y-axis. The displacements due to earthquake load and wind load in each case are demonstrated in tables shown below.

Variations of displacements

Table 2 Displacement in X-direction (mm) due to earthquake load

Cases	1st story	2nd story	3rd story	4th story	5th story	6th story	7th story	8th story	9th story
Case 1	3.71	7.4	11.23	14.92	18.32	21.36	23.99	26.15	27.83
Case 2	3.66	8.27	13.11	17.74	21.99	25.75	28.93	31.46	33.31
Case 3	2.83	5.98	9.29	12.45	15.33	17.85	19.95	21.58	22.73
Case 4	3.47	6.69	10.06	13.23	16.21	18.79	20.97	22.69	23.93
Case 5	3.89	7.26	10.77	14.11	17.15	19.83	22.07	23.84	25.10

Table 3 Displacement in X-direction (mm) due to wind load

	1st story	2nd story	3rd story	4th story	5th story	6th story	7th story	8th story	9th story
Case 1	9.88	19.62	29.36	38.25	46.04	52.61	57.91	61.95	64.92
Case 2	6.94	15.66	24.42	32.33	39.14	44.72	49.04	52.09	54.07
Case 3	7.85	16.27	24.70	32.29	38.77	44.03	48.03	50.76	52.42
Case 4	9.80	18.54	27.25	35.11	41.88	47.42	51.69	54.71	56.64
Case 5	10.77	19.78	28.13	36.79	43.70	49.36	53.71	56.78	58.76

ANALYTICAL ANALYSIS

In analytical analysis girder design, stiffness calculation and factor of safety against overturning are included.

Girder design

To design the girder doubly reinforced girder design is used where materials properties f'c=3ksi and fy=60ksi. From numerical analysis the girder size can be found but to make the structure more economical it is also included in analytical analysis.

Stiffness calculation

To calculate the story stiffness, Muto's expressions (Muto, 1974) has been used.

For first story:

$$K_C = \left(\frac{12E_C I_C}{H^3}\right)\left(\frac{K_{GA}+K_{GB}}{4K_C+K_{GA}+K_{GB}}\right) \quad (1)$$

For second story:

$$K_C = \left(\frac{12E_C I_C}{H^3}\right)\left(\frac{K_C+K_A}{4K_C+K_{GA}}\right) \quad (2)$$

Where, E_C = Elasticity of concrete, I_C = moment of inertia of column, H = height, K_C = column stiffness, K_G = girder stiffness.

Factor of safety against overturning

Factor of safety against overturning = (resisting moment/ overturning moment) **.** The moment that is created by an applied force which causes a structure to turn over is called overturning moment. A moment produced by internal tensile and compressive forces that balance the external bending moment on a beam.

Table 4 factors used to calculate FS against overturning

factors	*description*
Wind velocity	120mph
Average pressure	0.0030 x (wind velocity)2
Surface area	Area of the plan (35'x85')
V wind	Average pressure x Surface area
Overturning moment	V wind x (Height of the building/2)
Minimum weight of building	0.2 kip/sq. ft./floor
Resisting moment	Weight of building x center of gravity of building

RESULT AND DISCUSSION

Table 4 Variations of different criteria for all the cases

VARIATIONS	*Case 1*	*Case2*	*Case 3*	*Case 4*	*Case 5*
Girder size (in x in)	25 x 45	25 x 36	25 x 34	20 x 30	20x 34
Moment(k-ft)					
+ve	805	1314	2217	1216	1705
-ve	3948	2216	1836	1044	1685
Shear (kips)	524	473	260	235	270
Stiffness					
Ground floor	2.86	2.20	2.04	1.50	1.70
First floor	3.98	3.27	3.13	1.64	1.86
FS against over turning	2.84	4.22	5.68	5.68	5.68

Form the results shown in Table 4 it can be seen that case 4 required lower girder size, moment variation and shear is low. Factor of safety against overturning is satisfied. Story stiffness is lowest and displacement is lower than other all cases. In case 4 girder is used at ground floor and extension of corner column connects the super structure with the foundation. In this case at ground floor four column have used, the two corner column and two displaced column. The spacing between two displaced column that make the structure irregular is 17.5'.

CONCLUSION

The displacement of ground floor column in Case 4 produced an irregular structure leads to following conclusion

- If the structure is soft story eight car parking can be provided but displacing the ground floor column sixteen parking can be allocated
- For building a structure balancing of positive and negative moments of any structural elements is must otherwise crack will develop. the balancing of positive and negative moments is done in the analysis
- The discontinuous loading path from the super structure to the foundation can cause sudden failure of structure but in the extending the corner column has created contact with the super structure to the foundation.
- Irregularity can cause to change the centre of gravity of the structure and in that case it is very essential for any structure to be safe against overturning. From the analysis it can be been that though the structure is irregular but it is safe against overturning
- Stiffness is an important factor in case of irregularity. In the analysis though the ground floor column is displaced but the variations of stiffness in ground floor to first floor is not significantly changed.

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ID: SEE 002

LATERAL STRENGTHENING SYSTEMS OF MULTIPLE SPAN FLAT PLATE BUILDING AND DYNAMIC ANLYSIS

R.K Biswas[1]*, M. Muazzam[2], R. Jany[2] & M. Rahman[2]

[1] *Dept. of Civil & Environmental Engineering, Uttara University, Dhaka 1230, Bangladesh.*
[2] *Dept. of Civil Engineering, AUST, Dhaka, 1208, Bangladesh.*
**Corresponding Author, <rajibbiswas_aust@yahoo.com>*

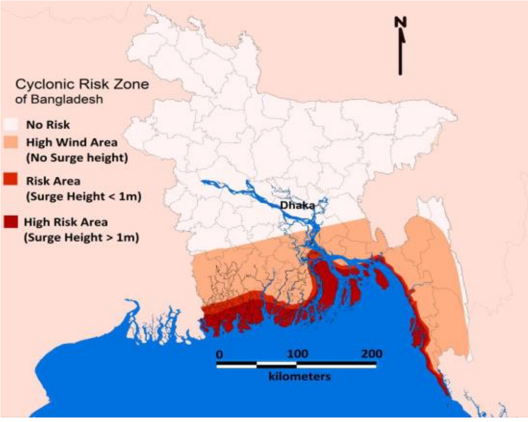
ABSTRACT

This research was carried out to find overall lateral strength of building with multiple span in one direction. In current state, flat plate buildings with multiple span are normally designed and constructed although it is been severely warned against earthquake. Even in Dhaka, capital of one of the most earthquake-warned-country (Bangladesh) construction work is not up-to-the-mark in all cases. Those ordinary structures are designed without proper seismic consideration could be vulnerable to damage even under low level of ground shaking from distant location. Recent event of rana plaza can be held as example; where people got smashed under slabs. Not only in this case but also in every earthquake incident building had got sandwiched. It is called “Pie Effect”. As a matter of fact, the sole concern of this research to find most reliable lateral strengthening systems for flat plate buildings without the concern of material used in constructions. This research paper investigated the lateral stiffness increment of five types of cases based on whether they are retrofitted or not under dynamic and static loading. Among the cases four cases are consisted of dual system i.e. that retrofitted by shear wall and diagonal bracing only at exterior or exterior and centre frame. Valid results are obtained by Finite Element Analysis in STAAD Pro V8i and SAP-2000 platform. The results that are shown in this research work are comparative with different cases. Lateral stiffness increased as this chronology: Shear wall at Exterior & Centre Frame> Shear wall at Exterior Frame>Diagonal Bracing at Exterior & Centre Frame>Diagonal Bracing at Exterior Frame>General Flat plate Structure. In lateral displacement and axial force reduction shear wall at Exterior & Centre Frame shows extraordinary effectiveness. But on the account of serviceability shear wall at Exterior Frames is recommended. Nonetheless, in flat plate structure most heart aching scene is when people got hurt due to “Pie Effect” of structure at the time of Earthquake. So, using shear walls that may be reduced.

Keywords: Flat plate, Response Spectrum Analysis, Equivalent Static Analysis, Time-history Analysis, Lateral Stiffness.

INTRODUCTION

Most of the buildings are vulnerable to lateral forces and flat plate comprising greater gravity load is susceptible to seismic although may not be affected by wind. Practically wind has greater effect on building structures of coastal areas. This work will provide the data that are helpful to the areas where wind speed is high. Different areas have different magnitudes of wind speeds in Bangladesh. Moreover the data of this work may come in handy at the time of wind-based disaster like cyclone, typhoon, tornado etc. It is found from several data and figure 1 that Dhaka is at no risk area unlike the coastal areas. So concern of wind load on structures of Dhaka can be temporarily unconsidered but cannot be ruled out. Also Bangladesh is ranked 6th globally in terms of vulnerability and human exposure to cyclone. As per magazine Forbes’ Bangladesh is the most vulnerable country to any kind of natural disaster. Bangladesh is ranked 17th out of 153 countries as earthquake prone country. Due to topographical conditions and geographical position there are multiple fault lines run between Bangladesh. As along fault lines generally movements of tectonic plates occur and as a matter of fact creates ground shaking so Bangladesh is a country of massive earthquake active zone. The nature and the distribution of the earthquake events in different seismic zones of the country are intrinsically related to various tectonic elements. The increased frequency of earthquakes events in Bangladesh in the last 30 years suggests reviving tectonic activity. The overall tectonics of the Bangladesh and adjoining region is conductive for the frequent and recurring earthquakes. The intensity of earthquake of recent tears and the earthquake zoning will show us why seismic effect is a matter of concern



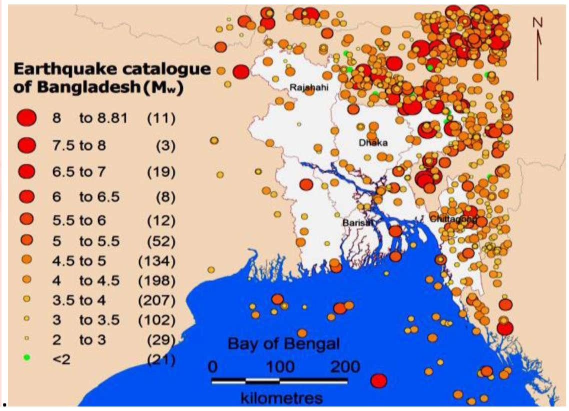


Fig.1: Wind Hazard Zone of Bangladesh. Fig.2: Magnitudes of Earthquakes nearly effecting Bangladesh

The earthquakes in the map of Bangladesh shown in figure 2 can really put our minds into thoughts and it does. So it is without question that this type of earthquake and wind load i.e. lateral forces will damage tall buildings in a small developing country with dense population, especially the flat plate one. More popular word for flat plate building structure to express its damage criteria is “Sandwich”. Mitigate approaches to reduce aftermath of wind and seismic effect are as follows:-

- Specific engineering and non-engineering needs and construction guideline;
- Retrofitting of existing buildings;
- Enactment of legislation as per BNBC codes;
- Improving the safety of essential services;
- Public information and aware

OBJECTIVE

Before go through a experimental work objectives are need to be set. The targets that are to be achieved in this work are as follows:-

- To propose an effective retrofitting system for multiple bay flat plate buildings.
- To carryout modeling and detail analysis of the building with the retrofitting system
- To study and compare the deflection pattern, axial force, bending moments of the structure.
- To achieve lateral stiffness of that structure for most vulnerable conditions.
- To attain a preliminary design of the 15-storied building.
- To analyze dynamic effect of loads on building.

SCOPE OF THE WORK

As previously objectives of the work are pointed out which necessarily create some scope for the works’ present and future. Scope of the work can be depicted as follows:-

- This work will give general view about the effect of static load and dynamic load.

- Analyzed data of axial load, shear force and bending moments will be helpful before building reaches to its failure conditions.
- Found data on displacement both for static and dynamic analysis are to be compared.
- Acceleration of the loads will provide knowledge of time duration of movement of moment of inertia and response of the structures against it.
- It will provide story drift data for static and dynamic analysis which will be useful in maintenance work especially in big industrial work.
- Finding of good retrofitting system will provide help for construction of economical and seismic-resistant building structure.
- Lateral Stiffness is determined for structure and based on this retrofitting method is chosen.
- Lateral Stiffness is compared for further use.
- Finite element analysis in STAAD Pro gives knowledge about structures.
- This may provide scope for development of software in future 3-D analysis.

MATERIAL AND GEOMETRY OF THE BUILDING

Static and dynamic analysis is done here with a 15th floor multi-span flat plate building with and without retrofitting system. Each floor is provided with a 9.5 inch slab 120 ft in length and 40 ft in width whereas column sections are different in different floor as satisfactory with loads. Every floor is divided in 6 spans and 2 spans along long and short direction respectively. Column sections are: GF-3rd floor = 75 cmX75 cm; 4th-5th floor = 67.5 cmX67.5 cm; 6th-8th floor = 60 cmX60 cm; 9th-14th floor = 50 cm X 50 cm. 20 cm shear wall and 30 cm X30 cm diagonal bracings are used retrofitting systems. All the structures are RCC (Reinforced Cement Concrete) where cement provides 4000 psi and steel provides 60,000 psi strength. Bar dia. is #8 with 4-2-2-4 layers for each column.

CASE DEFINITIONS OF RETROFITTING METHODS

To give this work proper direction it is useful to consider different cases. The cases for this literature are as follows

- ❖ Case 01: General Case: Flat Plate Building without any Retrofitting Systems;
- ❖ Case 02: General Case Retrofitted with Shear Walls: Flat Plate Buildings with Shear Walls at Most Exterior Panels;
- ❖ Case 03: General Case Retrofitted with Shear Walls: Flat Plate Buildings with Shear Walls at Most Exterior Panels & Interior Centre Panel;
- ❖ Case 04: General Case Retrofitted with Diagonal Bracings: Flat Plate Buildings with Diagonal Bracings at Most Exterior Panels;
- ❖ Case 05: General Case Retrofitted with Diagonal Bracings: Flat Plate Buildings with Diagonal Bracings at Most Exterior Panels & Interior Centre Panel;

METHODOLOGY

After determining slab thickness and column section and selecting retrofitting methods now the consideration about the loads comes. Gravity load as detected to determine slab and column section will be used. Lateral loads are needed to be calculated. Lateral loads means as a whole of wind and earthquake loads. The calculation of wind load and earthquake load is main hold up for development of this work. Without these loads the analysis holds no important contribution. After calculating loads In purpose of elaborating analysis and comparison of the work load combination as per BNBC is selected as follows:
Load Combination 1: Basic Load Combination: 1.4DL +1.7LL.

Load Combination 2: 1.05DL + 1.275LL + 1.275W.

Load Combination 3: 1.05DL + 1.275LL + 1.4E.

Load Combination 4: 1.4(DL + LL + E).

Where,

DL= Dead Load for Self weight, Partition Wall and Floor Finish.

LL= Live Load.

W = Wind Load.

E = Earthquake Load.

So, the work design gets summarized in following five phases:-

→ *Phase 1*: Determination of Slab Thickness only considering Gravity Loads.
→ *Phase 2*: Determination of Column Sections only considering Gravity Loads.
→ *Phase 3*: Calculation of Wind and Seismic Load.
→ *Phase 4*: Selecting Shear Wall and Diagonal Bracing as Retrofitting Methods.
→ *Phase 5*: Modeling and Analysis.

RESULTS AND DISCUSSION

With the help of software and using above mentioned load combination and retrofitting systems comparative data of axial force, displacement and story drift is collected to achieve fruitful conclusion. Result of this work is sub-divided in static and dynamic analysis. Only deflection part is analyzed for both static and dynamic case. Here, all comparisons are shown for exterior corner columns and Load Combination (LC) 2.

Axial force Analysis

To prevent building from "Pie Failure" it is important to reduce compression (+ axial force) in column as column is susceptible to compression. So, here reduction of compression is emphasized while comparing different cases. Now, % reductions for 14th floor corresponding to Case 01 for LC 2 are:
Case 02 = 77.53%, Case 03 = 77.29%, Case 04 = 17.06%, Case 05 = 17.32%

Moment Analysis

For further inurement, moment comparison is also done for same case and criteria. But data is only shown for floor 8th to 14th floor and moment is graphed against height above ground

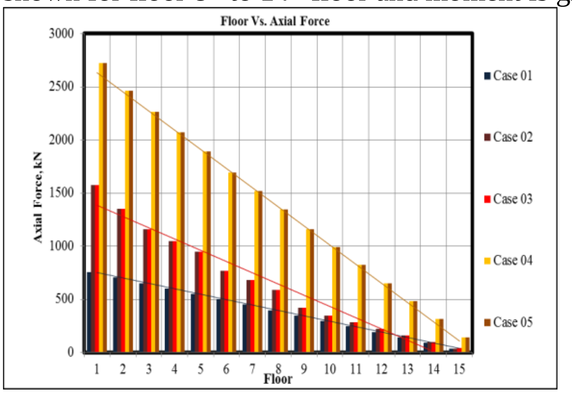


Fig. 3: Comparison of Axial Forces for Different Cases.

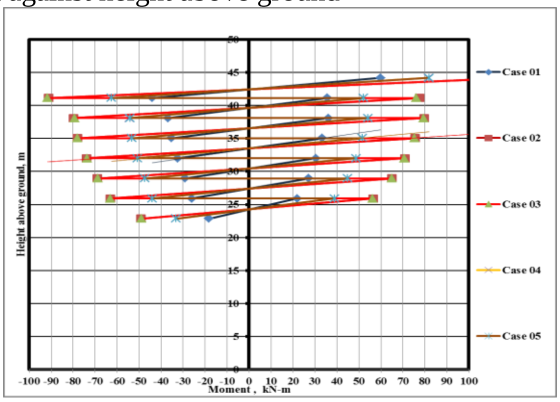


Fig. 4: Comparison of Moements for Different Cases.

Displacement Analysis

Most important factor that comes in this work as demarcation is displacement. For 14th floor along weak direction for center column %reduction in displacement corresponding to case 01:
Case 02 = 64.55%; Case 03 = 74.24%; Case 04 = 49.49%; Case 05 = 53.77%

Storey Drift Analysis

Storey drift is the difference between two storey displacement divided by the storey height. Storey Drift provides the value that how much a building can accelerate from its original position. For 14th floor along weak direction %reduction in storey drift corresponding to case 01:
Case 02 = 12,53%; Case 03 = 34.94%; Case 04 = 8.92%; Case 05 = 10.36%

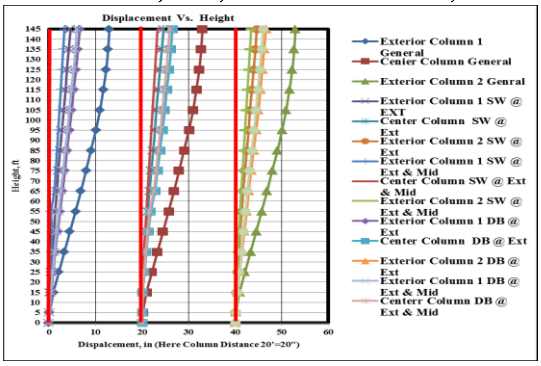


Fig. 5: Comparison of Displacements for Different Cases.

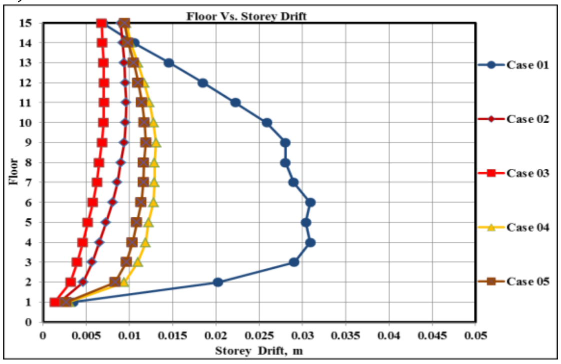


Fig. 6: Comparison of Storey Drift for Different Cases.

N.B. Only In displacement graph representation the authors had to follow FPS unit for appropriate observation and concise view.

Lateral Stiffness Analysis

In measuring lateral stiffness, displacements and storey drift hold a great importance. In this work, displacement means deflection of column in Z-direction i.e. along the lateral forces. For 14th floor %increment in lateral stiffness corresponding to Case 01: Case 02 = 179.71%; Case 03 = 285.44%; Case 04 = 97.28%; Case 05 = 115.59.

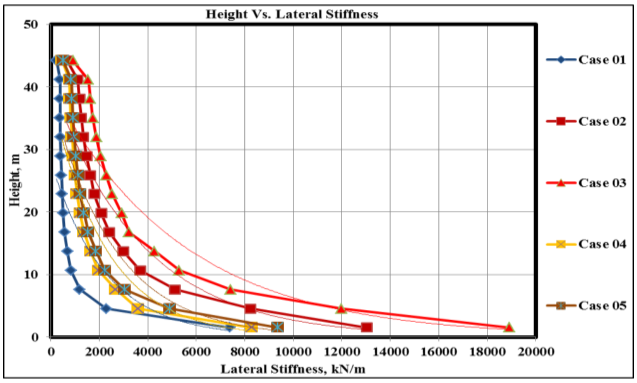
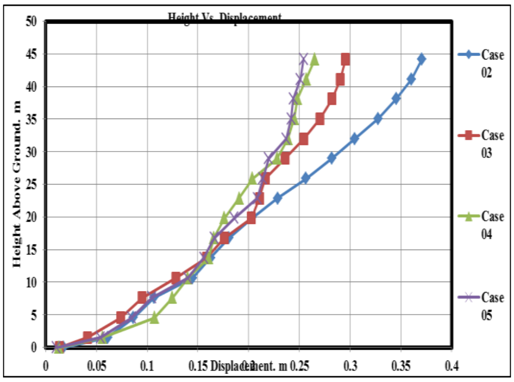


Fig. 7: Comparison of Lateral Stiffness for Different Cases.

Dynamic Analysis

Dynamic analysis is consists of Time History and Response Spectrum Analysis. Displacements from these Analyses are compared for different cases. In time history analysis for 14th floor %reduction in displacement in corresponding to Case 02: Case 03 = 20.34%; Case 04 = 28.49%; Case 05 = 31.44%. In response spectrum analysis for 14th floor %reduction in displacement in corresponding to Case 02: Case 03 = 5.29%; Case 04 = 21.44%; Case 05 = 23.71%.



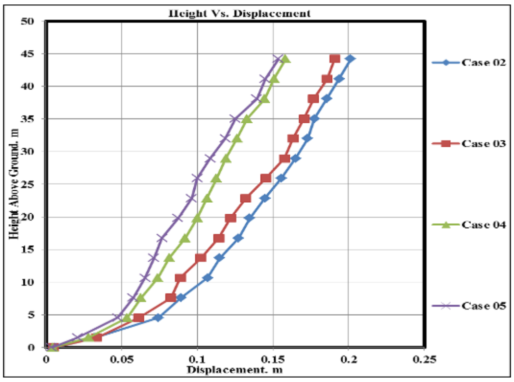


Fig. 9: Comparison of Displacements from Response Spectrum Analysis for Different

Fig. 8: Comparison of Displacements from Time History Analysiss for Different Cases.

CONCLUSION AND RECOMMENDATION

The findings of the study presented in the previous chapter are summarized below:

- Lateral stiffness increased at a great extent for different structural retrofitting system.
- Lateral displacement also reduced greatly for different structural features.
- Column axial load also reduced significantly for the cases of Flat Plate with Shear Walls at Most Exterior Panels and Flat Plate with Shear Walls at Most Exterior and interior Panels.
- In Dynamic analysis we obtained better lateral stiffness for Flat Plate with Shear Walls at Most Exterior Panels and Flat Plate with Shear Walls at Most Exterior and interior Panel cases.

It is hereby recommend that, A flat building should be retrofitted with shear wall at exterior panels along weak direction when gravity loads govern and with diagonal bracings where buildings are highly susceptible to lateral loadings.

ACKNOWLEDGEMENTS

It is indeed an enormous pleasure and great privilege for the authors to express their deepest gratitude to the supervisor Dr.Md. Mahmadur Rahman, Professor, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh for his constant guidance and continuous encouragements to carry out the research and his invaluable suggestions. Acknowledgements are also due for Dr. Md. Anwarul Mustafa, Professor and Head, Department of Civil Engineering, Dr.Md.Shahid Mamun, Associate Professor, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh for his support to carry out the research works. The authors also like to pay their deepest homage to their parents and family members for their continuous cordial encouragements.

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AN EXPERIMENTAL STUDY ON INFLUENCE OF RECYCLED AGGREGATES ON CONCRETE COMPRESSIVE STRENGTH IN MARINE ENVIRONMENT

M. R. Mukhlis[1*], M. R. Hoque[1] & M. T. Alam[2]

[1] *Department of Disaster and Environmental Engineering, Chittagong University of Engineering and Technology, Chittagong – 4349, Bangladesh.*
[2] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong – 4349, Bangladesh.*
**Corresponding Author, <raihan.ce@live.com>*

ABSTRACT

Every year a lot of newer structure is rising and making the land for construction smaller day by day which leads the construction heading towards the coastal areas. But seawater intrusion in coastal regions has a great impact on the strength of concrete structures. Again recycled aggregates of demolished concrete are being wasted without any beneficial use. These recycled aggregate can be reused in concrete construction in the coastal regions if it is possible to gain a rational percentage of strength after some specified curing periods.So to study the optimum percentage of recycled aggregate on recycled concrete in marine environment, three types of variables are used in this study. Percentage of recycled aggregate, curing period and concentration of saline water are those three variables involved. For this study 3 various percentage of recycled aggregate used in concrete casting which are 30%, 40% and 50% recycled aggregate. For the 3 different types of aggregate samples 3 different types of concrete has been cast in 100 mm cube specimen and cured in 3 different concentrations of 1N, 3N and 5N curing water. The specimens are then tested for compressive strength for curing periods of 28 days, 60 days and 90 days. The results of compressive strength test of these recycled concrete are then compared with the compressive strength of plain concrete with 0% recycled aggregate cured in plain water by considering it as a standard.

From the investigation work it has been found that concrete cast with 30% recycled aggregate achieves 95.4% compressive strength for 28 days curing in 1N concentration of saline water, which is comparatively higher among other recycled concrete used here. Percentage of strength achievement are higher for 28 days curing but it starts to decrease with the increase of curing period in saline water compared to the strength of concrete cured in plain water. Compressive strength decreases with increasing percentage of recycled aggregate and increasing concentration of salinity. Thus 30% is the optimum percentage of recycled aggregate which is accessible to use in marine structures only when curing in high saline water can be avoided as much as possible.

Keywords: Recycled Aggregate, Saline Water, Curing Period, Compressive Strength.

1. INTRODUCTION

Nowadays in the process of development, lots of engineering construction is going on including high rise building, bridge etc. Every year a lot of newer structure is rising with the economic development of the country. As a result the land for construction is getting smaller day by day. So undoubtedly the construction companies are heading towards the coastal areas. Many concrete structures are needed to construct in the coastal regions such as sea walls, concrete blocks, ship yard, ports, dry docks as well as residential and commercial buildings. But seawater intrusion in coastal regions has a great impact on the strength of concrete structures in these regions. Meanwhile huge amount of recycled aggregates of demolished concrete are wasted without any beneficial use of it. But it is very much possible to use huge part of demolished concrete as recycled aggregate in concrete constructions. These two practical problems lead us to do this investigation work.

Using various percentage of recycled aggregate for casting and saline water for curing it is tried to find out the adverse effect of saline water in the compressive strength of recycled aggregate concrete to that of ordinary concrete. This study will show the allowance of using recycled aggregate on concrete construction in marine environment.

If optimum amount of recycled aggregate is used in concrete construction in marine environment then construction cost will be reduced. But saline water has different concentration at different places so the effect of saline water on concrete strength differs. So it is quite difficult to investigate the behavior of compressive strength of concrete in marine environment. Comparing the strength obtained for different samples of concrete cured in different concentration of saline water, a rational conclusion for the investigation can be drawn. This investigation will be beneficial for further use of recycled concrete in construction works of sea wall, dockyard etc. only if it is possible to gain a rational percentage of strength after some specified curing periods.

2. MATERIALS AND METHODS

Relatively higher strength concrete with a lower permeability is commonly used for the construction of marine structures. Recycled aggregates of demolished concrete are used for the investigation in addition with all common construction materials. The methods used for the investigation includes ACI mix design for achieving desired compressive strength after 28 days by casting 100 mm cube specimens for different combinations of material samples.

2.1 Materials

A brief description of the constituent materials used in the present investigation is given below. The materials used are ordinary Portland cement, fine aggregates, natural and recycled coarse aggregates, plain water for casting and saline water for curing and curing tank. The fine aggregates used are locally available Sylhet sand physical properties of the fine aggregates are given below in Table 1.

Table 1: Properties of fine aggregates

Fineness modulus	2.54
Specific gravity	2.59
Absorption capacity (%)	1.70
Moisture content (%)	2.65

The coarse aggregates used are locally available natural coarse aggregates and recycled coarse aggregates from demolished concrete cubes and cylinders of materials laboratory of Civil Engineering Department, CUET. The spare cube and cylinder specimens remain unused and generating wastes which are demolished first and then sieved by standard sieves. The physical properties of the coarse aggregates are given below in Table 2.

Table 2: Properties of coarse aggregates

Sample Property	0R	30R	40R	50R
Unit Weight (Kg/m^3)	1515	1505	1494	1487

Bulk specific Gravity(SSD)	2.53	2.53	2.52	2.52
Absorption capacity (%)	1.1	1.4	1.7	2.1
Moisture content (%)	0.3	0.4	0.5	0.6

* 0R, 30R, 40R, 50R: 0%, 30%, 40%, 50% recycled aggregates respectively

2.2 Mix Proportions

Concrete mix ratio is designed on the basis of ACI mix design where the design compressive strength of concrete is taken as 3000 psi and theoretical water cement ratio as 0.45 considering different limiting criteria. This specific strength is taken in this experimental investigation as this concrete strength is most widely used in our country. Among the 4 different samples of coarse aggregate 0% recycled aggregate (0R) indicates the natural aggregate, which is used for casting normal concrete and served as reference concrete in this investigation. As 4 different samples of coarse aggregates have been used, 4 different concrete mix ratios have been found which are mentioned in the following Table 3.

Table 3: Mix ratio for different coarse aggregate sample

Sample	Natural aggregate	Recycled Aggregate	Mix Ratio
Sample 1 (0R)	100%	0%	1 : 1.61 : 2.95
Sample 2 (30R)	70%	30%	1 : 1.62 : 2.93
Sample 3 (40R)	60%	40%	1 : 1.62 : 2.92
Sample 4 (50R)	50%	50%	1 : 1.63 : 2.91

* 0R, 30R, 40R, 50R: 0%, 30%, 40%, 50% recycled aggregates respectively

2.3 Preparation of Test Specimens

A total of 144 cubes are cast for 4 different samples of coarse aggregates, where each sample is used to cast 36 cube specimens. The size of each cube was 100 mm × 100 mm × 100 mm. The 144 cubes are then cured in 4 curing tanks containing 4 different type of curing water, where each tank contains a total of 36 cubes. Salinity of different curing water in different curing tanks is shown in following Table 4.

Table 4: Salinity of different curing tank

Curing Tank	No. of cube specimen	Curing water type	Weight of NaCl in 1m^3 water
Tank 1	36	Plain water (0N)	0 kg
Tank 2	36	1Normal saline water (1N)	25.053 kg
Tank 3	36	3 Normal saline water (3N)	75.159 kg
Tank 4	36	5 Normal saline water (5N)	125.265 kg

2.4 Compressive strength test details

Three different curing periods of 28 days, 60 days and 90 days are used here to compare the compressive strength of concrete in different salinity. Among the 36 cubes of each tank 12 cubes are tested for compressive strength after each specified curing periods. For example after 28 days 12 cube specimens from each of the 4 curing tank resulting a total of 48 cubes are withdrawn for concrete compressive strength test. Thus 48 cubes are tested after each specified curing periods. The 12 cube specimens withdrawn every time from each curing tank includes 4 different types of concrete cast with 4 different samples of coarse aggregates mentioned earlier. Each of the 4 different types of concrete consists of 3 similar types of test specimens on the basis of coarse aggregate, curing water and curing period. By averaging the compressive strength of these 3 similar types of test specimens every time, a rational comparison between different concrete test specimens and with reference concrete test specimens can be conducted.

3. RESULTS & DISCUSSIONS

Compressive strength value found for different samples of concrete in different curing condition have been compared to each other to find out the optimum amount of recycled aggregates to be used and the effect on the compressive strength of the concrete due to curing water having different concentration of salinity after some specified age of curing. For the variation of compressive strength with respect to salinity of curing water as shown in figure 1, sample cast with natural aggregate and cured in plain water for 28 days (N_{28}) shows higher strength of 3050 psi whether sample cast with 50% recycled aggregate and cured in 5N saline water for 28 days ($50R_{28}$) shows lowest strength of 2410 psi. The concrete sample cast with 30% recycled aggregate and cured in 1N saline water for 28 days ($30R_{28}$) shows comparatively better and reasonable compressive strength of 2970 psi compared to other samples of concrete which are cast with some percentage of recycled aggregate and cured in some concentration of saline water as shown in following figure 1.

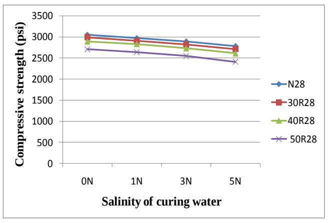

Fig. 1: Compressive strength vs. salinity of curing water for 28 days curing

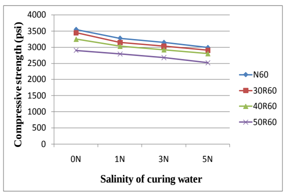

Fig. 2: Compressive strength vs. salinity of curing water for 60 days curing

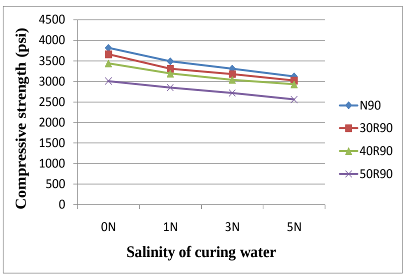

Fig. 3: Compressive strength vs. salinity of curing water for 90 days curing

Similarly figure 2 and 3 show not only the effect of replacing natural aggregate with recycled aggregate but also the effect of different concentration of salinity in curing water on concrete compressive strength for 60 days and 90 days curing respectively as like figure 1 shows for 28 days curing. It is clearly observed that compressive strengths are decreasing with the increase of percentage of recycled aggregates and also with the increase of concentration of salinity in curing water. In every figure mentioned above concrete cast with 30% recycled aggregate and cured in 1N saline water (represented by $30R_{28}$, $30R_{60}$, $30R_{90}$ in figure 1, 2 and 3 respectively) shows reasonable and nearest compressive strength to that of reference concrete sample cast with 0% recycled aggregate (100% natural aggregate) and cured in plain water which are represented by N_{28}, N_{60} & N_{90} in figure 1, 2 and 3 respectively.

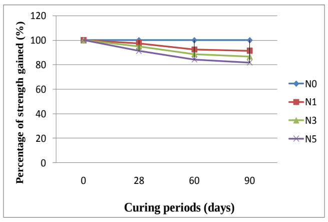

Fig. 4: Relative Compressive strength vs. curing period for Normal concrete

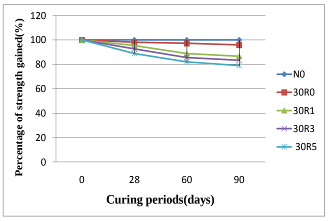

Fig. 5: Relative Compressive strength vs. curing period for 30% Recycled concrete

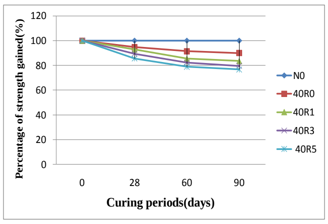

Fig. 6: Relative Compressive strength vs. curing period for 40% Recycled concrete

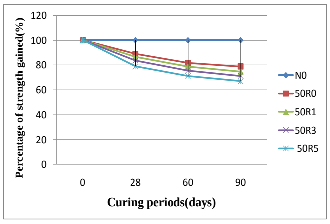

Fig. 7: Relative Compressive strength vs. curing period for 50% Recycled concrete

Taking the compressive strength of sample cast with natural coarse aggregate and cured in plain water (N_0) as 100% all the relative strength gained by other sample can be easily analyzed for any periods of curing. Figure 4, 5, 6 and 7 represents all the relative compressive strength gained by concrete with respect to curing periods for concrete samples cast with natural coarse aggregate, 30%, 40% and 50% recycled aggregates respectively. Figure 4 represents the percentage of compressive strength gained by normal concrete cured in plain water, 1N, 2N and 3N saline water respectively. Comparatively to N_0 sample concrete, the relative strength gained by samples of 30% recycled concrete cured in plain water ($30R_{0}$) are largest among all recycled concrete samples cured in plain water such as 98%, 97.2% and 95.8% for 28, 60 and 90 days respectively as shown in figure 5. Among all recycled concrete samples cured in 1N saline water, sample $30R_1$ gains maximum amount of relative strength of 95.4%, 88.7% and 86.6% for 28, 60 and 90 days respectively as shown in figure 5. Similarly among all recycled concrete samples cured in 3N saline water and 5N saline water, sample $30R_3$ and $30R_5$ gains maximum amount of 28 days relative strength of 92.5% and 88.9% respectively

Among all recycled concrete samples cured in any saline water, sample $50R_5$ gains worst amount of relative strength of 79%, 71% and 67% for 28, 60 and 90 days respectively. From which it is clear that a maximum value of 33% of relative strength gaining is not possible for the worst sample. The permeability of recycled aggregate as well as recycled concrete is higher than ordinary concrete. So sea water intruded rapidly at the time of moist curing of recycled concrete. The various chemical reagents present in the sea water reacts with the constituents of concrete and imposed adverse effect on compressive strength gaining property of cement and concrete. Lower densities and higher water absorption levels of recycled aggregates compared to that of natural aggregates are the main reasons of lower strength of recycled concrete.

4. CONCLUSION

The behavior pattern of concrete compressive strength and the detailed findings of the investigation can be summarized in short as below:

1. Saline water has adverse effect on the compressive strength of ordinary concrete cast with natural aggregates as coarse aggregate. But saline water shows more tremendously adverse effect on the compressive strength of recycled concrete cast with large percentage of recycled aggregates as coarse aggregate.
2. 30% recycled aggregate is the optimum amount of recycled aggregate to be used in recycled concrete construction in marine environment as concrete cast with 30% recycled aggregate achieves more than 95% compressive strength for 28 days curing in 1N concentration of saline water.
3. For recycled concrete with higher percentage of recycled aggregate cured in high saline water the percentage of strength gained decreases more steeply and rapidly.
4. Larger the amount of recycled aggregates lesser will be the compressive strength of recycled concrete.
5. Larger the amount of salinity in curing water lesser will be the compressive strength of both ordinary and recycled concrete.
6. Larger the amount of salinity and recycled aggregate lesser will be the percentage of strength gaining with respect to time.

5. ACKNOWLEDGMENTS

The authors wish to acknowledge immeasurable grace and profound kindness of Almighty ALLAH, the great gracious and merciful, who enables the authors to complete the dissertation.

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EXPERIMENTAL INVESTIGATION OF FLEXURAL TENSION, SPLITTING TENSION AND DIRECT TENSION CAPACITY OF STEEL FIBER REINFORCED CONCRETE (SFRC)

M. M. Islam[1]*, M. A. Sayeed[1], E.A. Hossain[1], S.S. Ahmed[1], M.A. Chowdhury[1] & A. Siddique[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.*
**Corresponding Author, <mashfiq7777@gmail.com>*

ABSTRACT

Concrete is a brittle material and is very weak in tension. When steel fibers are added to a concrete mix, fibers distribute randomly and act as crack arrestors, changing concrete from a brittle material to a ductile one and prevent brittle failure. Experimental investigations are conducted to increase the tensile strength of concrete by using steel fiber. Volume fraction of steel fiber used in this study is 1.5%. In this study low aspect ratio of fiber is used and sufficient capacity enhancement is attained. Two different aggregate types are used to make concrete i.e. stone and brick aggregate and the effects of SFRC on these two types of concretes are evaluated. Stone and brick concretes show substantial increase the flexural tension, splitting tension and direct tension (by dogbone specimen) capacity as well as ductility due to the presence of steel fibers. Total 8 compression cylinders, 8 split cylinders, 8 dogbone specimens (direct tension) and 8 beams (flexural tension) are casted and analyzed in this study. The compressive strength of brick SFRC with 1.5 percent of steel fiber volume ratio is increased by about 12-41%. In tensile strength test, the tensile capacity of brick SFRC increases by about 74-144% for direct tension, about 2-68% for splitting tension and about 13-18% for flexural tension. For stone SFRC, the compressive strength increased about 31-57%, tensile strength increased for direct tension about 52-154%, splitting tension about 17-82% and flexural tension about 11-24%. The use of steel fibers yet not started in the construction industry of Bangladesh for the absence of reliable experimental results. In context of Bangladesh, this study investigates the effectiveness of locally available steel fiber in tensile capacity enhancement of concrete members.

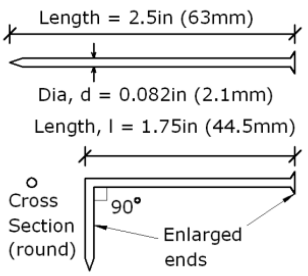
Keywords: Steel Fiber Reinforced Concrete (SFRC), Flexural Tension, Splitting Tension, Direct Tension, Dogbone Specimen.

INTRODUCTION

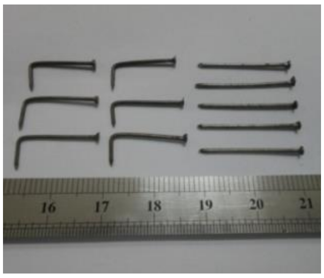
Concrete shows a low tensile strength in comparison to the compressive strength, which grows only less proportionally with increasing compressive strength; at the same time, the brittleness increases. Therefore, steel fibers are added to improve ductility and to increase the tensile strength. For many applications, however, conventional reinforcement in the tensile zone is still necessary. Fibers are increasingly being used in concrete structures to compensate for concrete's weak and brittle tensile behavior relative to its compression response. One of the most beneficial aspects of the use of fibers in concrete structures is that non-brittle behavior after concrete cracking can be achieved with fibers. The tensile stress in concrete rapidly decreases immediately after cracking. In steel fiber reinforced concrete (SFRC), on the other hand, fibers crossing the crack interfaces significantly contribute to the load-carrying mechanism so that considerable tensile stress, being the sum of the tensile resistance provided by fibers and tension softening of the concrete matrix, respectively, can be achieved even with large crack widths (Lee et al. 2011). Therefore, the enhanced tensile stress behavior attainable with fibers should be realistically evaluated to accurately predict the post-cracking response of SFRC. The bond resistance of reinforcing bars embedded in concrete depends primarily on frictional resistance and mechanical interlock. The chemical adhesion bond, if any, fails at very small slips. Frictional bond provides initial resistance against loading and further loading mobilizes the mechanical interlock between the concrete and bar ribs. Mechanical interlock leads to inclined bearing forces, which in turn lead to transverse tensile stresses and internal inclined splitting (bond) cracks along reinforcing bars (Chao et al. 2009). To this end, this research is designed to investigate the flexural tensile, splitting tensile and direct tensile strength of steel fiber reinforced concrete (SFRC) and compared with plain concrete. The current research aims to investigate the capacity enhancement and stress field of the SFRC from experimental viewpoint as well as to introduce this new engineering material in the construction industry of Bangladesh.

EXPERIMENTAL PROGRAM

This research intended to study the tensile capacity of SFRC. To this end, three types of tension specimen is considered, they are split tension (represented as T in specimen ID), flexural tension (F) and direct tension (D). In this research, dog-bone specimens are introduced to determine direct tensile strength of plain concrete and SFRC. Two types of SFRC and plain concretes i.e. brick (represented as CB in specimen ID) and stone (CS), are tested experimentally. The fiber volume is taken 1.5% to cast the tensile specimens. According to ACI-544.4R-88, enlarged-end fibers show the maximum energy absorption as well as tensile strength enhancement. In this research, SFRC specimens are prepared with enlarged end (EE) fibers, straight (ST) fibers and enlarged end & smaller straight 50-50 mixed (ES) fibers to evaluate the performance of SFRC. The tensile strength of steel fiber is 552 MPa (80 ksi) which satisfies the minimum requirement of ASTM A 820/A 820M-06 (i.e. 345 MPa or 50 ksi) and the aspect ratio of fiber is 21.6. Steel fibers available in market are customized to make enlarged ends for better anchorage (Fig. 1a,b). A total of 8 compression cylinders, 8 split cylinders, 8 dogbone specimens and 8 flexural specimens are made for this research. The dia of the cylinders are 100mm (4in) and length is 200mm (8in). The web of the dogbone specimen is 100x100x150mm (4x4x6in) and the size of the flexural specimens is 100x100x400mm (4x4x16in). The dog-bone specimens are notched at the middle of the web in four sides which acted as stress concentrator as well as to control the failure location. The mould for casting of dogbone specimen is shown in Fig. 1c and Fig. 1d shows SFRC split cylinder specimen test setup. All the specimens are tested in a 1000kN capacity displacement controlled digital universal testing machine (UTM) (Fig. 1e). Strain data are measured by applying digital image correlation technique (DICT) using high definition (HD) images and high speed video clips and these data are synthesized with the load data from the load cell of UTM (Islam et al. 2011, Islam et al. 2014a,b).



(a)



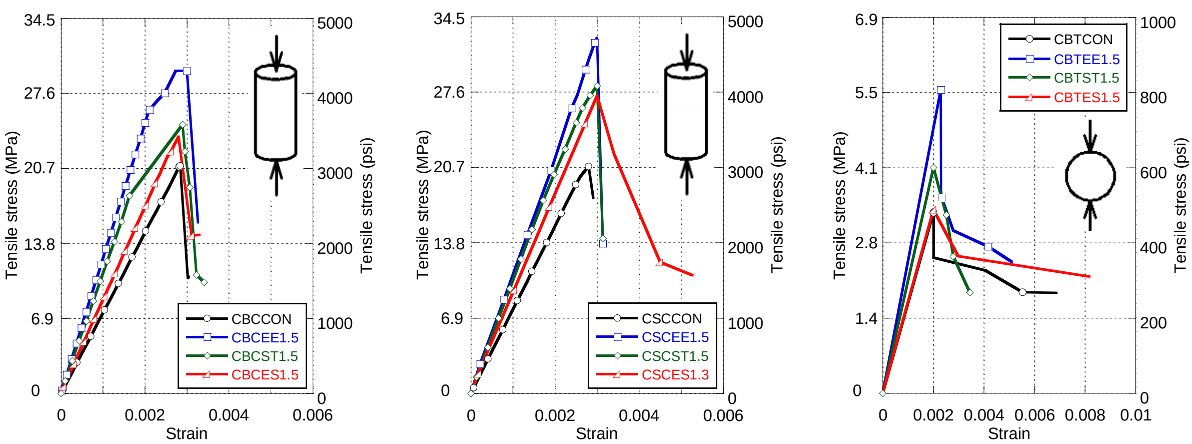
(b)

(c) (d) (e)

Fig. 1. (a), (b) Geometry and image od steel fiber, (c) mould of dogbone specimen, (d) split tension test setup, (e) test setup for dogbone specimen.

RESULTS AND DISCUSSION

The graphical representations of the experimental results are shown in Fig. 2. The compressive strength is increased 41.3%, 17.9% and 12.3% for brick SFRC made of EE, ST and ES fibers respectively (Fig. 2a) and in the case of stone SFRC these values are 57.3%, 36.1% and 31.5% respectively (Fig. 2b). The spliting tensile strength is increased 68.9%, 25.4% and 2.8% for brick SFRC made of EE, ST and ES fibers respectively (Fig. 2c) and in the case of stone SFRC these values are 82.5%, 23.7% and 17.6% respectively (Fig. 2d). The direct tensile strength is increased 144.3%, 97.0% and 74.4% for brick SFRC made of EE, ST and ES fibers respectively (Fig. 2e) and in the case of stone SFRC these values are 154.2%, 64.5% and 52.7% respectively (Fig. 2f). The flexural tensile strength is increased 18.6%, 14.3% and 13.0% for brick SFRC made of EE, ST and ES fibers respectively (Fig. 2g) and in the case of stone SFRC these values are 24.5%, 16.6% and 11.9% respectively (Fig. 2h). These results are summerised in Table 1. The failure pattern of tension specimens are represented in Fig. 3. The plain concrete specimens showed wider crack with complete separation whereas SFRC tensile specimens showed crack bridging, slow crack propagation and no separation of the pieces. SFRC showed non-brittle fracture during loading which will help during deadly seismic loading. The SFRC made of enlarged end (EE) fibers showed better tensile capacity enhancement compared to straight (ST) and mixed (ES) fibers. Enlarged end fibers provide better anchoring in the concrete matrix that helped to make crack bridging. Stone SFRC specimens showed better enhancement compared to brick SFRC.



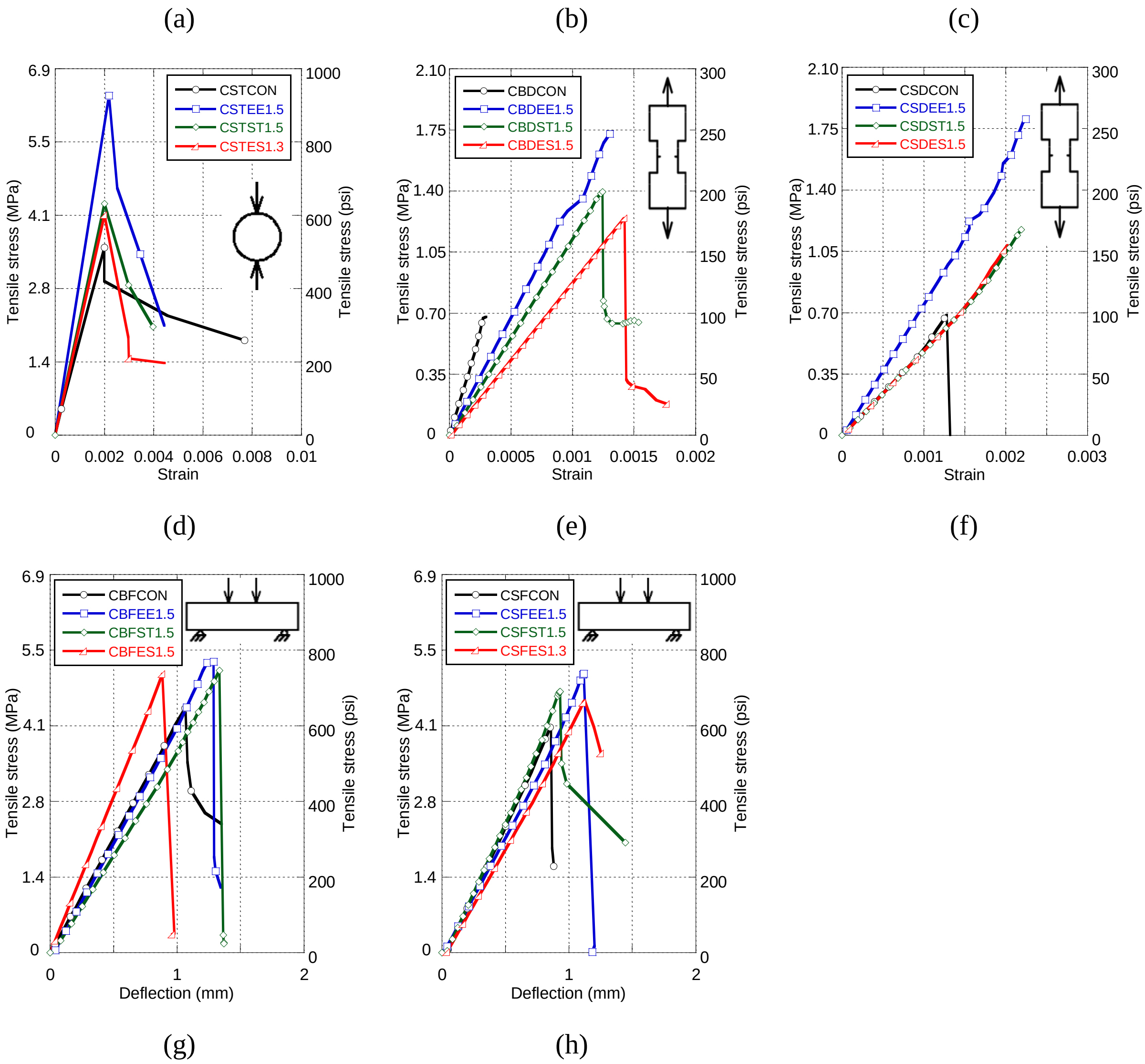


Fig. 2. Graphical representation of experimental results (a) & (b) compression test, (c) & (d) split tension test, (e) & (f) direct tension test of dogbone specimen, (g) & (h) flexural tenison test made of brick and stone concrete/SFRC respectively

Table 1: Experimental test results of shear specimens

Specimen ID	Comp. Stress (MPa)	% Increased	Specimen ID	Tensile stress (Mpa)	% Increased	Specimen ID	Tensile stress (MPa)	% Increased
CBCCON	21.0	-	CBTCON	3.3	-	CSTCON	3.5	-
CBCEE1.5	29.7	41.3	CBTEE1.5	5.6	68.9	CSTEE1.5	6.4	82.5
CBCST1.5	24.8	17.9	CBTST1.5	4.1	25.4	CSTST1.5	4.3	23.7
CBCES1.5	23.6	12.3	CBTES1.5	3.4	2.8	CSTES1.5	4.1	17.6
CSCCON	20.7	-	CBDCON	0.7	-	CSDCON	0.7	-
CSCEE1.5	32.6	57.3	CBDEE1.5	1.7	144.3	CSDEE1.5	1.8	154.2
CSCST1.5	28.2	36.1	CBDST1.5	1.4	97.0	CSDST1.5	1.2	64.5

CSCES1.5	27.2	31.5	CBDES1.5	1.2	74.4	CSDES1.5	1.1	52.7
			CBFCON	4.5	-	CSFCON	4.1	-
			CBFEE1.5	5.3	18.6	CSFEE1.5	5.1	24.5
			CBFST1.5	5.1	14.3	CSFST1.5	4.8	16.6
			CBFES1.5	5.1	13.0	CSFES1.5	4.6	11.9

CBDCON CBDEE1.5 CSDCON CSDEE1.5 CBDCON & CSDCON

CBTCON CBTEE1.5 CSTCON CSTEE1.5

CBFCON CBFEE1.5 CSFCON CSFEE1.5

Fig. 3. Typical failure of tension specimens

CONCLUSION

SFRC construction is not yet been started in the construction industry of Bangladesh though it possess improved engineering and mechanical properties due to lack of reliable experimental results. The compressive strength of brick SFRC with 1.5 percent of steel fiber volume ratio is increased by about 12-41%. In tensile strength test, the tensile capacity of brick SFRC increases by about 74-144% for direct tension, about 2-68% for splitting tension and about 13-18% for flexural tension. For stone SFRC, the compressive strength increased about 31-57%, tensile strength increased for direct tension about 52-154%, splitting tension about 17-82% and flexural tension about 11-24%. SFRC made of enlarged end (EE) fibers showed better tensile capacity enhancement compared to straight (ST) and mixed (ES) fibers. Stone SFRC specimens showed better enhancement compared to brick SFRC. Moreover, SFRC specimens showed better crack control properties as well as ductility. This paper provides reliable experimental results and analysis which will help to introduce SFRC in the construction industries of this country.

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Lee, SC; Cho, JY and

ID: SEE 007

SEISMIC ANALYSIS OF SOME HISTORICAL MASONRY STRUCTURES IN BANGLADESH

F. Mehzabin[1*] & I. Anam[1]

[1] *Department of Civil Engineering, University of Asia Pacific, Dhaka, Bangladesh.*
**Corresponding Author, <mehjabin-ce@uap-bd.edu>*

ABSTRACT

This paper combines a general study on the seismic behaviour of unreinforced masonry with emphasis on particular historic buildings. Lateral stiffness calculated by an analytical method is found to be within 10% of the result from the software ETABS, justifying its subsequent use in this work. Structural dynamic analysis is carried out for four historical masonry mosques and two Panam Nagar buildings. The masonry buildings, modelled as lumped mass dynamic systems, are subjected to recorded ground motions from two major earthquakes. Results show that bond stresses in the historic mosques are almost invariably less than their minimum bond strengths, indicating their likely survival in the earthquakes studied. However, the corresponding stresses for the Panam Nagar buildings are almost always found to exceed their bond strengths. The very thick walls of the historic mosques are also found to satisfy the out-of-plane stability criteria, whereas the Panam Nagar walls fail again, indicating their possible instability and collapse in major earthquakes.

Keywords: Historic building, unreinforced masonry, earthquake, bond strength, out-of-plane stability.

INTRODUCTION

Unreinforced masonry (URM) structures are the most vulnerable during an earthquake due to the brittle nature of URM, large mass of masonry structures, large initial stiffness and large variability in masonry material properties. The breakdown of earthquake fatalities by cause for each half of the last century indicates that 75% of the fatalities are due to collapse of buildings (Coburn & Spence, 2002), more than 60% of which are due to the collapse of URM. However, accurate structural analysis of masonry structures remains relatively neglected due to the composite nature of brick masonry and limited knowledge of their structural behaviour.

In this work, some historic masonry structures are analysed numerically subjecting them to seismic ground motions, using the material properties of masonry. This includes the analytical derivation of lateral stiffness of masonry walls using an approximate method. These stiffnesses are then used for dynamic analysis of the masonry structures using the lumped mass model.

RIGIDITY OF MASONRY WALL

The lateral load capacity of shear wall is mainly dependent on the in-plane resistance rather than out-of-plane stiffness. The distribution of lateral load to the masonry walls is based on the relative wall rigidities, which is dependent on its dimensions, modulus of elasticity, modulus of rigidity and the support condition. The following steps are required to calculate the rigidity of a wall with opening (Drysdale et al. 1994), combining the deflection of the solid wall as cantilever ($\Delta_{soild(C)}$), the cantilever deflection of an interior strip having a height equal to that of the highest opening ($\Delta_{strip\ of\ highest\ opening(C)}$) and the deflections of all the piers as fixed within that interior strip ($\Delta_{pier(f)}$). The text by Agarwal & Shrikhande (2006) has been used for part of the theoretical formulation in the present work.

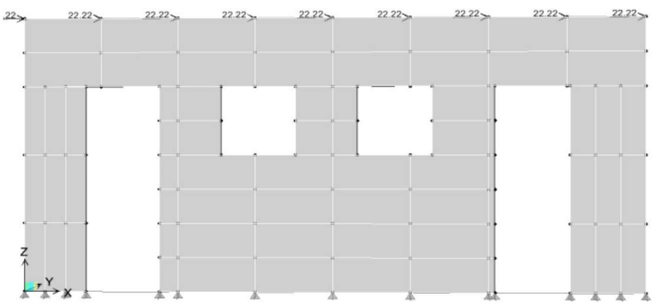


Fig. 1: Elevation of a Wall Model in ETABS

A sample masonry wall with two windows and two doors is used as structural model (Fig. 1) and subjected to a distributed lateral load of 200 kN. Here the modulus of elasticity of the masonry wall is assumed to be 957000 kPa (i.e., 957 MPa), while the wall thickness is taken as 230 mm.

Fig. 2 shows the output data for the lateral deflection of the left and corner of the wall, where the principal deflections are shown to

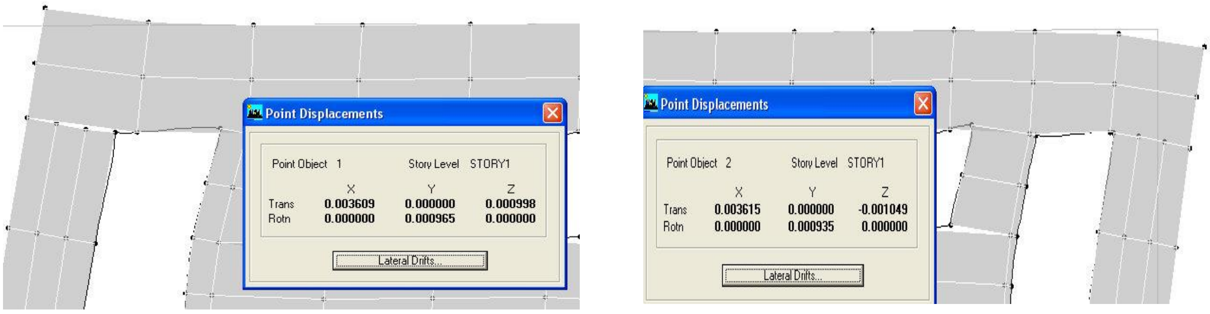


Fig. 2: Deflection at left and right Corner of Wall Model using ETABS

be 0.003609 m and 0.003615m.

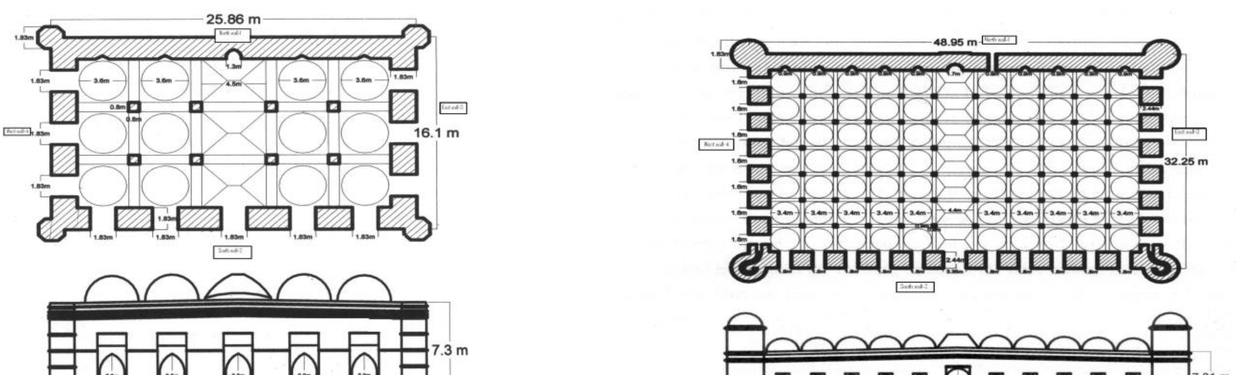
$\therefore$ Average deflection = 0.003612 m = 3.612 mm $\Rightarrow$ Rigidity of the wall = 200/3.612 = 55.37 kN/mm

Using the approximate method, rigidity of the wall = 0.279Et = 0.279 × 957 × 230 = 61.41 kN/mm

$\therefore$ Horizontal deflection due to the 200 kN lateral load is, Δ = 200/61.41 = 3.257 mm
Therefore, the results from the approximate method are within 10% of the results from ETABS, which justifies the use of the approximate method in this work.

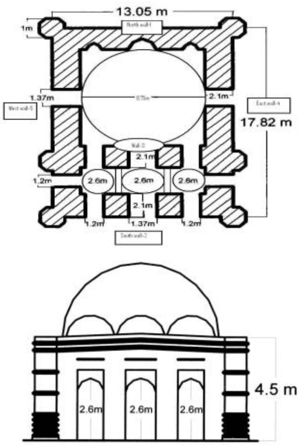
SELECTED MASONRY STRUCTURES

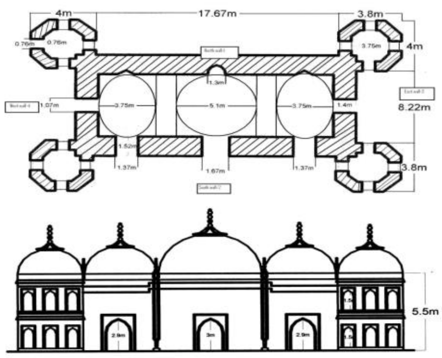
Masonry structures selected for subsequent structural analyses are four masonry mosques built between 13th and 18th century and are distinguished by their historic appeal. Also presented are two buildings of Panam Nagar, a distinctive example of early urban settlement of Bengal, located near Sonargaon, one of the prosperous capital cities in Mediaeval Bengal. Among the various masonry structures studied (e.g., Mehzabin, 2012), the Chhoto Sona Mosque (Chapainawabganj), Shait Gambuj Mosque (Bagerhat), Rajbibi Mosque (Gaur Chapainawabganj), Sat Gambuj Mosque (Dhaka) [Fig. 3(a)~(d)] and two buildings of the historical Panam city (Sonargaon); i.e., Chhoto Sardar Bari, Building No. 38 [Fig. 3(e,f)] are analyzed for ground vibration due to recorded motions from El Centro (1940) and Kobe (1995) earthquake.



(a) Chhoto Sona Mosque

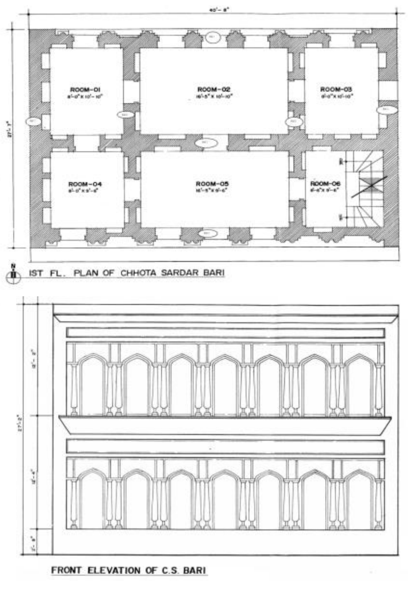
(b) Shait Gambuj Mosque

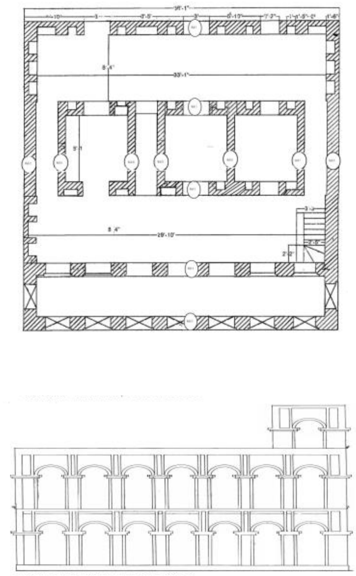




(c) Rajbibi Mosque

(d) Sat Gambuj Mosque





(e) Chhoto Sardar Bari

(f) I

Fig. 3: Floor Plan and Front Elevation of historical mosques (Kamruzzaman, 2011) and Panam Nagar Buildings (Ahmed, 2006; Mowla, 2008)

RESULTS FROM DYNAMIC ANALYSES

This section presents the numerical results from the dynamic analysis of the brick masonry buildings under study, including their relative floor deflections when subjected to ground motions from the El Centro and Kobe earthquake in the North-South (NS) as well as East-West (EW) directions.

Figs. 4 and 5 show the variation of the floor deflections (NS, EW for El Centro and Kobe earthquake motion) with time, for two typical buildings under study (i.e., Chhoto Sona Mosque subjected to El Centro motion and Chhoto Sardar Bari for Kobe motion). Table 1 shows a summary of the numerical results, including the maximum relative deflections as well average shear strains and bond shear stresses involved in each case. The results show that while most of the historical mosques under study should again be safe against shear bond failure (assuming a range of strengths between 0.05~0.25 MPa), almost all the Panam Nagar buildings would again be at risk and some of them would definitely suffer shear bond failure between walls, leaving the walls vulnerable to out-of-plane failures.

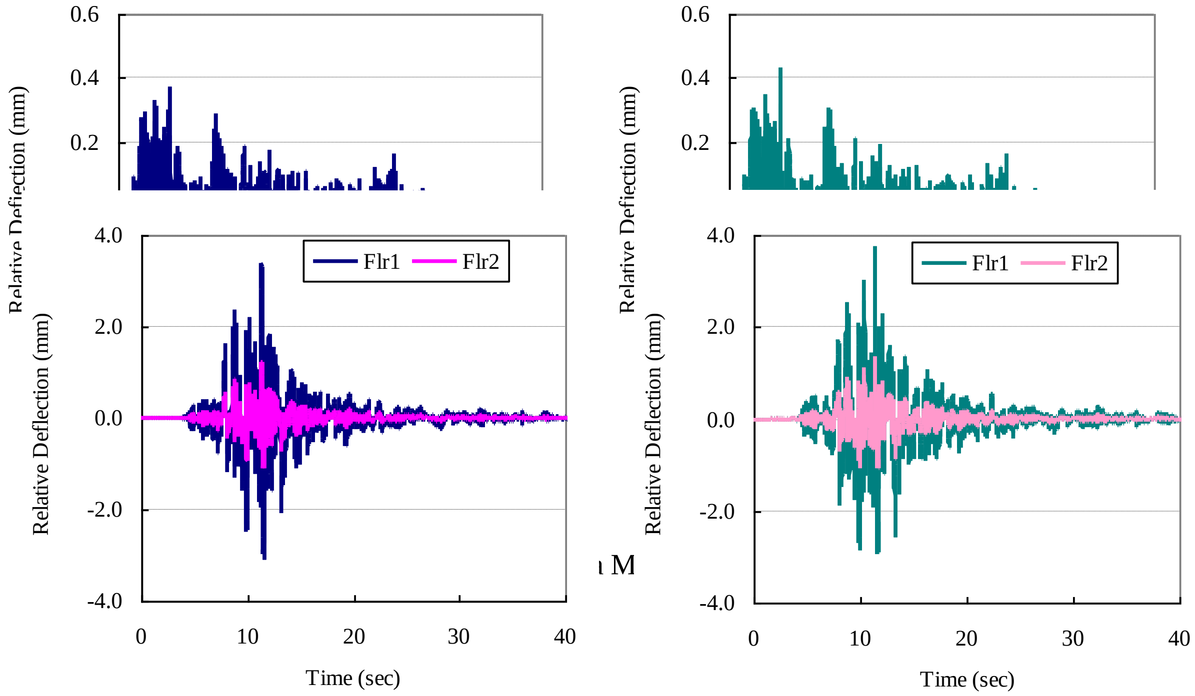


Fig. 5: Relative deflection of Chhoto Sardar Bari for Kobe along (a) NS, (b) EW

Table 1: Results from Dynamic Analysis

Structure	Maximum Value	Earthquake			
		El Centro NS	El Centro EW	Kobe NS	Kobe EW
Chhoto Sona	$\Delta_{rel(1)}$ (mm)	0.586	0.564	0.517	0.568

Mosque	$\tau_{avg(1)}$ (MPa)	0.051	0.049	0.045	0.049
Shait Gambuj Mosque	$\Delta_{rel(1)}$ (mm)	0.661	0.236	0.689	0.312
	$\tau_{avg(1)}$ (MPa)	0.057	0.020	0.060	0.027
Rajbibi Mosque	$\Delta_{rel(1)}$ (mm)	0.057	0.061	0.095	0.103
	$\tau_{avg(1)}$ (MPa)	0.008	0.009	0.013	0.015
Sat Gambuj Mosque	$\Delta_{rel(1)}$ (mm)	0.353	0.131	0.394	0.193
	$\tau_{avg(1)}$ (MPa)	0.041	0.015	0.045	0.022
Chhoto Sardar Bari	Floor1				
	$\Delta_{rel(1)}$ (mm)	1.773	2.612	3.31	3.769
	$\tau_{avg(1)}$ (MPa)	0.304	0.448	0.568	0.647
	Floor2				
	$\Delta_{rel(2)}$ (mm)	1.091	1.57	1.205	1.357
	$\tau_{avg(2)}$ (MPa)	0.187	0.270	0.207	0.233
Panam Nagar Bldg. 38	Floor1				
	$\Delta_{rel(1)}$ (mm)	0.765	1.292	0.626	1.179
	$\tau_{avg(1)}$ (MPa)	0.148	0.250	0.121	0.228
	Floor2				
	$\Delta_{rel(2)}$ (mm)	0.457	0.755	0.324	0.625
	$\tau_{avg(2)}$ (MPa)	0.089	0.146	0.063	0.121

Checking the *h/t* ratios (Table 2) with the prescribed design conditions, all the mosques are found to be safe due to the enormously thick walls. However, the 2-storied Panam Nagar buildings are found to be vulnerable; i.e., the scenario following bond shear failure would lead to overturning of the out-of-plane walls and possible collapse of the entire buildings

Table 2: Check for Out-of-Plane Stability

Structure	*h*(m)	*t*(m)	*h*/*t*	Allowable *h*/*t*
Chhoto Sona Mosque	7.30	1.83	3.99	< 14
Shait Gambuj Mosque	7.31	2.44	3.00	< 14
Rajbibi Mosque	4.50	2.10	2.14	< 14
Sat Gambuj Mosque	5.50	1.14	4.82	< 14
Chhoto Sardar Bari	3.69	0.23	16.04	> 9
Panam Nagar Bldg. 38	3.27	0.28	11.68	> 9

CONCLUSIONS AND DISCUSSION

This work explores numerically the seismic performance of historic masonry structures using concepts of structural dynamics. After an initial study of the lateral load carrying capacity of masonry buildings using an approximate method and verification by the structural analysis software ETABS, the concepts are used to assess the potential seismic performance of ten existing historic structures, including four mosques and two buildings of Panam Nagar, Sonargaon, subjecting them to recorded ground motions from the El Centro and Kobe earthquakes.

Lateral stiffnesses from the approximate method are within 10% of the results from ETABS, which justifies the subsequent use of the approximate method in this work.

Results from dynamic analyses show that bond stresses in the historic mosques are almost invariably less than their minimum bond strengths, indicating their possible survival under the earthquakes studied. However, the corresponding stresses for the Panam Nagar buildings are almost always found to be beyond their bond strengths, which indicate the possible bond failures at the wall interface and likely structural distress and failure.

Walls of the historic mosques are also found to satisfy the out-of-plane stability criteria, where the Panam Nagar buildings fail again, indicating their possible instability and collapse in major earthquakes.

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SPATIAL VARIATION OF PHYSICAL PROPERTIES OF AGGREGATES IN BANGLADESH

A. Rahman[1*], M. A. Noor[2] & S. K. Das[3]

[1] *Department of Civil Engineering, World University of Bangladesh, Dhaka, Bangladesh.*
[2] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
[3] *College of Engineering and Science, Victoria University, Melbourne, Australia.*
**Corresponding Author, <anisur.cebd@gmail.com>*

ABSTRACT

The properties of concrete may vary widely because of seasonal and spatial variations in properties of locally available aggregates. As the geological and geo-morphological processes of rock are different, the aggregates formed at various places are also different. The specific objectives of this study has been to observe spatial variations on physical properties of coarse and fine aggregates collected from different sources in Bangladesh. Coarse and fine aggregates have been collected from different sources of Sylhet, Moulvibazar, Mymensingh, Munshiganj, Rajbari, Kushtia, Dinajpur, Lalmonirhat and Panchagarh. According to ASTM and BS specifications, sieve analysis, specific gravity [Oven Dry (OD) and Saturated Surface Dry (SSD) conditions] and unit weight (loose and compacted) have been performed in the laboratory for both coarse and fine aggregates. According to ASTM and BS specifications, aggregate crushing value (ACV) and los angles abrasion test (LAAT) have been performed in the laboratory for coarse aggregates. The spatial variations in the properties of coarse and fine aggregates have been observed during this study for all the tests. All the test result values of coarse and fine aggregates have been compared with standard values of ACI, BS, AASHTO, IS and PWD. Coarse aggregate samples collected from Modhopara in Dinajpur, and fine aggregate samples collected from Sunamganj in Sylhet and Fulbari in Dinajpur satisfied most with the standard values of above mentioned specifications. On the basis of this study, the government and construction companies in Bangladesh can get a clear idea of aggregates from different sources to ensure the quality and strength of concrete.

Keywords: Physical properties of aggregates, Spatial variation, Standard aggregates tests, Bangladesh.

INTRODUCTION

Many desirable properties of concrete such as high compressive strength, excellent durability and fire resistance contributed toward its wide range of applicability. The most advantageous and unique feature of concrete is that it can be produced using locally available ingredients. Due to the variations in properties of locally available aggregates, the properties of concrete may vary widely. Aggregates generally exhibit two types of properties; namely physical properties and chemical properties. These properties of aggregates vary a lot according to aggregates sources. The geological and geo-morphological processes of all rocks are not same from which the aggregates are processed. Moreover temperature, humidity and rainfall vary in different seasons. For this reason, remarkable variations on the properties of aggregate are observed according to various sources. At present, high strength concrete is being used in many construction projects of Bangladesh. High strength of concrete mainly depends on aggregate properties, quality of cement, proper mix design and curing. Aggregates at different places in Bangladesh are different in size, shape, formation and gradation. Therefore, it is very difficult to determine a general concrete conception on aggregate in Bangladesh.

Where concrete of high strength and good durability is required, aggregate grading curve is very essential which has much greater effect on workability of concrete (Shetty, 2000). The relative volume of the aggregate also affects workability. Grading and maximum size of aggregate affect relative aggregate proportions as well as cement and water requirements, workability, pump ability, porosity, shrinkage and durability of concrete (ASTM STP-169-A, 1966). Very fine sands are often uneconomical, and very coarse sands and coarse aggregates can produce harsh, unworkable mixes. In general, aggregates that do not have a large deficiency or excess of any size, and give a standard grading curve produce the most satisfactory results (Houston, 1962). Thom and Brown (1988) studied the behavior of crushed limestone materials at different grading and arrived at the conclusion that the resistance to permanent deformation decreased with increasing fines content. Analysis using the materials with different gradations, it was found that different materials behaves differently in regard to the change in the grading (Land Transport New Zealand Research Report-325, 2007).

Development program of country like Bangladesh has a great emphasis on infrastructure. Construction sector is one of the prominent sectors to contribute in gross domestic product (GDP) in Bangladesh. It has a great impact in employment generation, expansion of markets for materials and other commercial activity. Aggregate is mostly used as construction material in all most every construction work in Bangladesh where aggregates are collected from different sources. Generally in the construction works of Bangladesh, spatial variation of aggregate is not taken into consideration. It may have significant effects on aggregate properties as well as overall structural stability. Limited studies on aggregate properties have been conducted in Bangladesh mainly their effects on the strength of concrete e.g. (Ahmad and Amin, 1998; Ashraf and Noor, 2011; Muhit and Haque, 2013; Sharmin et al., 2006; Zakaria and Cabrera, 1996). House Building Research Institution (HBRI) and Local Government Engineering Department (LGED) also conduct experiments on aggregate properties for particular places and particular projects. A comprehensive study on the spatial variation of aggregate properties can set a guideline for aggregates on construction works collected from different locations. Therefore, the objective of this study has been to observe spatial variation on physical properties of coarse and fine aggregates collected from different sources in Bangladesh.

MATERIALS AND METHODS

Coarse aggregates have been collected from different places in Bangladesh like Tamabil and Volaganj at Companiganj in Shylhet; Zaflong and Bisnakandi at Gowainghat in Shylhet; Sreepur at Jaintiapur in Shylhet; Vozonpur at Tentulia in Panchagarh; Modhopara at Fulbari in Dinajpur and Patgram in Lalmonirhat. Fine aggregates also have been collected from different places in Bangladesh like Bheramara in Kushtia; Gazaria in Munshiganj; Bhaluka in Mymensingh; Vozonpur at Tentulia in Panchagarh; Patgram in Lalmonirhat; Fulbari in Dinajpur; Sunamganj in Sylhet; Jaganathpahar and Sreemangal in Moulvibazar and Pangsha in Rajbari.

Aggregates collected from different sources of Bangladesh have been processed and tested in the laboratory of Bangladesh University of Engineering and Technology (BUET). According to ASTM and BS specification, the following tests have been performed in the laboratory: Sieve analysis, Specific gravity (OD and SSD), Voids and Unit weight (loose and compacted) of coarse and fine aggregates. Aggregate crushing value (ACV) and Los angles abrasion value (LAAV) tests have been performed for coarse aggregates.

RESULTS AND DISCUSSIONS

Standard specifications of the tests have been maintained during the laboratory tests. Results of those tests are compared with the standard values of ACI, BS, PWD (Public works department, Bangladesh), IS and AASHTO

Physical observations

Physical characteristics of collected sample have been identified according to different literature (Kulkarni, 1998; Neville and Brooks, 1987; Shetty, 2000). Coarse aggregates of Bisnakandi (Sylhet), Volaganj (Sylhet) and Sreepur (Sylhet) are mainly regular in shape; brown to mixed in colour but not uniform in size. Coarse aggregates of Patgram (Lalmonirhat), Modhopara (Dinajpur), Vozonpur (Panchagarh) and Zaflong (Sylhet) are almost regular in shape, uniform in size and brown to mixed in colour.

Fine aggregates of Vozonpur (Panchagarh), Sunamgang (Sylhet), Sreemangal (Moulvibazar) and Jaganathpahar (Moulvibazar) are mainly reddish in colour; free from clay and organic matters. In fine aggregates of Sunamgang (Sylhet), comparatively coarse particle is high, and others sources are comparatively finer in size. Also fine aggregates of Patgram (Lalmonirhat) are mixed bright in colour whereas fine aggregates of Bheramara (Kushtia), Pangsha (Rajbari) and Fulbari (Dinajpur) are mainly black and white in colour.

Spatial variation of properties of coarse aggregates

The test results of coarse aggregates have been shown in Fig. 1 to Fig. 2. Bisnakandi (Sylhet) has the maximum FM value of 8.93 in comparison with other sources. Samples of Modhopara (Dinajpur) have the maximum specific gravity (OD and SSD) values of 2.84 and 2.86 respectively, and the minimum absorption capacity of 0.90% in comparison with other sources. This is because of the samples of Modhopara are uniformly sized and almost in regular shape. Also, samples of Modhopara have the higher unit weight (loose and compacted) of 1459 and 1672 kg/m3 respectively than other sources whereas other source samples have almost similar values of unit weight. Moreover, samples from Modhopara show the maximum void (loose and compacted) of 48.53% and 41.03% respectively, and Vozonpur (Panchagarh) samples show the minimum voids (loose and compacted) of 45.17% and 38.61% respectively. Samples of Modhopara also have the lowest ACV(17.87) and LAAV(15.17) values compared to other sources, because of these samples having maximum unit weight, specific gravity and void, minimum absorption capacity, and uniformly size and almost in regular shape.

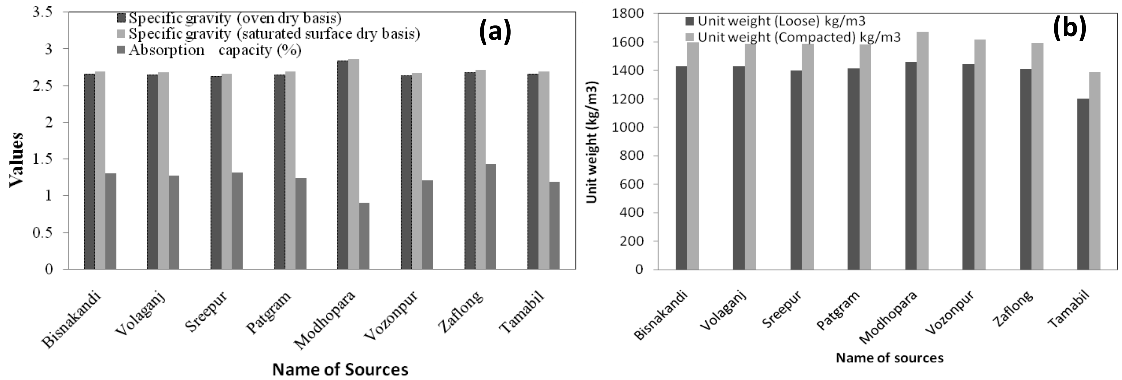

Figure 7 (a) Specific gravity and absorption capacity and (b) Unit weight of coarse aggregates

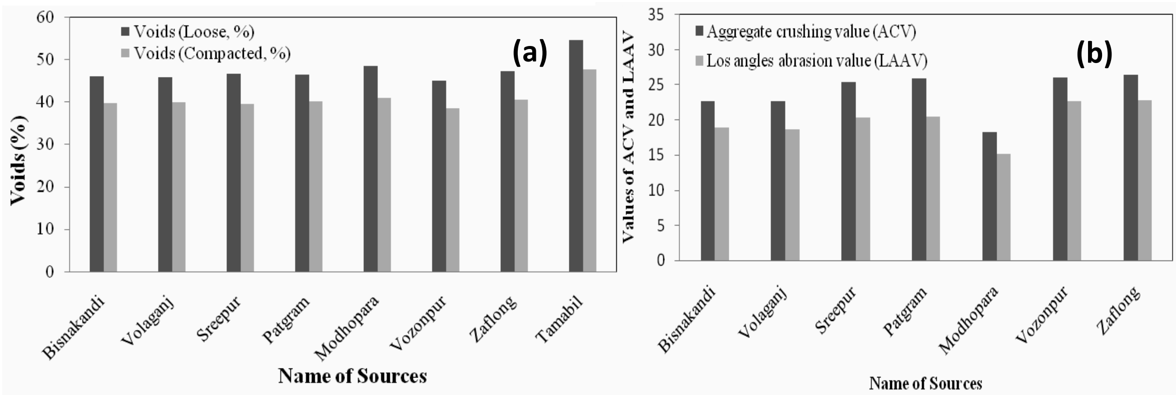

Figure 8 (a) Voids percentage and (b) ACV and LAAV of coarse aggregates

In general, properties of coarse aggregates compared well with the values of standard specifications like specific gravity values of ACI (2.3 to 2.9) and PWD (2.2 to 2.6); absorption capacity values of ACI (0.5 to 4%) and PWD (1 to 6%); unit weight values of ACI (1280 to 2250 kg/m^3) and PWD (1400 to 2250 kg/m^3); ACV values of AASHTO, BS and IS (less than 30%) and LAAV values of AASHTO, BS and IS (less than 40%). However, samples of Modhopara in Dinajpur are most satisfied with the values of standard specifications.

Spatial variation of properties of fine aggregates

The test results of fine aggregates are shown in Fig. 3 to Fig. 4. The grading curves (Fig. 3a) represent that the samples of Sunamganj (Sylhet) have larger particle size in comparison with other sources, and the samples from Sreemangal (Moulvibazar) have the finest particle size. The other samples show nearly similar distribution in their sizes. The sieve analysis of fine aggregate shows that Sunamganj has the maximum FM value of 2.67, and Sreemangal has the minimum FM value of 1.48. Also, samples of Fulbaria (Dinajpur) have the maximum values of Specific gravity (OD and SSD) of 2.77 and 2.79 respectively in comparison with other sources; these samples have comparatively high coarse particles. Moreover, Jaganathpahar (Moulvibazar) samples have the minimum (0.26%) absorption capacity, and Sunamganj samples have the maximum (1.24%) absorption capacity. Samples of Sunamganj also have higher values of unit weight (loose and compacted) of 1490 and 1653 kg/m3 respectively, and Gazaria (Munshiganj) samples have the lower values of unit weight of 1215 and 1532 kg/m3 respectively. Samples of Sunamganj have the minimum voids (loose and compacted) of 40.55% and 33.85% respectively, and Samples from Gazaria have the maximum voids (loose and compacted) of 53.46% and 41.29% respectively.

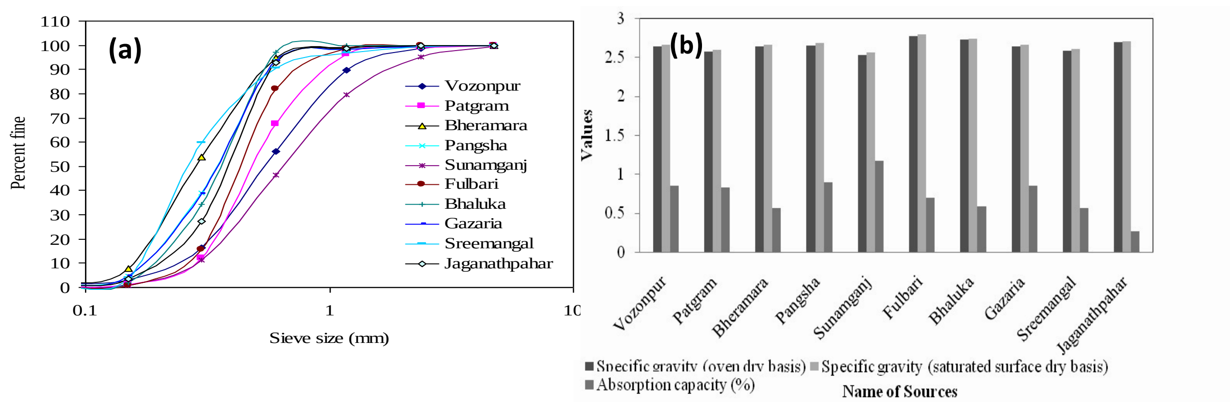

Figure 9 (a) Grading curves and (b) Specific gravity and absorption capacity of fine aggregates

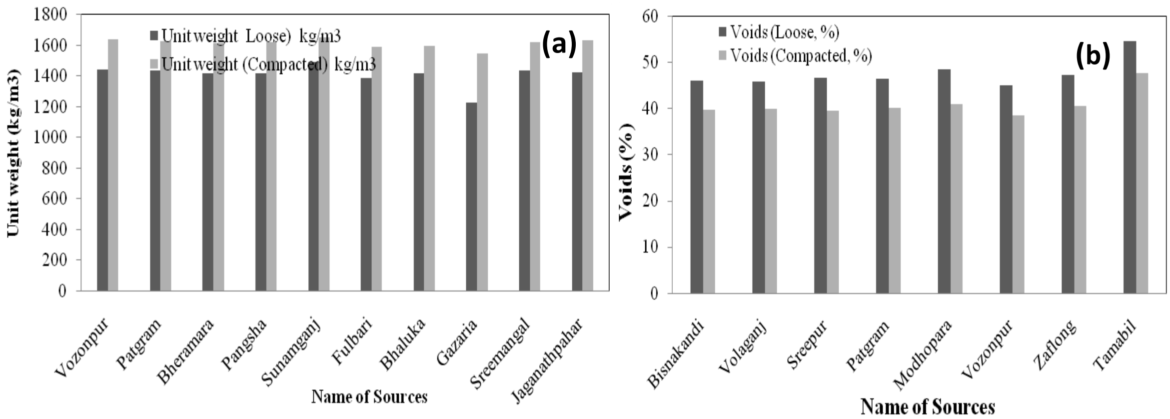

Figure 10 (a) Unit weight and (b) Voids percentage of fine aggregates

Samples of fine aggregates from Sunamganj in Sylhet are satisfied most with the values of standard specifications like FM values of ACI and PWD (for RC.C. work FM value is 2.0 to 3.0); absorption capacity values of ACI (0.5 to 4%) and PWD (1 to 6%) and unit weight (loose and compacted) values of ACI (1280 to 2250 kg/m^3) and PWD (1400 to 2250 kg/m^3) in comparison with other sources. Samples from Fulbari in Dinajpur have maximum specific gravity [standard values of ACI (2.3 to 2.9) and PWD (2.2 to 2.6)] and voids percentage in comparison with other sources.

CONCLUSION

This research work represents in depth analysis of aggregate properties from different sources in Bangladesh. The spatial variations in the properties of coarse and fine aggregates have been observed during this study for all the tests. Physical observations on the collected samples reveal that coarse aggregates of Modhopara in Dinajpur, Bisnakandi and Volagonj in Sylhet are uniform in size and almost regular in shape. Among the collected samples, fine aggregates of Sunamganj in Sylhet, Bheramara in Kushtia, Vozonpur in Panchagarh and Patgram in Lalmonirhat have comparatively high coarse particle and free from clay and organic matters.

All test results of coarse aggregates also have been compared with standard values of ACI, BS, AASHTO, IS and PWD. Coarse aggregates of Modhopara in Dinajpur are most satisfied with standard values of ACI, BS, AASHTO, IS and PWD among all the sources. Samples of fine aggregates from Sunamganj in Sylhet are satisfied with maximum values of FM, absorption capacity and unit weight (loose and compacted) in comparison with other sources. Samples from Fulbari in Dinajpur have maximum specific gravity (OD and SSD) and voids percentage in comparison with other sources. On the basis of this study, the government and construction companies in Bangladesh can get a clear idea of aggregates from different sources to ensure the quality and strength of concrete.

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EVALUATING PLASTIC SHRINKAGE OF FIBER REINFORCED CONCRETE

S. D. Gupta[1*] & G. M. S. Islam[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong – 4349, Bangladesh.*
**Corresponding Author, <payelcuet@gmail.com>*

ABSTRACT

Concrete is prone to plastic shrinkage cracking in dry and windy conditions. Plastic shrinkage cracks are typically observed in thin concrete elements with a high surface area to volume ratio (e.g. slab, wall). Remedial measures against plastic shrinkage cracking include preventing rapid drying of the surface of concrete and adopting good curing practices. The addition of non-metallic fiber such as "polypropylene" is reported to reduce plastic shrinkage crack. This study dealt with plastic shrinkage of concrete incorporating fiber. The shrinkage property of high strength concrete was evaluated by adding polypropylene fiber in various proportions (viz. 0.1%, 0.15%, 0.2%, 0.25% and 0.3%) by volume of concrete. A significant reduction in crack visualization, appearance period of first crack and crack area between plane concrete and fiber reinforced concretes were found. The experimental result indicated better performance of polypropylene fiber concrete over plane concrete against plastic shrinkage crack.

Keywords: Fiber reinforced concrete, Plastic shrinkage, Polypropylene fiber.

INTRODUCTION

Concrete is a universal building material. It is a composite material composed of water, coarse granular material (the fine and coarse aggregate or filler) embedded in a hard matrix of material (the cement or binder) that fills the space among the aggregate particles and glues those together (Nilson *et al*., 2013). Plain, unreinforced concrete is a brittle material with a low tensile strength and a low strength capacity (Nemati, 2013). Concrete made with Portland cement has certain characteristics such as; it is relatively strong in compression but weak in tension and tends to be brittle. These two weaknesses have limited its use. Another fundamental weakness of concrete is that cracks start to form as soon as concrete is placed and before it has properly hardened. These cracks are major cause of weakness in concrete particularly in large onsite applications leading to subsequent fracture and failure and general lack of durability. The weakness in tension can be overcome by the use of conventional reinforcement and to some extent by the inclusion of a sufficient volume of certain fibers (Saeed *et al*., 2006).

Three-dimensional volume changes in fresh concrete occur primarily due to rapid loss of surface bleed water on evaporation which tends to bring the neighboring solid particles closer. All this leads to shrinking of cement paste; the resultant restraint offered by aggregates leads to cracking on the surface of fresh concrete. Plastic shrinkage cracks are typically observed in thin concrete elements with a high surface area to volume ratio (Sivakumer *et al.*, 2007).The role of randomly distributes discontinuous fibers is to bridge across the cracks that develop some post cracking "ductility" (Nemati, 2013). "Polypropylene" is a synthetic hydrocarbon polymer, the fiber of which is made using extrusion processes by hot drawing the material through a die. Its use enables reliable and effective utilization of intrinsic tensile strength of the material along with significant reduction of plastic shrinkage cracking (Saeed *et al.*, 2006). In this paper, an attempt has been made to study the behavior of non-metallic fiber (polypropelene) in controlling plastic shrinkage cracks. This provides a systematic approach for quantifying the effect of fiber reinforcement on plastic shrinkage cracking.

The paper also deals with the effects of addition of various proportions of polypropylene fiber on compressive, tensile strength of concrete.

MATERIALS AND METHODS

Materials

In this experiment, Ordinary Portland cement (OPC) of strength class 52.5N was used. The percentage of clinker and gypsum in the cement was 95 - 100% and 0-5% respectively while the specific gravity OPC was found to be 3.15. Retarding superplasticizer based on polycarboxylic ether was used as an admixture. This is commercially available in liquid form and dispensed into the concrete mixing water before adding it into the mix. Properties of admixture are given in Table 1.

Table 1. Properties of admixture

Aspect	Light Brown liquid
Relative Density	1.08±0.01 at 25º C
pH	≥6
Chloride ion content	<0.2%
Expected water reduction, (%)	>20
Conforming standards	ASTM C-494, EN 934-2, IS 9103

Coarse sand was used as fine aggregate while crushed stone chips conforming to ASTM C33 was used as coarse aggregate. Both aggregates were obtained from Syhlet. Physical properties of aggregates are shown in Table 2. Sieve analysis of coarse aggregate was conducted. The cumulative passing from 25, 19, 12.5, 9.5, 4.75, 2.36 mm sieve size were maintained 100, 90, 40, 30, 10, 5 percent respectively by mixing each size coarse aggregate to achieve the ASTM C33 Grading limit for nominal 19 mm size of coarse aggregate. 'Polypropylene' fiber is a reinforcing and crack-resistant material for concrete. According to the manufacturers, the material is resistant to high temperature, corrosion. It also gives high strength, chemical stability. Properties of the fiber are given in Table 3.

Table 2. Physical properties of aggregates

Property		Sand	Stone Chips
Bulk Specific Gravity (OD Basis)		2.54	2.66
Absorption Capacity	(%)	1.34	0.69
Fineness Modulus (FM)		2.62	-
Dry Rodded Unit Weight	kg/m^3	1590	1550

Table 3. Properties of the fibers used in the experimental works

Length (mm)	6±1
Diameter (μm)	20±5
Aspect ratio (*l/d*)	300
Density (g/cm^3)	1.36~1.38
Elongation at break (%)	≥15
Tensile strength(MPa)	≥500
Color	Natural white
Materials form	Polyester
Type of the fiber	monofilament
Recommend Adding Amount	0.2-0.3% of mixes

Concrete Mix Proportions

Mix design was conducted as per American Concrete Institute (ACI, 2009). Trial mixtures were prepared to obtain target strength of 35 MPa at 28 days considering a slump value of 75–100 mm. All the aggregates brought to the saturated and surface dry (SSD) condition before mixing. The detailed concrete mix proportions of constituent materials (SSD condition where applicable) of the concretes used for the study are presented in Table 4. Cements and aggregates content kept same for all mixes while water and superplasticizer content were varied to obtain target slump value.

Table 4. Mix proportions of the concretes used in experimental work

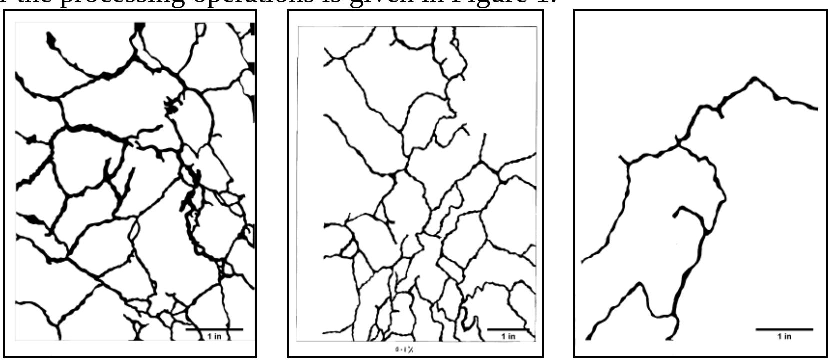
MIX	Water (kg/m^3)	Cement (kg/m^3)	Coarse aggregate (kg/m^3)	Fine aggregate (kg/m^3)	Superplasticizer (kg/m^3)	Fiber (kg/m^3)
F1(0)	150	440	1000	735	3.30	0.0
F2(0.1)	145	440	1000	735	3.96	2.30
F3(0.15)	148	440	1000	735	3.96	3.50
F4(0.2)	150	440	1000	735	3.96	4.65
F5(0.25)	154	440	1000	735	3.96	5.80
F6(0.3)	156	440	1000	735	3.96	7.00

Concrete Mixing

Concrete was mixed using a machine mixer. For every trial mixer a 40 liter volume was considered. Appropriate quantity of coarse aggregate, fine aggregates (SSD) and cement, were first dry mixed for a period of 2 minutes. The superplasticizer was then mixed thoroughly with the mixing water and added to the mixer. Fibers were dispersed by hand in the mixture to achieve a uniform distribution throughout the concrete, which was mixed for a total of 4 min. After mixing, the workability of concrete was determined using slump cone. The concrete was placed in the fabricated mould and tamping is done using a tamping rod. A smooth steel trowel was used to finish the fresh concrete.

Shrinkage Testing

Measurements of the shrinkage of specimens were carried out according to the standard ASTM C490. In this case, slab mould of dimension 20″×10″×3″ was prepared. A thin polyethylene sheet was placed over the base to eliminate base friction between the concrete and base. After mixing the concrete was into the slab mould and were stored in a specific environment. These slabs were exposed to a constant temperature of 35±1ºC, a relative humidity of 60±1%. Then the slabs were checked visually for any signs of cracking at approximately 30-min intervals and image was captured. The slabs (inside the moulds) were kept in this environment for 24 hours. Then the captured images were printed and cracks were drawn by using tracing paper. After that, the tracing papers in which crack were drawn scanned. Then the scanned images were edited with suitable color by using computer software. The images were then analyzed by computer image analysis software to quantify the percentage of crack. The crack measurements comprise of the crack width (average) and percentage of total crack area. Images of the crack were captured using an optimal zoom camera. Subsequently, the captured images were processed and edited with image analysis software to get a clear crack profile. An example of few images after the processing operations is given in Figure 1.



(a) Concrete type F1 (b) Concrete type F2 (c) Concrete type F4

Fig 1: Shrinkage crack in concrete slabs (drawn in tracing paper from printed image)

After that the edited image was analysed by image 'J' software and percentage of total crack was estimated by using this software. A picture of overall procedure is shown in Figure 2.

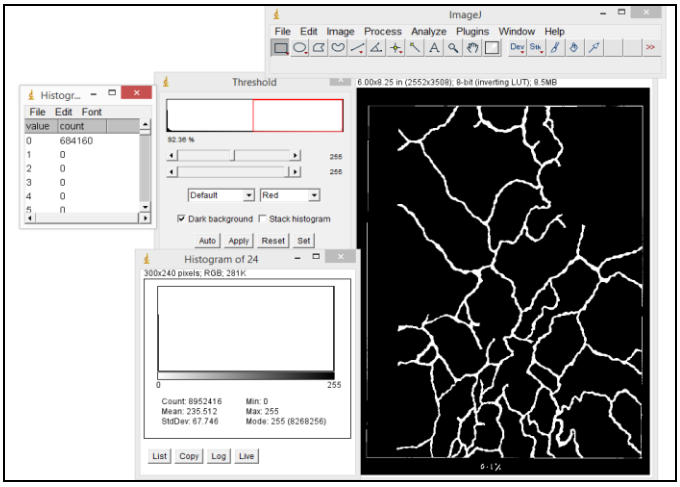


Fig 2: Image analysis for shrinkage crack (control concrete specimen)

RESULT AND DISCUSSION

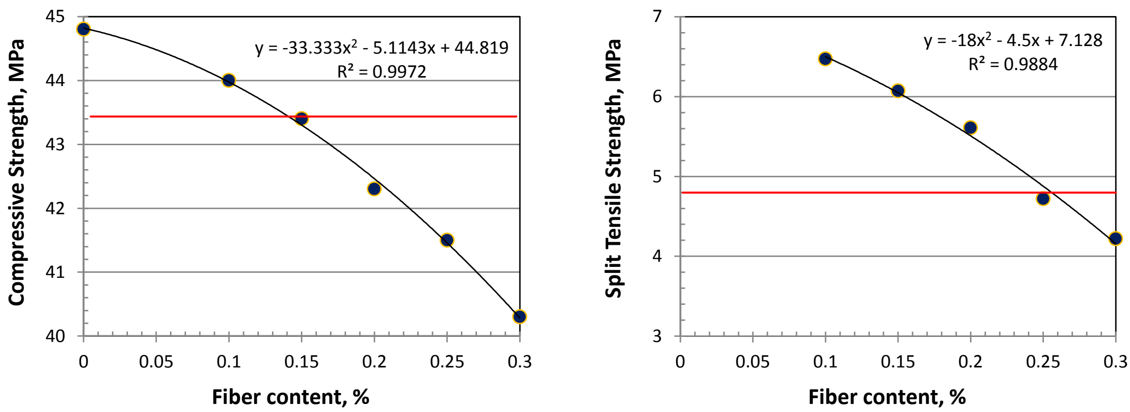
Compressive and tensile strength

Concrete compressive and tensile strength data are given in Table 5. The target mean strength for laboratory was set to 43.5 MPa. Relationship between the polypropylene fiber content and corresponding concrete compressive and split tensile strength with respect to the control concrete are shown in Fig. 3.

Table 5. Strength characteristics of fiber reinforced concretes

Type of concrete	Compressive strength (MPa)	Tensile strength (MPa)
F_1	44.8	4.80
F_2	44.0	6.47
F_3	43.4	6.07
F_4	42.3	5.61
F_5	41.5	4.72
F_6	40.3	4.22

Note: Target mean strength at laboratory is 43.5 MPa



(a) (b)

Fig 3: Relationship between fiber content and (a) compressive strength of concrete; and (b) tensile strength of concrete

It is observed that with the increasing fiber content compressive strength of concrete was decreased. This is due to fibers' interference and thus the cohesiveness of the concrete matrix affected. A maximum of 10% reduction in compressive strength (with respect to the control concrete) was noted with addition of 0.3% volume of fibers in the concrete. In contrast, the tensile strength was improved with addition of small quantity of fibers. Best result achieved for 0.1% fiber addition and approximately 35% increase in the split tensile strength was noted, while this was 17% for 0.2% fiber addition as shown in Figure 3(b). The crack-bridging capacity of the fibres mainly prevented from the splitting of concrete. With addition of higher volume of fibers though the tensile strength of concrete was decreased slightly, it was remained comparable up to fiber volume addition of 0.25% with respect to the control concrete.

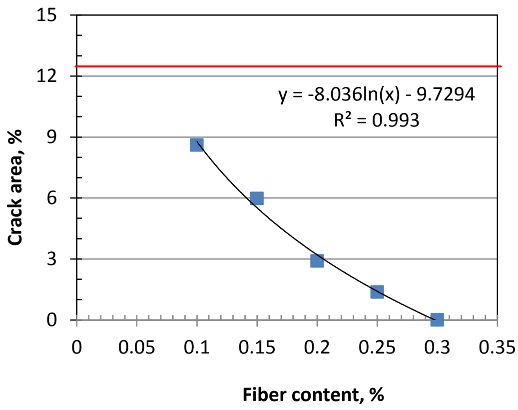
Shrinkage crack

The time of occurrence of first crack was noted for all slabs during the experimental work. For control concrete, approximately after 150 min (since water was added), a fine hairline crack was observed running throughout the width of the slab. This fine crack, which could have possibly been caused due to settlement, was found to widen upon further drying. In case of fiber reinforced concrete specimens, the appearance of the first crack took as long as more than 7 hours. The appearance period for fiber reinforced concrete was thus 3-time longer than plain concrete. Few samples of shrinkage cracked concrete are shown in Figure 4. These phenomena could be attributed to the availability of bleed water on the top surface, which delays drying of the surface. In case of F6 (0.3) concrete (Figure 4c) no crack was visible.

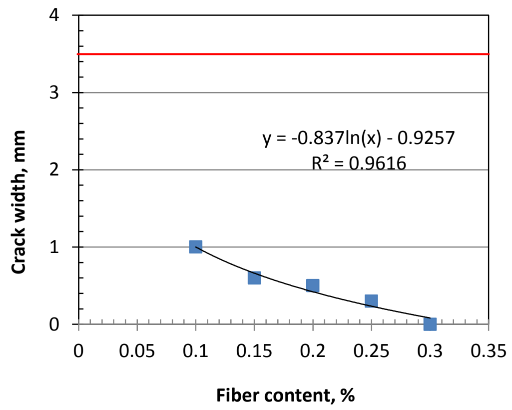
(a) Concrete type F1 (b) Concrete type F2 (c) Concrete type F6

Fig. 4: Shrinkage crack in concrete slabs (Area covered 4.5″×3.37″)

The percentage of crack and crack width decreases with addition of fiber as shown in figure 5. By using about 0.30% fibers (by volume) plastic shrinkage cracking reduced to such an extent that no cracks were observed. With the addition of 0.10 - 0.25% fibers, visibly restrained the crack width compared to control sample.



(a)



(b)

Fig 5: Relationship between (a) fiber content and percentage of crack of concrete; and (b) fiber content and crack width of concrete

The shrinkage cracking is reduced by 50 - 99% by addition of fibers up to 0.30%. The percentage of crack for 'F5' and 'F6' type concretes are approximately 1.37% and 0% respectively. According to recommendation of ACI (224.1R-07) the plastic shrinkage crack width should be within 3 mm. Though this was not satisfied for the control concrete, with the addition of fiber these criteria was also satisfied. In general fibers can act for crack bridging mechanism and therefore contribute against shrinkage cracking occurrence.

CONCLUSION

This paper reported experimental results on plastic shrinkage crack of concrete incorporating 'polypropelene' fibers. The following conclusions can be drawn from the above experimental results.

- With the addition of polypropylene fibers, the compressive strength decreased from by 2 - 10%.
- The optimum fiber content for the compressive strength is 0.1% for which reduction of compressive strength of this content is about 2%.
- The tensile strength increases about 17%~35% up to 0.20% fiber addition after which it decreases.
- The optimum fiber content for the tensile strength is 0.1% for which increase of tensile strength of this content is about 35%.
- Plastic shrinkage cracks were reduced by 50–99% compared to the control concrete by addition of fibers.
- With an increase in the non-metallic fiber (polypropelene) content, the crack width was also reduced significantly and remained within the limit specified by the ACI (224.1R-07).

ACKNOWLEDGEMENT

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ID: SEE 013

TRANSFLOOR SLAB SYSTEM- A CONVENIENT APPROACH OF RC CONSTRUCTION

O. C. Debanath[1*], G. M. S. Islam[1] & M. S. Islam[2]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.*
[2] *XL Precast, Sydney, 2 Pine Road Yennora, NSW 216, Australia.*
** Corresponding Author, <debnathopu60@gmail.com>*

ABSTRACT

Slab is a key structural element for most of the Civil Engineering construction projects. The transfloor system is one of the newly developed slab construction technique. In its final form, the slab system is very much similar to a conventionally reinforced slab beam and column concrete structure. The main difference lies in the methodology of constructing a reinforced concrete building. The system takes advantage by using half-slab precast panels (transfloor slab panels), in conjunction with half-beam precast modules arranged onto in-situ vertical elements, such as columns and shear walls. Moreover, the lattice girder truss ensures a practical bond between the precast and in-situ concrete. For convenience, the suggested construction and reinforcement details of this slab system is different from conventional slab. It offers convenience considering the large space to be constructed with respect to the minimized extensive use of formworks. The cost and time savings are very apparent with this system. A direct equivalent of an in-situ slab, transfloor provides the facility to combine precast and in-situ concrete and offers major benefits to Designers, Engineers and Builders.

Keywords: Slab, Transfloor, Precast, Formwork.

INTRODUCTION

The structural frame system is one of the most important decisions for multistoried building project. The choice of construction material and system is an important issue for the builders, consultant, owner and engineers. Reinforced Cement Concrete (RCC) structural frame is mainly used for multistoried building construction in Bangladesh. This requires longer construction period and huge manpower. Generally, slab and beam is cast integrally, which makes complex to maintain the proper design features. Especially for slab, with traditional formworks, maintaining proper level is not easy.

The transfloor slab system is a newly developed composite flooring system with permanent precast formwork (CCAA T49, 2003). The system, known by several names, depending on the manufacturer, incorporates precast concrete slabs, usually 55-mm thick (bottom part), with embedded reinforcement and trusses. The irregular ends of precast panel can easily be prepared. To complete the floor, an in-situ concrete topping acts compositely with the precast bottom panels. The bottom reinforcement embedded in the precast panel consist of a layer of mesh, the bottom chords of the trusses and additional reinforcing bars is required to select by the designer. The embedded trusses also provide strength and stiffness for handling and transport, allow panels to support construction loads with a minimum of temporary propping, contribute to the top and bottom reinforcement, and act as bar chairs to support the top reinforcement (CCAA T49, 2003). After the formation of proper bonding the precast and cast in-situ portion acts as a composite unit. Moreover, the surface finishing is not required for this system. The length, width, thickness, plan geometry and reinforcement can be varied to suit the design requirements.This paper focuses on the comparison between the transfloor slab and conventional slab with respect to economy, design flexibility and other features for an identical building plan.

METHODS

In this study, design of a 4-storied building was considered to compare between conventional beam-column-slab and column-transfloor system. The building under consideration is designed for residential purpose. The typical floor plan of the building with precast transfloor panel alignment is shown in the Fig. 1. In both system of analysis, staircase, partition wall and other fabrications are considered as same and were not considered for cost estimation, but their corresponding loading effect is considered in design.

Design considerations

The RCC frame and the structural elements were analyzed and designed for the residential occupancy live load prescribed in Bangladesh National Building Code (BNBC, 2006). Frames analysis was carried out using moment distribution method. The other design details are given in Table 1. The conventional RC slab is designed following ACI coefficient method where, the slab depth is governed by deflection control criteria and the reinforcement is chosen form flexural requirements, The load combination is 1.2 × Dead load + 1.6 × Live load as prescribed in the ACI-318 (2011). The shear reinforcement requirement is checked and generally found that concrete section's shear capacity is sufficiently above the ultimate shear.

Table 1: Design details of conventional and transfloor slab system

Particulars	**RC Slab System**	**Transfloor Slab System**
Slab	Coefficient method	Australian Standard (AS3600)
No of stories	4	4
Total floor area	238 m^2	238 m^2
Total slab depth	125 mm	190 mm
Live load	2 kN/m^2	2 kN/m^2
Dead load	3.95 kN/m^2	(3.9+0.5) kN/m^2
Foundation	Suitable	Suitable
Walls and detailing	N/C	N/C
Staircase and elevator	N/C	N/C

The transfloor slab section is designed in accordance with AS3600 (2014) Clause 8.4. The load combination used for this system is 1.252×Dead load + 1.62 × Live load, the dead load includes 0.5 kN/m^2 superimposed load. Accepted principles of Ultimate Strength Theory apply to the design of Transfloor since the finished slab can be considered as monolithic. The only restriction of precast panel is transport limitations that generally limit maximum width to 2.5 m and maximum length to approximately 12 m. Special lifting frames are required for units over 8 m long (CCAA T49, 2003). Five trusses is the practical minimum for 2.5 m wide panel. Irregular shape of precast panel can also be easily prepared. Corresponding to slab depth the other necessary property of section such as thickness of polystyrene void formers, top and bottom reinforcement, truss height etc. is shown in Fig. 3. The typical size of polystyrene void formers is 1000 × 550 mm and thickness can vary as required. The spacing of void formers is taken generally 200 mm with a 65 mm minimum clear cover.

Voids must be avoided in regions of high shear (at supports and point loads) and will generally not be included within one slab depth from the section at which the ribs are just sufficient to resist the applied shear. However, the shear through the void section is calculated by the formula (1.15wl)/2. The void percentage is variable from 0-35%. Clear cover of reinforcing bar is usually 20 mm. The bottom reinforcement is embedded in precast panel and the top reinforcement is fixed on site using continuous reinforcing bar. As with conventional floors, attention must be given to anchoring of steel reinforcement at the supports for transfloor design. In general, it is sufficient to anchor 50% of the total positive moment steel reinforcement required at mid span.

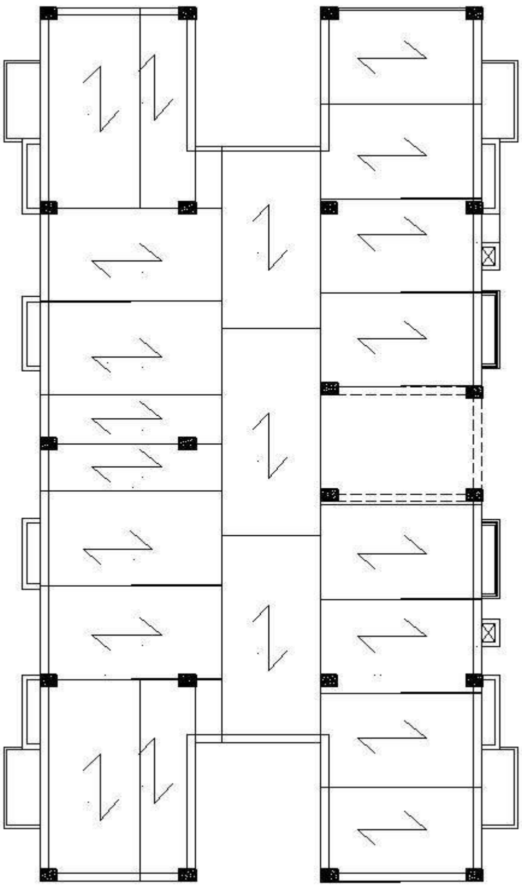
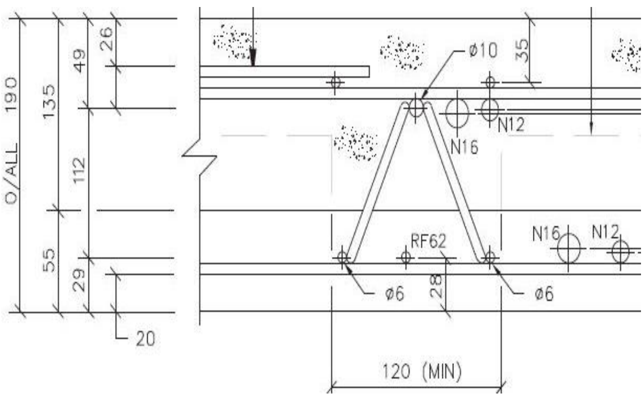


Fig.2 Site installation of precast panel



lignment

Fig.3 Cross sectional details of transfloor slab

The expanded polystyrene (EPS) void formers are made of a light weight cellular plastic material comprising 98% air, which reduces the self-weight of the slab and provides cost savings in foundations, columns and beams. The void formers also reduce the volume of in-situ concrete. The another performance influencing factor is Typical Cycle of RC slab construction, which means the time required to complete per unit area of slab. One of the major advantages is the typical cycle is 0.62 hr/m^2 for traditional formwork, which is only 0.29 hr/m^2 for transfloor system. Therefore, total building construction period can be reduced significantly. Traditional formwork can be totally eliminated and panels provide both the working platform and part of the completed floor. Typically up to 150 m^2 per hour can be placed by crane. A class 2 off- form grey finish is easily achieved which means that will not require further treatment and suitable for painting with minimum preparation. Small penetrations for electric wiring and plumbing could cut on-site. According to Sika (2003) transfloor slab system offers lower shrinkage compared to conventional slab.

RESULTS & DISCUSSION

Cost and Quantity

After analysis, design and estimation the results for conventional and transfloor slab were compared quantitatively in Table 2. For cost estimation Bangladesh market price of different items are considered. Propping requirements are reduced when compared with traditional formwork which means less cluttering of the floor below and earlier access by following trades. Fewer trades are required resulting in a less cluttered, cleaner and safer building site as well as an immediate work platform is provided. The fire resistance of transfloor is satisfactory when top concrete cover is minimum 65 mm, however if additional safety is required the designer has the flexibility to increase the covering.

Table 2: Analysis results of RC slab system and transfloor slab system

Elements	No.	Description	Unit	RC SLAB SYSTEM			TRANSFLOOR SLAB SYSTEM		
				Quantity	Rate (Tk.)	Amount (Tk.)	Quantity	Rate (Tk.)	Amount (Tk.)
Slab	1	Precast Concrete(30MPa)	m^3	0	0	0	52.5	13800	724500
	2	In- situ Concrete(30MPa)	m^3	122	14100	1720200	90	14100	1269000
	3	Steel for Slab(420Mpa)	Tons	4.4	60,000	264000	8.2	60,000	492000
	4	EPS	kg	0	0	0	488	1200	585600
Beam	5	Concrete(28Mpa)	m^3	75	13700	1027500	0	0	0
	6	Steel(420Mpa)	Tons	7	60,000	420000	0	0	0
Column	7	Concrete(28Mpa)	m^3	40	13700	548000	31	13700	424700
	8	Steel(420Mpa)	Tons	2.3	60,000	138000	1.25	60,000	75000
Foundation	9	Concrete(20Mpa)	m^3	45	13000	585000	26	13000	338000
	10	Steel(420Mpa)	Tons	1.5	60,000	90000	0.7	60000	42000
	11	Earth cutting	m^3	180	80	14400	100	80	8000
	12	Earth filling	m^3	130	93	12090	74	93	6882
	13	Bricks	Nos.	4200	8	33600	2500	8	20000
Special cost	14	Scaffolding cost (steel Supported)	$/m^2$	0	0	0	952	350	333200
	15	Crane cost	/hr.	0	0	0	7	8000	56000
	16	Welding	$/m^2$			0	952	100	95200
Total overall cost						4852790			4470082
Savings using transfloor slab						Tk.	**382708**		

% Savings using transfloor slab	**7.89%**

For an identical load arrangement, the transfloor slab depth is greater than conventional slab, therefore, the deflection control and shear resistance is superior in case of transfloor (Glynn, 1981). The deflection under construction load should not exceed 2 mm. Micro cracks may develop during lifting, which is eliminate after formation of composite unit. But there are some special arrangement is required for transfloor panel than RC slab. The precast panel is handled using crane, for which extra cost and experienced manpower is required. The suitability of transfloor slab depends on availability of polystyrene void formers. Lattice girder truss provides proper bonding between top and bottom slab elements, for which the rigidity of transfloor slab is higher. The transfloor slab is cost effective than any other precast slab like flat slab, grid slab, hollow card slab.

The cost comparison for whole building and different structural elements are compared in Figure 4. It is observed that the total cost of construction can be reduced by approximately 7.9% using transfloor slab system. Although the cost of slab installation is higher in transfloor system the beam can be completely omitted. As a result considering overall cost of beam-slab system, transfloor slab system could save up to 10.5% cost than conventional slab system. As the self-weight of beam is omitted the ultimate load on column is also reduced. In some cases the number of columns can also be reduced which could save around 25% cost. The cost and even type of foundation can also be benefited using transfloor slab system although the foundation depends on soil condition. Due to reduction of column load the foundation cost reduced dramatically to 43% considering isolated footing condition. However, some special cost such as transportation of precast, crane hiring is added in case of transfloor slab system.

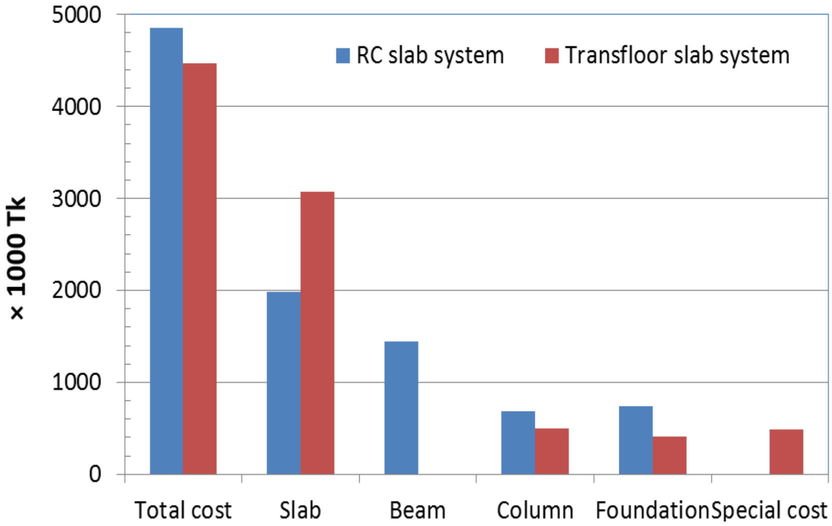


Fig. 4 Cost comparison for buildings with conventional and transfloor slab system

In high seismic zone designer may need some additional measurement, such as edge beam could be added. The transfloor, acting monolithically, will adequately transmit lateral loads through diaphragm action. The strength and ductility of the overall structural system will depend on the integrity of the joint detailing and in particular, the connections between the floor (horizontal diaphragm) and the supporting structure. The flexibility of thin cast in-situ topping slab that forms the horizontal diaphragm causing overstressing and cracking could result in separation from the precast elements. A common form of construction for medium rise residential buildings is to use precast concrete panels or frames for the vertical elements and precast concrete floor planks without the addition of a topping slab. These precast systems performed poorly due mainly to the inadequate provision of viable load paths through inadequate tying of the horizontal floor planks to the vertical elements and to each other for effective diaphragm action.

CONCLUSION

Transfloor slab is one of most popular precast slab system used worldwide at present time. This comparison is based on structural components only. The price of materials and other necessaries may vary from place to place. The transfloor system requires some other additional cost but overall the construction cost is lower in comparison with conventional slab system. From above discussion it is clear that transfloor slab is a safe, time saving as well as money saving (approximately up to 7.9%) approach. But the suitability of transfloor slab system is depended on the availability of additional materials (such as EPS) and instrument (for example precast plant, crane and transportation truck). For developing country such as Bangladesh, this system is not so familiar, but if the proper supply of additional materials and instruments can be ensured it could be cost effective and popular in practical construction.

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DYNAMIC RESPONSE OF CURVED-SKEW MULTI-GIRDER BRIDGE UNDER MOVING VEHICLES

Md Basir Zisan[1*], Toshiro Hayashikawa[1], Tetsuji Ohta[1], Takashi Matsumoto[1]
Xingwen He[1]

[1]*Faculty of Engineering, Hokkaido University, Sapporo, Japan <basirzisan@gmail.com>*
**Corresponding Author <basirzisan@gmail.com>*

ABSTRACT

Distortion-induced fatigue on steel-girder composite bridge resulting from out-planes web bending caused by differential deflection of adjacent girders. Bridge curvature and vehicle eccentricity are important parameters significantly effects deflection behavior because of low torsional stiffness of the unstiffened curved I-girder and causes of severe fatigue problems. Current design practice gives recommendation to mitigate the effect of distortional fatigue damage, but does not directly address the secondary, out-plane bending forces or provides any guidance in determining the magnitude of out-plane bending stress. The objective of this paper is to evaluate the effect of skew angle and transverse loading position on distortion-stress and dynamic behavior of the curved - skew bridge using a three-dimensional finite-element analysis. For numerical study, a 3-D Bridge-Vehicle-Interaction (BVI) model developed for curved-skew bridge using the ANSYS FE program for both parallel and right angle diaphragm arrangement. A 3-D FE model of AAHSTO HS20-44 truck modeled using mass-spring -damper system. This analysis considered the inertia force, centrifugal force, deck friction, road roughness and approach road with practical aspects. ANSYS contact technique used to integrate the interaction of vehicle tire and road surface. The analysis results show that eccentric vehicle position and skew angle caused a higher differential deflection, distortion-stress and bearing force. In addition, distortion-stress is not proportional to differential deflection for vehicle eccentricity. The effect of skew angle is higher near the bridge end location rather than mid-span location.

Keywords: BVI, Curved-skew bridge, Vehicle eccentricity, Distortion-stress, Dynamic behaviors.

INTRODUCTION

Fatigue is one of the main forms of structural damage in steel-composite bridge caused by distortion-stress. In multi-girder bridge, girder differential deflection induces out-plane deformation of web gap where the stiffener is terminated. Such distortion, especially severe where the top flanges of the girders are fully restrained by stiff concrete slab and bottom flange are free to move out of plane (Fisher *et al.* 1990). This out-of-plane distortion at the toe of the web stiffener leads to stresses several times that of flange stresses (Jajich, 2003) and considered as the largest source of fatigue cracking in steel bridge (Keating, 1994; Cornor,2006). Zhao (2007) and Fisher *et al.* (1979) reported distortion-induced stresses at top flange web are much higher than those at the bottom flange web-gap. Hassel *et al.* (2013) investigated the distortion-induced fatigue for different cross-frame layout and skew angle using ABAQUS through the static analysis. Several researchers mentioned the span length, girder spacing, slab thickness and girder stiffness, skew angle (Berglund, 2006; Lenwari, 2012) affecting differential deflection as well as distortion stress. However, eccentric vehicle position or bridge curvature that causes torsional vibrations and significantly increase the differential deflection between interior and exterior girder not studied yet. In addition, most of the above-mentioned researches use experimental data to explain the distortion-stress. A little research found in literature that considered the bridge-vehicle -interaction and effect of a moving vehicle on distortion-stress, but no previous research found that directly investigated the dynamic behavior and distortion-stress on the curved-skew bridge under moving vehicle.

The purpose of this research to develop a 3-D bridge-vehicle-interaction (BVI) model to evaluate the dynamic behavior and distortion-stress in curved-skew bridge under moving vehicles. For this purpose, a versatile and computationally efficient BVI model has proposed using spaced bridge and vehicle modeled by ANSYS program. This analysis considered the approach road, inertia force, centrifugal force, deck friction, surface to node contact technology and vehicle model includes the effect of pitching, rolling, bouncing and separation between tires and bridge surface. The parameters influencing distortion-stress and dynamic behavior considered are transverse loading position and skew angle. The dynamic behaviors are evaluated in terms of differential deflection and bearing force.

2 FINITE ELEMENT MODELLING

2.1 Bridge Modelling

Table 1 and Fig. 1 describe the basic geometric properties of the bridge. A 3-D FE model of the bridge has developed using ANSYS using solid45 and shell63 element for both parallel and right angle diaphragm as shown in Fig. 2a and Fig 2b. For each model, the skew angle varies from 00 to 450 having 150 intervals. The web-gap assumed to100 mm and Fig. 1c and Fig. 2c describe the web-gap location. Fig. 3a and Fig. 3b describe the locations for response measurement. In addition, a number from 1 to 4 marks the diaphragms; 1st and 2nd end bearings are marked by G1 to G5 with corresponding girder identification. The girder's mid-span locations are independent of skew angle and bridge curvature for all cases. Cylindrical coordinate system having origin at center of curvature has used to define all geometric properties for all FE models. Simply supported boundary conditions used at the bottom flange node at each girder end. The moduli of elasticity and mass density for structural steel and concrete are taken as 210 GPa & 7850 kg/m3 and 28.57 GPa & 2500 kg/m3, respectively. The Poisson ratios for steel and concrete material are assumed 0.3 and 0.2, respectively.

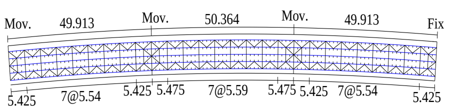
2.2 Vehicle Modeling

A 3-D finite element model of the HS20-44 truck as shown in Fig. 4a is developed using ANSYS, which consists of five lump-masses with rotary inertia representing tractor, semi-trailer and three axle sets as shown in Fig. 4b. All masses connected with rigid beam and supported by linear spring-damper formed the vehicle body. The rigid beams, masses and spring damper were modeled by BEAM4, MASS21 and COMBIN40 element. The separation between tire and road surface is integrated using gap element at the lower spring-damper. The tire stiffness and spring suspension values are found from Wang et. al. (1992) as shown in Table 2. To simulate the effect of road surface roughness, an actuator modeled by LINK11 element has connected with gap element. The suspension force consists of linear elastic spring force and constant interleaf friction force. The tire spring and all dampers are assumed linear. Fig. 5 indicates the transverse position of vehicles on the bridge. Total eight loading positions are considered where positions L01-L07 termed as outer lane and L09-L15 as inner lane vehicle. The inner and outer lane vehicle positions are equidistant from center girder are symmetrical. To obtain the initial condition of the vehicle, it is subjected to run an approach road of 45 m length, having roughness condition same as that of bridge deck before entering the bridge. Bridge Vehicle Interaction (BVI) is modeled by ANSYS node to surface contact technology. ANSYS CONTA174 and TARGE170 element are used to generate node to contact surface and target surface respectively. CONTA174 element supports large sliding, large deformation, Coulomb friction and provides better contact result (ANSYS, 2012). In this

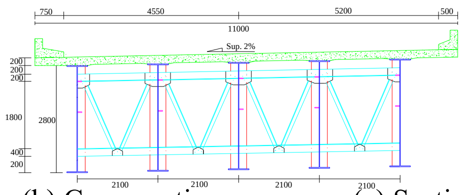
Table 1 Basic geometric property of Kita-go bridge

Span length [mm]	49913			
Deck width*thickness [mm]	11000*20			
Web of main girder [mm]	WEB 2800*10			
Flange of main girder [mm]	FLGG1 540*25	FLGG2 350*16	FLGG 3&4 370*14	FLGG5 510*25
Vertical Stiffener of main girder [mm]	145*12			
Horizontal Stiffener of main girder [mm]	115*11			
Flange and Web of Intermediate Diaphragm [mm]	IFLG 100*8 and IWEB 200*8			
Vertical Stiffener of Intermediate Diaphragm [mm]	145*12			

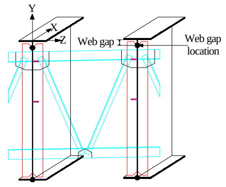
Flange and Web of End Diaphragm [mm]	EFLG 250*10 and EWEB 2400*9
Vertical Stiffener of End Diaphragm [mm]	145*22
Flange and Web of Lateral Bracing	LWEB 150*10 and LFLG 150*12
Web gap, g [mm]	100
Skew angle	0^0, 15^0, 30^0 and 45^0
Radius [m]	1000



(a) Pan view

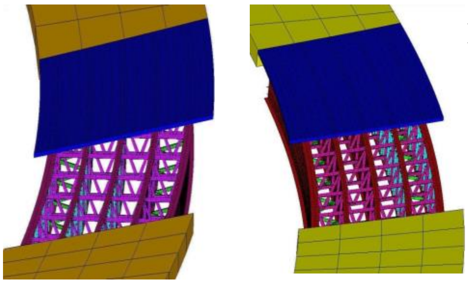


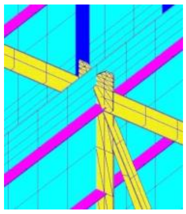
(b) Cross section



(c) Section and web gap location

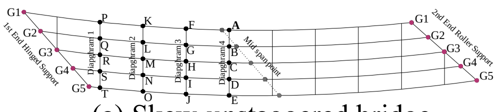
Fig. 1 Geometric properties of Kita-go bridge



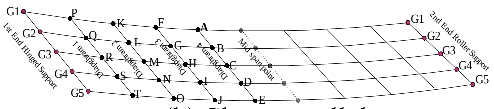


(a) Skew-unstaggered (b) Skew-parallel bridge bridge (c) Web-gap

Fig. 2 FE model of curved-skew bridge

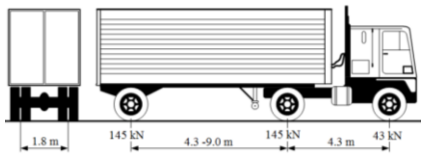


(a) Skew-unstaggered bridge

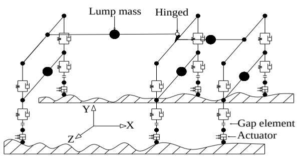


(b) Skew-parallel

Fig. 3 Web-gap location for response calculation



(a) AASHTO HS20-44 truck



(b) FE model of HS20-44 truck

Fig. 4 AASHTO HS20-44 design vehicle

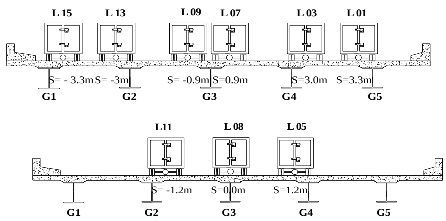


Fig. 5 Trsnaverse truck loading position

Table 2 Stiffness coefficient of HS20-44 Design truck (Wang *et al.* 1992)

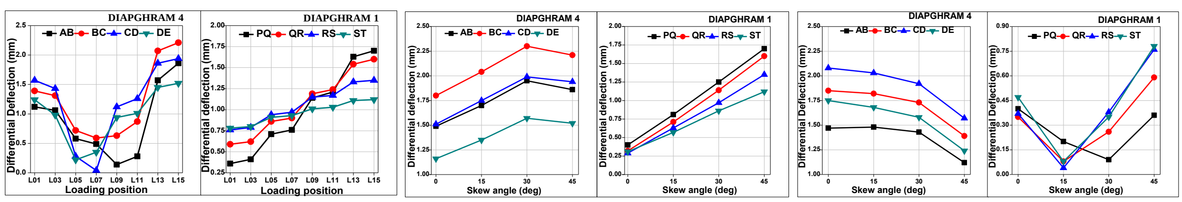
	Front axle (kN/cm)	Drive axle (kN/cm)	Semi-trailer axle (kN/cm)
Tire	8.75	35.03	35.07
Suspension spring	2.43	19.03	16.69

study, the isotropic coulomb friction of value 0.18 is assumed in all cases as used by Samman et al. (2007). This contact analysis dramatically adds non-linearity of the analysis. Newmark's β and Newton-Raphson methods with full transient analysis option used to calculate the structural response at each discrete time step.

3 NUMERICAL RESULTS

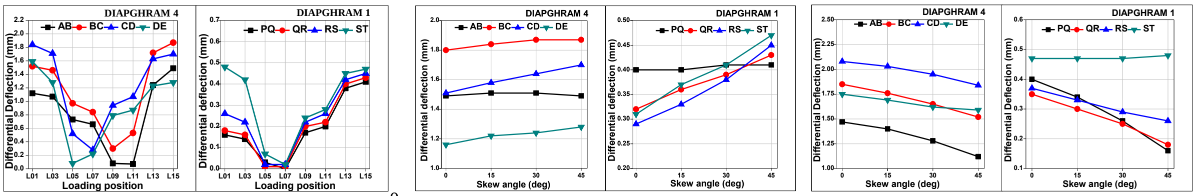
3.1 Differential Deflection

Fig. 6 and Fig. 7 describe the absolute value of differential deflection for skew-unstaggered and parallel bridge, respectively for vehicle positions and skew angles. Fig. 6a and Fig. 7a show, the highest value of differential deflection found for diaphragm 4 and lowest for diaphragm 1. Both inside (L15) and outside (L01) vehicle positions are critical and for skew-unstaggered bridge, differential deflection



(a) For loading position and 45^0 (b) For inside loading (L15) (c) For outside loading (L01)

Fig. 6 Loading position and skew angle effect on differential deflection for skew-unstaggered bridge



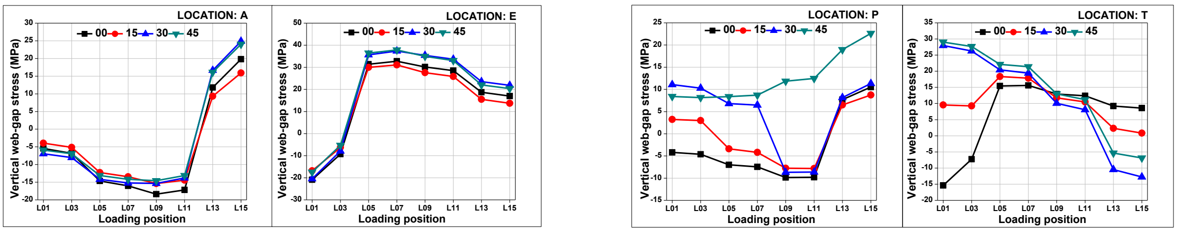
(a) For loading position and 45^0 (b) For inside loading (L15) (c) For outside loading (L01)

Fig. 7 Loading position and skew angle effect on differential deflection of skew-parallel bridge

increase as the vehicle moves from outside to inside, especially for diaphragm 1. The vehicle eccentricity causes high torsional rotation of girder as well as higher differential deflection at diaphragm 4. In addition, because of skewness, the diaphragm locations on outside girder comes close to bridge end and load of outside vehicle easily transfers to bridge supports through the combination of deck and girder, which causes less deflection compare with inside vehicle position. However, the skew-parallel bridge has quite similar value for diaphragm 4 and shows different behavior for diaphragm 1 mainly for the diaphragm arrangement. Again, it found under each loading condition the maximum differential deflection does not occur between a vehicle-supporting girder and its nearest girder. The maximum value found between nearest two unloaded girders. For example, under the loading position L01, point E lays on supporting girder G5. It found that differential deflection between girder G5 and G4 indicated by the DE is minimum. The maximum deflection found between nearest two unloaded girder G4, G3 indicated by CD. The reason is that, the girder spacing 2.1 m is very close to the vehicle wheel spacing 1.8 m. Hence, the supporting girder and its adjacent unloaded girder undergo similar and large vertical deflection. However, for stiffness of bridge deck, the remaining girders tend to stay in their initial position. As a result, the differential deflections between nearest two unloaded girders show maximum. It found differential deflection increase with the increase of the skew angle under inside loading (L15) and show reverse for outside loading (L01) as shown in Fig. 6b, Fig. 6c and Fig. 7b, Fig. 7c, respectively. In addition, this effect is higher for skew-unstaggered bridge in comparison with parallel diaphragm and significant for diaphragm 1. Increasing of the skew angle means shortening the normal distance between obtuse corner and outside diaphragm location closing to the bridge supports. Hence, a major portion of vehicle load is transferred to the bridge end along the shortest direction through bridge deck and thus reducing the differential deflection under outside loading (L01). Again, skew effect on diaphragm 1 is highly distinct from others. With an increase of skew angle, diaphragm 1 and bridge support line forms triangular geometry whose acute angle coincides with obtuse corner of the bridge. Hence, most of the load transferred to obtuse corner support through triangular bridge deck. Hence, loading L15 and L01 yields maximum differential deflection for girder G1, G2 and G4, G5 defined by PQ and ST respectively, and proportionally decreases for other girders. The same trend found in parallel diaphragm also. Again differential deflection at 30^0 and 45^0, diaphragm 1 of skew-unstaggered bridge increase under outside loading L15 as shown in Fig. 6c. For small curvature, deflection of point T has been always higher than S. However, when the skew angle became 30^0 and more, deflection of S becomes higher than T.

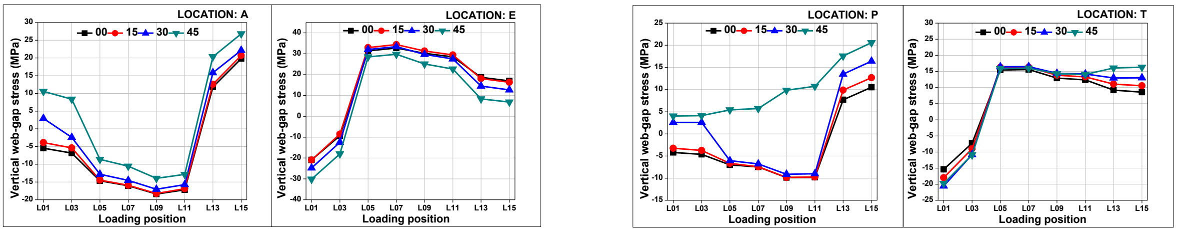
3.2 Distortion-stresses

Maximum stresses, whether it is tension or compression picked up under each loading position for every location. Fig. 8 and Fig. 9 describe the distortion-stress for location A and E for both type bridge. The loading causes alternate stress condition for same location, which is highly susceptible to fatigue problems. This alternate stress causes higher stress range, especially for outside girder and obtuse corner location. For example, location E and T as shown in Fig. 8a and Fig. 8b has stress range from -22 to 38 MPa and -10 to 30 MPa which is about 60 MPa and 40 MPa, respectively. Both types of bridge show



(a) Vertical web-gap stress at diaphragm 4 (b) Vertical web-gap stress at diaphragm 1

Fig. 8 Loading position and skew angle effect on web-gap stress of skew-unstaggered bridge



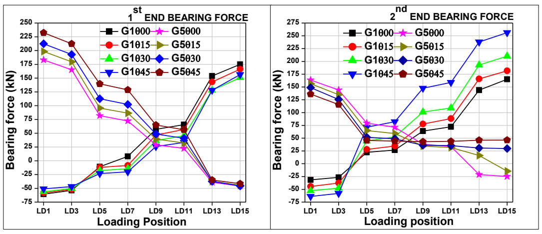
(a) Vertical web-gap stress at diaphragm 4 (b) Vertical web-gap stress at diaphragm 1

Fig. 9 Loading position and skew angle effect on web-gap stress of skew- parallel bridge

similar behavior for location A, E and highly fluctuating for P, T in case of skew-unstaggered bridge. Again, distortion-stress is not only a function of differential deflection. For a curved bridge subjected to eccentric loading, the distortion-stress influenced by lateral torsion or torsional buckling. From the previous section, the maximum differential deflection for location A, E obtained for outside (L01) and inside (L15) loading. However, the maximum distortion-stress as shown in Fig. 8a and Fig. 9a are not correspondence to maximum differential deflection at all. The effect of the skew angle is remarkable for right angle diaphragm rather than parallel, especially for a diaphragm location near to bridge end. Increasing of skew angle also increase the web-gap stress. It found that the maximum stress obtained for skew angle 30^0-45^0. Again the effect of the skew angle on the web-gap stress at diaphragm 1 near to support end is highly distinguishable compare to others location. The maximum web-gap stress obtained for skew angle 45^0, which decreases with lowering the skew angle. However, web-gap stress for diaphragm 1 in case of parallel diaphragm shows a small change with skew angle compared with the right angle diaphragm arrangement.

3.3 Bearing Force

Fig. 10 and Fig. 11 presents both ends bearing force for loading conditions and skew angles for both type bridge. It found that bearing force for support at the obtuse corner (1st end, G5 and 2nd end, G1) increase and for acute corner (1st end, G1 and 2nd end, G5) decrease with an increase of the skew angle for both type bridges. Since the shortest distance between opposite two obtuse end decreases with the increase of skew angle, most of the wheel load transfer along the shortest diagonal. Hence, bearing force for support near to obtuse corner increase and for acute corner decrease with an increase of skew angle. It found in skew angle 45^0, bearing force for 1st end girder of G5 increase from 183 kN to 232 kN which is about 27% and for 2nd end girder G1 is 55% from 165 kN to 256 kN as described in Fig. 10 For skew-parallel bridge this amount is 20% and 38%, respectively, and magnitude of bearing force also smaller than skew-unstaggered bridge as shown in Fig. 11. Uplift bearing force on remotest girder generates due to the eccentricity of the vehicle from the centroidal axis of the bridge. The maximum change of uplift force for skew angle in girder G1 is -32 kN to -65 kN (102%) and for girder G5 is -46 kN to -41 kN (12%) for skew-unstaggered bridge whose are -29 to -56 (93%) and -43 to -33 (30%) for



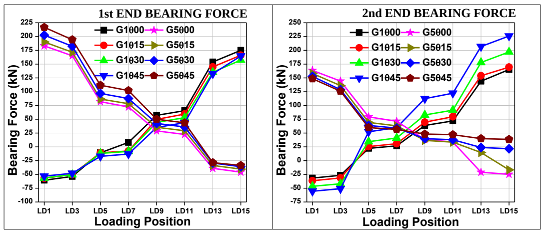


Fig. 10 Bearing force for skew-unstaggered bridge   Fig. 11 Bearing force for skew-parallel bridge

parallel bridge. Inside girder has both higher magnitude and change of bearing force with skew angle for curvature effect. This higher uplift force in higher skew bridge may cause of girder uplifting under heavy traffic condition. So extra care needs to be taken in bearing design of skew bridge.

4 CONCLUSION

The effect of skew angle and loading position to distortion-induced stress and dynamic behavior of curved skew bridge has evaluated by performing a 3-D BVI analysis using ANSYS program. For computer simulation, a 3-D model of the entire system has developed considering road roughness, approach road, deck friction, inertia and centrifugal forces. The following conclusions were drawn by analyzing the result of computer simulations:

Differential deflection is critical for both inside and outside vehicle position and inside position is significant for skew-unstaggered bridge. Bending and bridge torsional rotation caused by curvature and vehicle eccentricity causes the highest value near mid-span location. Inside loading position increases differential deflection, whereas decrease for the outside position with increase of skew angle and skew effect is highly remarkable for diaphragm 1.

The distortion-stress is not proportional to differential deflection in presence of load eccentricity or curvature because of torsional rotation of the bridge causes higher differential deflection without changing distortion-stress. The position of a vehicle on the bridge during running condition has a significant influence on alternative distortion-stress on same web-gap location as well as web bending. The maximum stress ranges obtained for location on outside diaphragm. This alternative web bending is highly responsible for fatigue crack formation. Skew angle also increases the distortion-stress.

Bearing force at obtuse corner increases and acute corner decrease with increase of skew angle both for skew-staggered and parallel bridge for any position of vehicle. It found bearing force increasing 27% for 1st end and 55% for 2nd end obtuse corner for skew angle 45^0 in skew-unstaggered bridge and for skew parallel bridge 20% and 38%, respectively.

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SEISMIC VULNERABILITY ANALYSIS OF OPEN GROUND STORY REINFORCED CONCRETE BUILDINGS

M. N. Islam[1*] & T. K. Nath[1]

[1] *Department of Civil Engineering, Dhaka University of Engineering & Technology, Gazipur, Gazipur-1700, Bangladesh.*
**Corresponding Author, <nazrul2100@yahoo.com>*

ABSTRACT

Phenomenon of seismic vulnerability of open ground story (OGS) reinforced concrete (RC) buildings and their past experiences in earthquakes have attracted researchers in this field. This study evaluates the seismic vulnerability of four OGS RC buildings and determines their performance level in terms of base shear versus roof displacement, spectral acceleration versus spectral displacement, hinges formed, story drift and story stiffness. Seismic vulnerability was assessed by nonlinear static (pushover) analysis using ETABS software. The study shows that OGS buildings are highly vulnerable to seismic load. The study also shows that the seismic vulnerability of OGS RC buildings can be reduced by providing shear walls in the service core.

Keywords: Open ground story, soft story, seismic analysis, story drift, spectral displacement.

INTRODUCTION

Providing ground story opened is a common practice for architects and proprietors of buildings all over the world. Need for car parking is the main reason for planning open ground floor. But, unfortunately, such types of buildings are vulnerable to shaking of ground derived even from a minor earthquake (Murty, 2005). Recent earthquakes show the vulnerability of such types of buildings. Soft story mechanism has been reported by researchers by which this open ground story phenomenon can be categorized. It is true that stiffness irregularity is the main cause of this problem (Haque & Amanat, 2008).

This paper presents the effect of open ground story (OGS) on the seismic vulnerability of buildings. A total of four numbers of G+7 story reinforced concrete (RC) building models have been sorted out from various types of models for this study to evaluate their vulnerability during earthquake shaking. The models have been analysed using nonlinear static (pushover) analysis method as per ATC-40 (1996) and BNBC (1993). The software ETABS 9.7 has been used for this purpose. The models have been developed for open ground story buildings with varying different parameters like providing masonry walls and shear walls in the service core and bigger section of columns at the ground floor.

MODELING OF OPEN GROUND STORY BUILDINGS

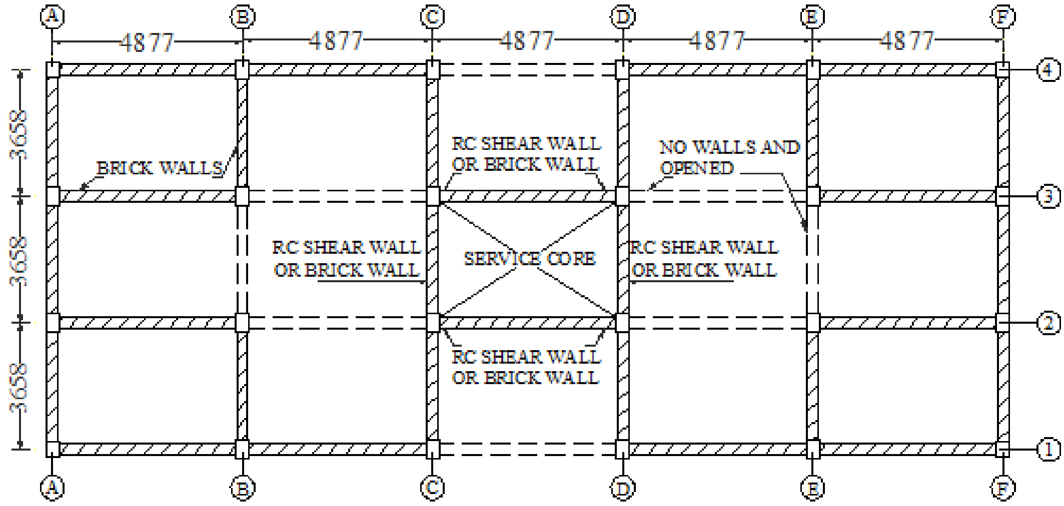
Four G+7 story reinforced concrete (RC) buildings (referred to as MODEL-1, 2, 3 and 4) have been taken and analyzed using software ETABS 9.7 to evaluate their vulnerability under seismic loading. The plan of the building is shown in Fig. 1(a). Sections at C-C of the buildings are shown in Figs. 1(b) through 1(e). The main characteristics of the buildings are as follows:

MODEL-1: Open Ground Story (OGS) with 5″ (127 mm) thick brick masonry wall in the upper stories [Fig. 1(b)].

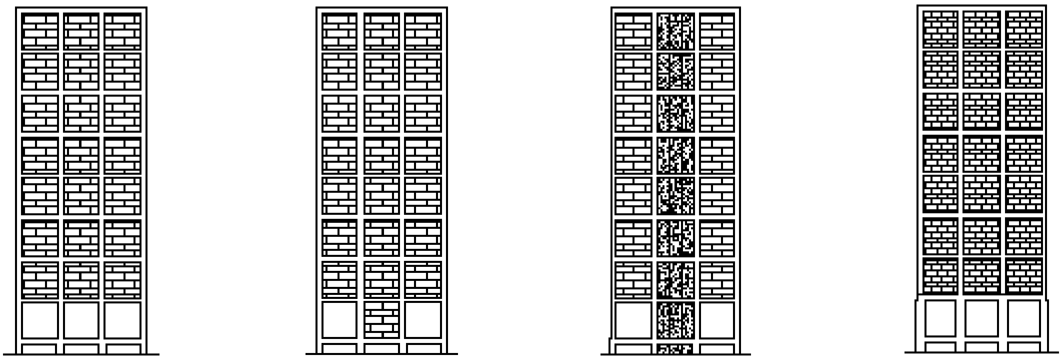
MODEL-2: Same as MODEL-1. In addition, 5″ (127 mm) thick brick masonry wall is provided in central service core of the ground floor [Fig. 1(c)].

MODEL-3: Same as MODEL-1. In addition, 8″ (203 mm) thick RC shear wall is provided in central service core [Fig. 1(d)].

MODEL-4: Same as MODEL-1. In addition, larger sections of columns are provided in ground floor [Fig. 1(e)].



(a)



(b) (c) (d) (e)

Fig. 1. Plan and sections of the buildings: (a) Plan, (b) Section C-C of MODEL-1, (c) Section C-C of MODEL-2, (d) Section C-C of MODEL-3, (e) Section C-C of MODEL-4. All dimensions are in mm.

The buildings are considered to be located in seismic zone-2 (Z = 0.15) as per BNBC (1993) and intended for residential purpose. Elastic modules of concrete and brick masonry are assumed as 2.5×10^4 N/mm^2 and 3.5×10^3 N/mm^2 respectively. The unit weight of concrete and masonry are taken as 2.3×10^{-5} N/mm^3 and 1.88×10^{-5} N/mm^3 respectively. The live loads and floor finish on floors are considered as 1.9 kN/m^2 and 1.2 kN/m^2 respectively. Size of beams and columns are taken as 254×406 mm and 305×508 mm respectively except for ground floor columns in MODEL-4 (508×635 mm). Three percent (3%) rebar is taken for smaller columns while 2.16% rebar is taken for larger columns. Slab thickness is taken as 127 mm. In seismic load calculation, 25% of live load is considered.

For pushover analysis using ETABS, the parameters of seismic zone, earthquake hazard levels, near source factor, soil profile type and type of structure are considered as Zone-2 (BNBC, 1993), 'Design earthquake', 'N=1 for seismic source type A ≥15 km', 'S_E' and 'type B' respectively (ATC-40, 1996). Brick walls have been modelled as strut (Holmes, 1961). Default hinges are considered as P-

M-M for columns, V2 and M3 for beams and 'P' for strut for its simplicity (Habibullah, 1995). Location of hinge is considered according to the literature (Inel & Ozmen, 2006).

SEISMIC VULNERABILITY ANALYSIS

(a) Vulnerability in terms of Roof Displacement

Fig. 2 shows base shear versus roof displacement diagrams for the models subjected to seismic loading along the long direction of the buildings. The roof displacements at performance point of MODEL-1, 2, 3 and 4 for the same seismic load were found as 104.25 mm, 89.60 mm, 26.94 mm and 82.12 mm respectively. This indicates that the base model (MODEL-1) is the most vulnerable to seismic load among the models considered.

Reductions in roof displacement in percent for the models with respect to that of MODEL-1 are also shown in Fig. 2. It can be seen that MODEL-3 (e.g. model with shear wall in service core) got the most reduction of roof displacement (i.e. 74%). On the other hand, the model (MODEL-2) having brick wall in service core shows the least reduction in roof displacement (i.e. 14%). The reduction of roof displacement of MODEL-3 (74%) is larger than that of MODEL-4 (21%). This indicates that MODEL-3 is the least vulnerable among the model considered. Therefore, it may be said that, by providing shear wall in central service core the vulnerability regarding roof displacement of OGS building can be reduced by nearly 74%. MODEL-2 and MODEL-4 show similar vulnerability because they are almost similar from structural point of view. Thus, inclusion of brick wall in service core or increase in column dimension at ground floor is not much effective in reducing seismic vulnerability of OGS RC buildings. However, between the two options, MODEL-2 is economical due to less cost of brick masonry than that of RC column.

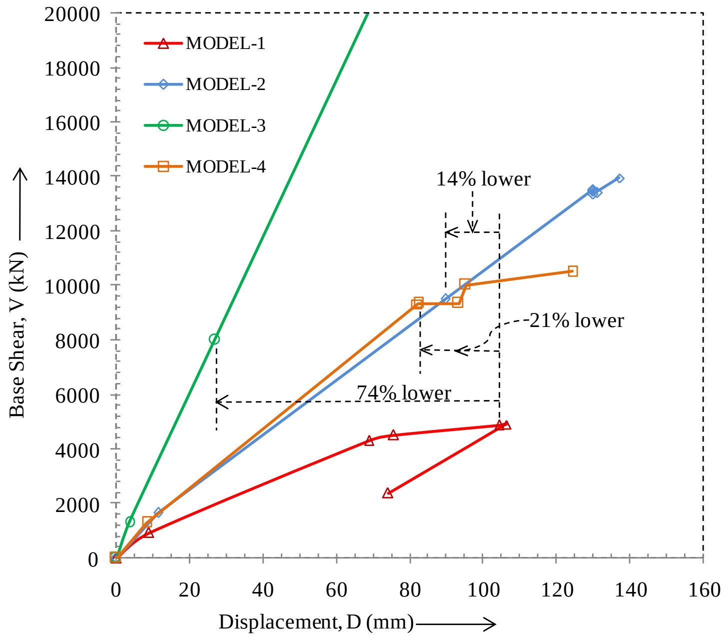


Fig. 2. Comparison of vulnerability in terms of roof displacement

(b) Vulnerability in terms of Spectral Displacement

Fig. 3 shows the spectral acceleration versus spectral displacement diagrams for the four models. From the figure, it can be seen that MODEL-3 shows the highest spectral acceleration with the lowest spectral displacement at the performance point. This indicates the ability of MODEL-3 to withstand a rapid ground shaking with producing the lowest spectral displacement.

It can also be seen that the capacity curves of MODEL-2 and MODEL-4 lie in the same region between the capacity curves of MODEL-1 and MODEL-3. Therefore it may be said that the MODEL-2 is better than that of MODEL-4 from the point of economical consideration.

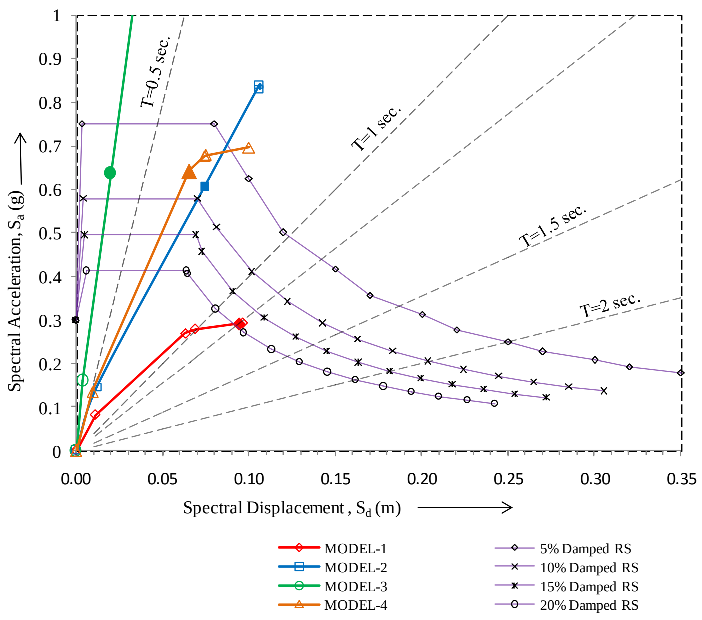


Fig. 3. Comparison of vulnerability in terms of spectral displacement

(c) Vulnerability in terms of Hinges Formed

Table 1 shows hinges formed at different stages of pushover analysis for the four models. Regarding total number of hinges formed, MODEL-1 and MODEL-2 are almost similar. On the other hand, total number of hinges formed in MODEL-3 and MODEL-4 are equal but less than those of MODEL-1 and MODEL-2. This indicates less vulnerability of MODEL-3 and MODEL-4 than those of MODEL-1 and MODEL-2.

Table 1 also shows that the performance level of the three models (MODEL-1, 2 and 4) are in the range of LS-CP (Life Safety-Collapse Prevention). But an additional hinge is formed in the range of C-D (Collapse-Damage) in each of the MODEL-1 and MODEL-2. On the other hand, MODEL-3 shows that its performance level does not exceed the range of IO-LS (Immediate Occupancy-Life Safety). Therefore it can be said that the MODEL-3 is the least vulnerable among the four models from hinges formed consideration.

Table 1. Comparison of hinges formed in four models

Model No.	Base Shear	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
MODEL-1	4888	1795	61	33	72	0	1	0	0	1962
MODEL-2	9494	1724	165	28	48	0	1	0	0	1966
MODEL-3	8001	1691	214	29	0	0	0	0	0	1934
MODEL-4	9321	1721	137	67	9	0	0	0	0	1934

A: Initial Stiffness, **B**: End of Elastic Stiffness, **IO**: Immediate Occupancy, **LS**: Life Safety, **CP**: Collapse Prevention, **C**: Collapse, **D**: Damage, **E**: Energy Loss

It was observed from the deformed shape of the models that the location of the major amount of hinges formed in MODEL-1 was in its ground floor, which indicates that the ground floor of the OGS building (MODEL-1) is most vulnerable. In the cases of MODEL-2, 3 and 4, these hinges formed have been found distributed in all floors and thus reducing their concentration in the ground floors. This indicates that the vulnerability of the ground floor for those buildings has been reduced.

(d) Vulnerability in terms of Story Drift

According to BNBC (1993), story drift should be ≤ 0.005h for T < 0.7 sec. and ≤ 0.004h for T ≥ 0.7 sec., where 'h' is the height of respective floor and 'T' is the time period. Fig. 4 shows the story drifts for the four models. It can be seen that the story drift at story-2 (ground floor roof) of MODEL-1 is 0.023h which exceeds the code limits. The story drifts of MODEL-2 and MODEL-4 have also exceeded the code limit. But story drift of the MODEL-3 is 0.0028h which is within the code limit. This is due to providing RC shear wall in the central service core in MODEL-3. It can be seen that, for all the models, the story drifts from story-3 to story-9 are almost similar in nature and within code limits. This is due to similar wall stiffness provided in those floors. However, in absence of required stiffness in ground floor, all the lower stories except that of MODEL-3 show drifts beyond the code limit. This indicates that all the models are vulnerable in terms of story drift except MODEL-3.

(e) Vulnerability in terms of Story Stiffness

Variation of stiffness in any floor may make a building weak or soft (low stiffness) in that level which may turn the building unsafe in seismic prone area. MODEL-1 is such kind of building with soft (low stiffness) nature in its ground floor.

Fig. 5 shows the variation of story stiffness with building height. It can be seen that the stiffness curves of MODEL-1, 2 and 4 are almost similar in nature with their lower part got radical change due to similar stiffness in upper floors. Analysis shows that the ground floor stiffness ratios (K_g) of MODEl-1, 2, 3 and 4 are 16%, 30%, 117% and 49% of their immediate upper stories, respectively. This indicates that only MODEL-3 meets the BNBC (1993) stiffness requirement.

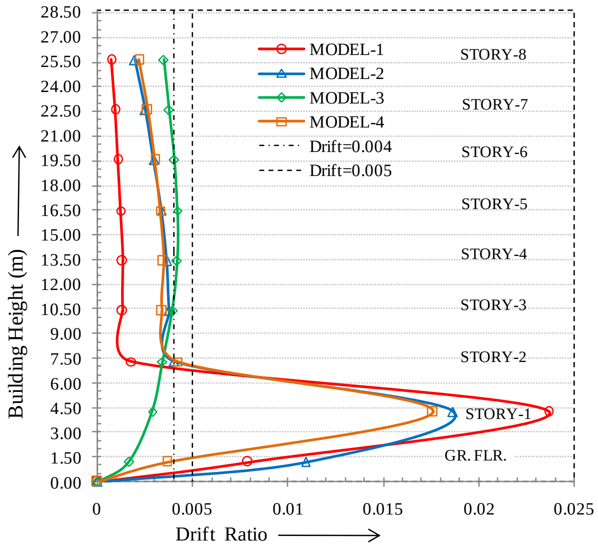


Fig. 4. Comparison of vulnerability in terms of story drift

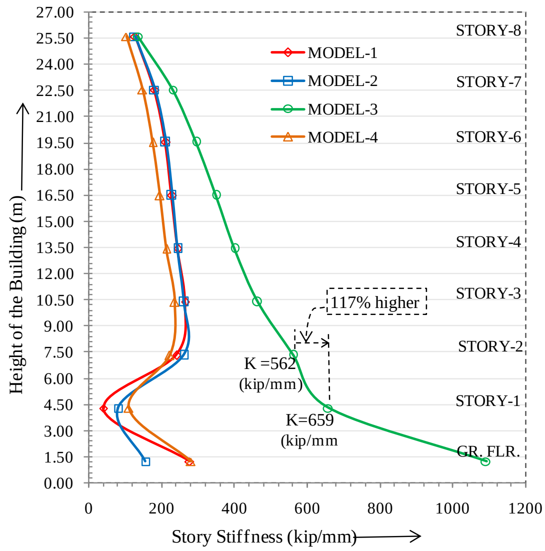


Fig. 5. Comparison of vulnerability in terms of story stiffness

It may be said that the value of stiffness ratio (K_g) of MODEL-2 provided with brick wall in central service core shows almost in the margin of code limit. This indicates that the MODEL-2 can be made acceptable by increasing the wall thickness slightly. Thus more research is needed to make the OGS building less vulnerable to seismic load economically.

CONCLUSION

The study shows that open ground story (OGS) reinforced concrete (RC) building is vulnerable to seismic load. MODEL-1 shows a large amount of roof displacement in comparison to other three models, which indicates that OGS building is vulnerable to seismic load in terms of roof displacement. This vulnerability may be overcome by providing shear wall in central service core as in MODEL-3 which has shown about 74% lower roof displacement. In terms of spectral displacement (S_d), the MODEL-1 also shows its vulnerability through higher value of S_d. MODEL-3 shows improved performance in this case. Due to large numbers of hinges formed in ground floor columns within the range of LS-CP, MODEL-1 proves again its vulnerability in this respect. Interestingly, by providing shear wall (MODEL-3), its hinges formed status can be improved within the range of IO-LS.

Furthermore, due to sudden reduction of stiffness in ground floor of OGS building (MODEL-1), story drift also does not meet the limit of BNBC (1993) code. This can be improved through incorporation of shear wall (MODEL-3) which turns the building safe in terms of story stiffness. The other two models (i.e. MODEL-2 and 4) show their capacity curves having almost similar in nature, lying at the intermediate place of the curves of OGS building (MODEL-1) and the building with shear wall in central service core (MODEL-3). However, they do not satisfy the drift and story stiffness criteria of BNBC.

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STRUCTURAL STRENGTH AND BEHAVIOUR OF MULTI-WEB PROFILED STEEL SHEET SUBJECTED TO WEB CRIPPLING FOR INTERIOR ONE FLANGE LOADING

S. M. Zahurul Islam[1*] & Md. Parves Hossain[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi-6204, Bangladesh.*
**Corresponding Author, <zahurul90@gmail.com>*

ABSTRACT

Profiled steel sheets have been increasingly used in architectural and structural application in roofing and walling of housing construction due to its attractive features in terms of durability, superior corrosion resistance, easy maintenance, ease construction, aesthetic appearance and recyclability of the material. Web crippling may occur at the highly concentrated loading or reactions in multi-web profiled steel sheet. Web crippling is a form of localised buckling or yielding that occurs at points of concentrated loads or supports of structural members. The web crippling failure of profiled steel sheet is a common failure mode of profiled steel sheet. The structural strengths and behaviour of profiled steel sheet subjected to web crippling is investigated in this study. The objective of the research is to determine the web crippling strength and behaviour of multi-web profiled steel sheet. The web crippling tests were conducted under Interior One Flange (IOF) loading conditions. The specimens were tested under unfastened and fastened to the support. An experimental investigation was conducted on multi-web members subjected to Interior One Flange (IOF) loading. The test strengths are also compared with the design strengths obtained using North American (2002) Specification. It is shown that the design strengths predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. The failure loads, failure modes and the load-web deformation behaviour of the profiled steel sheet are presented in this study. It is observed that the specimens tended to fail in the central portions unless the central portions were not reinforced by additional deck pieces of the same type. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet. It is recommended that the web crippling strength of multi-web profiled sheet should take into account for economic design.

Keywords: Design strengths, Experimental investigation, Profiled steel sheet, Web crippling.

INTRODUCTION

Profile steel shell structures are used in roofing elements popularly due to aesthetic and economical use of materials. Profiled steel sheet are increasingly used in structural applications in recent years due to their lightness, corrosion resistance, high strength-to-weight ratio, ease of production, recyclable and availability. Recent research and development efforts are directed towards developing a light, economical and structurally strong material that can be precast and easily erected (Jagannath and Sekar, 1989). Tests on structural strength and behaviour of cylindrical profiled steel sheet roofing elements have been performed by Zahurul-Islam et al. (2006). Test results showed that the parabolic profiled steel sheet roofing element provided significant improvements to the roof's structural performance. However profiled steel sheet are often experience web crippling failure due to the high local intensity of concentrated loads or reactions. Web crippling is one of the failure modes that must be taken into consideration in profiled steel sheet design.

In an experimental study at the University of Waterloo, Gerges (1997) investigated on web crippling of single web cold formed C-sections steel members subjected to End-One-Flange Loading. New parameter coefficients for Parabakaran's expression was developed for C-sections subjected to interior one flange loading. Young and Hancock (1998) investigated web-crippling behaviour of cold formed steel unlipped channel sections at the University of Sydney. The specimens were tested under four different load conditions of web crippling: End One Flange (EOF), Interior One Flange (IOF), End Two Flange (ETF) and Interior Two Flange (ITF). Based on the test results, the AISI-1996 web-crippling capacity equations were found to be unconservative for the unlipped channel cross sections and a new equation was proposed using a simple plastic mechanism approach. An experimental study was conducted on web-crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange (EOF) loading by Onur A. (2002). The North American Specification (AISI S100 2007) has new web crippling coefficients for different load cases and different end conditions. However, in the End One Flange (EOF) loading case of multi-web deck sections the coefficients for the unfastened configuration were used as a conservative solution for the fastened case. Existing studies have revealed that web crippling of thin-walled members could be one of the major failure modes.

The main objective of this study is to determine the web crippling strength and behaviour of profiled steel sheet. The structural strengths and behaviour of profiled steel sheet subjected to web crippling is investigated in this study. The web crippling tests were conducted under Interior One Flange loading conditions. The test specimens were fastened to the support. An experimental investigation was conducted on multi web members subjected to Interior One Flange (IOF) loading fastened to the bearing plate/support. The test strengths are also compared with the design strengths obtained using both AISI (1996) and North American (AISI S100, 2007) Specification. The design strengths which is predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. The failure loads, failure modes and the load-web deformation behaviour of the profiled steel sheet are presented in this study. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet on the basis of present results. The web crippling strength may be considered in the design of profiled steel sheet.

MATERIAL PROPERTIES

The material properties of the profiled steel sheet were determined by tensile coupon tests. The test set-up and tested specimen after tensile test is shown in Fig. 1.

Figure 1: Profiled steel sheet tensile coupon test setup and failure mode

The tensile coupons were prepared and tested according to the American Society for Testing and Materials Standard (ASTM, 1997) and the Australian Standard AS 1391 (AS, 2007) for the tensile testing of metals using 12.5 mm wide coupons. The tensile coupons were extracted from the center of the web plate in the longitudinal direction of the untested specimens. The coupons were tested in a Universal Testing Machine and the load is applied gradually. Deformation is measured by using deformation gauge.

TEST SPECIMEN

Test specimens lying inside and outside of certain geometric profiled parameter ranges of the specifications were tested under interior one flange loading. Photograph of test specimen is shown in Fig. 2. In the case of Interior One Flange loading, the length of specimen depends on the length of the bearing plate. According to the Canadian Specification, minimum length of the bearing plate is 1.9 mm. In this test, there are two different length of bearing plate was used, they were 50 mm and 75 mm respectively. Specimen length was used according o North American (AISI S100, 2007) Specification.

Figure 2: Photograph of test specimen before test

WEB CRIPPLING TEST

The web crippling tests were carried out under Interior-One-Flange (IOF) loading condition specified in the ASCE Specification (ASCE, 2002). Test setup and failure mode of web crippling test is shown in Fig. 3. The webs of the specimens were buckled outward.

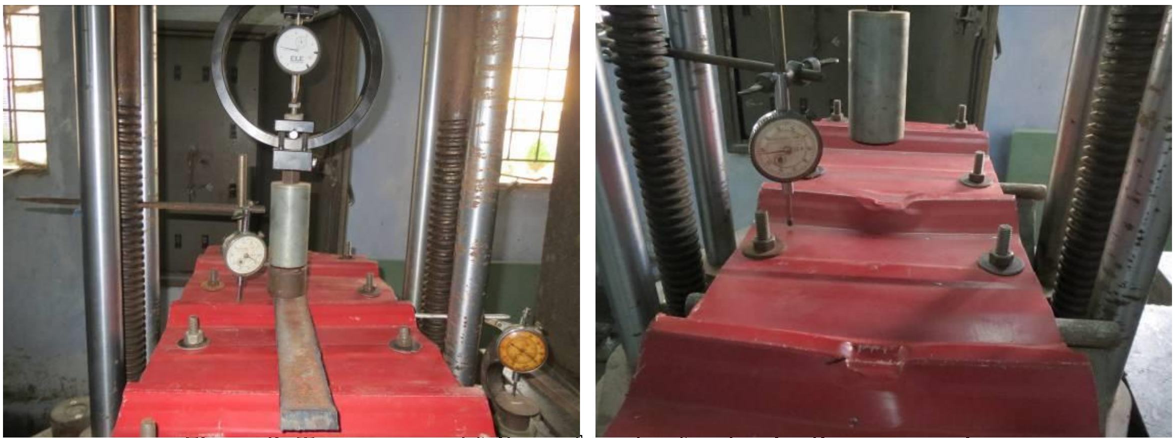

Figure 3: Test setup and failure of mode of web crippling test specimen

The load applied by the ram is simulated as a point load at the mid span location. Each specimen is prepared in a similar manner and simulated a simple beam in the entire experiment. The load applied by the ram is simulated as a point load at the mid span location. Photographs of the test setup of IOF loading condition is shown in Fig. 3. A servo-controlled hydraulic universal testing machine was used to apply a concentrated compressive force to the test specimens. In this test, the ends of the sample were bolted to the support. Specimens were fastened to the supports using 11 mm (7/16 in.) bolts with a washer being placed under the bolt head only. The bearing plate was placed under the loading point. The load is applied gradually by using Universal Testing Machine. The speed of the applying load is constant throughout the test for all specimens. Dial gauge was used to recorded deformation. The web deformations of the specimens were obtained by the readings of the dial gauge. Load–web deformation was recorded. The maximum load was recorded as the web crippling strength of the specimen as shown in Table 1.

TEST RESULTS AND DISCUSSIONS

The failure modes of profiled steel sheet are shown in Fig.3. It is observed that the specimens tended to fail in the central portions of multi-web case. The progression of crippling on the webs of the specimens initiated at an interior web as the load increased. The crippling of the webs caused deformation on the tension flanges of the specimens and moved the tension flanges upwards. Yu (2000) also observed this type of behaviour. The amount of resistance provided by the webs was higher in fastened cases than unfastened ones. Load-web deformation behaviour of profile steel sheet using single web and multi-web; fastened and unfastened and different bearing plate is shown in Fig. 4.

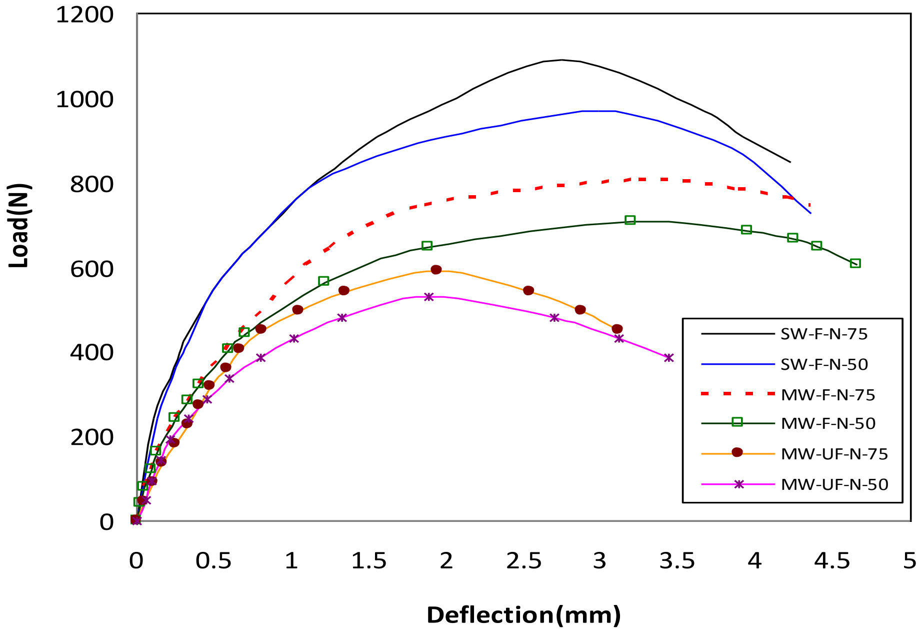

Figure 4: Comparison load-web deformation curves for under different condition

The maximum load carried by each specimen was recorded as the web crippling strength of the specimen. The web crippling test strengths is shown in Table 1 for IOF loading condition. Observation of the tests revealed that there is an increase in web crippling strength of specimens when bearing length increases. The web crippling strength for IOF loading condition was higher of fastened case than unfastened case. Comparing single web and multi web, the web crippling strength was higher for single web than multi web. It was observe that the specimens tended to fail in the central portions of multi-web case.

According to the Canadian Specification and using North American coefficient the web crippling strength is determined. The unified North American (AISI S100, 2007) equation is provided in equation 1.

$$P_n = Ct^2 f_y \sin\theta \left(1 - C_R\sqrt{\frac{r_i}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{1}$$

Where C is the coefficient, t is the thickness of the web, f_y is the yield stress ($\sigma_{0.2}$ proof stress), θ is the angle between the plane of the web and the plane of the bearing surface, N is the length of the bearing, h is the depth of the flat portion of the web, C_R is the inside corner radius coefficient, C_N is the bearing length coefficient and C_h is the web slenderness coefficient. The limits for the unified web crippling equation for profiled steel sheet web sections when $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $\theta = 90°$. Test and analytical attempts have been made to understand web crippling response on profiled steel sheet. The profiled steel sheet specimens are tested with Universal testing Machine and maximum crippling strength are obtained. A comparesion is carried out between the test value (P_t) and the design predicted value (P_n) as shown in Table 1. The test value (P_t) and the predicted value (P_n) for the web crippling capacity is not close for most of the specimens. Based on the Table 1 results, P_t/P_n is below unity (1). It is shown that the design strengths predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. Therefore, a unified web crippling equation with existing coefficients for profiled steel sheet under IOF loading conditions should be revised.

The test results are compared with the predicted design strength values using the P_t/P_n ratios for single web - multi-web; fastened –unfastened and different bearing plate. All test specimens resulted in P_t/P_n values lower than unity. North American Specification method resulted in P_t/P_n values lower than unity for most of the specimens, meaning that the tested web-crippling values are slightly lower than the predicted web-crippling values. This makes the analytical approaches unconservative. Here the test results for all the specimens are lower than analytical results. The web crippling strength of profile steel sheet can be calculated using the North American unified equation with revised the coefficient, and the material properties of longitudinal tension or transverse compression can be used. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet. It is recommendated that the web crippling strength should be considered to achieve more effective designs of profile steel sheet.

Table 1: Comparison of Analytical results with Experimental results for different condition

Condition/Case		**Bearing length (mm)**	**Length (mm)**	**Thickness (mm)**	**Web crippling strength P_t (kN)**	**Web crippling strength P_n (kN))**	**P_t / P_n**
Muli-web	Fasten	50	335.1	0.66	0.707	0.803	0.88
		75	460.0	0.66	0.805	0.956	0.84
	Unfasten	50	335.1	0.66	0.529	0.601	0.88
		75	460.0	0.66	0.592	0.704	0.84

Single web	Fasten	50	335.1	0.66	0.970	1.066	0.91
		75	460.0	0.66	1.090	1.267	0.86
	Unfasten	50	335.1	0.66	0.725	0.796	0.91
		75	460.0	0.66	0.817	0.950	0.86

CONCLUSIONS

The paper presents an experimental investigation of profiled steel sheet subjected to web crippling. The specimens were tested under Interior-One-Flange loading conditions in accordance with the ASCE Specification (2002). The concentrated loads were applied by means of bearing plates. It was observe that the specimens tended to fail in the central portions of multi-web case. The flanges of the specimens were fastened (restrained) to the bearing plates and unfastened. The failure modes, failure loads and load-web deformation behaviour of the profiled steel sheet sections have been also presented. It is found that the web crippling strength increases as the bearing length increases subjected to IOF loading. Web crippling strength of multi-web and unfasten case was lower than single web and factened case respectively. Test and numerical attempts have been made to understand web crippling response of profiled steel sheet. The test strengths were compared with the design strengths obtained using the current North American Specification. The ratio, P_t/P_n value is below 1. It is shown that the numerical design strengths predicted by North American specifications are unconservative. Therefore, the existing coefficients of North American unified web crippling equation for profiled steel sheet under IOF loading conditions can be revised. The design strengths should be calculated using the material properties obtained from coupon tests. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet in field application. It is recommended that the web crippling strength should be considered to achieve effective designs of profile steel sheet.

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SEISMIC SAFETY EVALUATION OF BAHADDARHAT HIGHWAY BRIDGE

H. Alim[1*], A.K.M. T. A. Khan[2], M. A. R. Bhuiyan[3]

[1] *Department of Civil Engineering, Lamar University, USA.*
[2] *Department of Disaster and Environmental Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.*
[3] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.*
**Corresponding Author, <choton_ce@yahoo.com>*

ABSTRACT

Seismic safety of Bahaddarhat Highway Bridge, located in Chittagong, Bangladesh, was evaluated following the guidelines suggested in various seismic codes. In this study, the ductility method of analysis as documented by Japan Road Association (JRA, 1996; 2002) was employed for safety evaluation of the highway bridge. The lateral strength, ductility and mode of failure of piers were considered as principal parameters in seismic safety evaluation of the highway bridge. Two approaches were utilised in estimation of lateral strength and ductility: the conventional analytical method and the nonlinear static method (i.e., pushover analysis method). In the second method of analysis, the fibre model with conventional constitutive models for concrete and steel for the piers was applied. A suite of acceleration spectra, for a set of ground motion records considering seismic risk around Chittagong city, was developed to incorporate in the seismic safety evaluation. Finally, the seismic safety of the highway bridge thus obtained was compared with that estimated for the design acceleration spectrum as recommended in Bangladesh National Building Code (BNBC, 2006; 2012 (Draft)).

Keywords: Highway Bridge, Lateral Strength, Ductility, Pushover Analysis, Seismic Risk, BNBC.

INTRODUCTION

Bridge structure plays very important role for evacuation and emergency routes for rescues, first aid, medical services, fire-fighting and transporting urgent disaster commodities. In view of the importance of Highway Bridge in transportation network, it is the key issue to minimize as much as possible loss of the bridge functions during earthquakes. In the last few earthquakes, for instance, the Kobe earthquake in 1995, the Northridge earthquake in 1994, the Chi-Chi earthquake in 1999, and the Chile and Haiti earthquakes in 2010 have demonstrated that a number of highway bridges have collapsed or have been severely damaged, even though they were subjected to earthquake ground shaking of an intensity that has been frequently less than the current code intensities.

The safety of the bridge bents are evaluated according to ductility design method (JRA, 2002). The design response spectra as suggested in the existing and revised National Building Codes (BNBC 2006; 2012 (Draft)) have been employed in the analysis. Three spectral accelerations corresponding to three peak ground accelerations (PGA) of 0.15g, 0.28g and 0.36g, as obtained from the design response spectra (BNBC, 2006; 2012) are used in safety evaluation. The PGA of 0.15g corresponds to the design peak ground acceleration for Chittagong region according to the existing National Building Code (BNBC, 2006) while the PGAs of 0.28g and 0.36g are the revised design peak ground accelerations for Chittagong and Sylhet regions as per the revised national Building Code (BNBC, 2012). Moreover, from a recent geological survey it has been observed that, a number of active faults distributed in and around Bangladesh are capable of generating moderate to strong earthquakes. These include the Dauki Fault, about 300 km long trending east-west and located along the southern edge of Shillong Plateau, the 150 km long Madhupur fault trending north-south located between Madhupur

Tract and Jamuna flood plain, the Assam-Sylhet fault, about 300 km long trending northeast-southeast situated in the southern Surma basin and Chittagong-Myanmar coast. The Chittagong-Myanmar plate boundary proceeds south to Surma where it ruptured in the disastrous event occurred on December 26, 2004 in the 2004 Sumatra earthquake (Steckler et al., 2008). These tectonic plate distributions in and around Bangladesh make it vulnerable for moderate to strong earthquakes. Following this aspects two additional PGA values (i.e., 0.28g and 0.36g) are used in the safety evaluation of the bridge bent.

MODELING OF THE BRIDGE BENT

Physical Model of the Bridge Bent

Chittagong city is surrounded by many primary and secondary road networks. In Chittagong Metropolitan Master Plan, there is a guideline for improvement of traffic network to reduce the traffic congestion within the city. A 1331.60m long Flyover (Abdul Mannan Flyover) connecting CDA (Chittagong Development Authority) Avenue road and Shah Amanat Bridge approach road has been constructed to reduce traffic congestion at the Bahaddarhat junction. A sectional elevation of the flyover is presented in Fig.1. There are 25 spans of variable lengths excluding the two approach roads at both ends of the flyover. The span length of the flyover varies from 35m to 42m. The length of each approach road located at the both ends of the flyover is 165.3m. The deck of the flyover comprises 6 to 7 pre-stressed concrete girders with 200mm reinforced concrete slab including asphalt wearing course. The girders rest on elastomeric rubber pad located on top of pier. There are 24 piers having variable heights ranging from 3.65m to 7.29m. The geometric dimensions of deck, piers and re-bar details of piers are presented in Fig.2 and Table 1. Material properties are presented in Table 2.

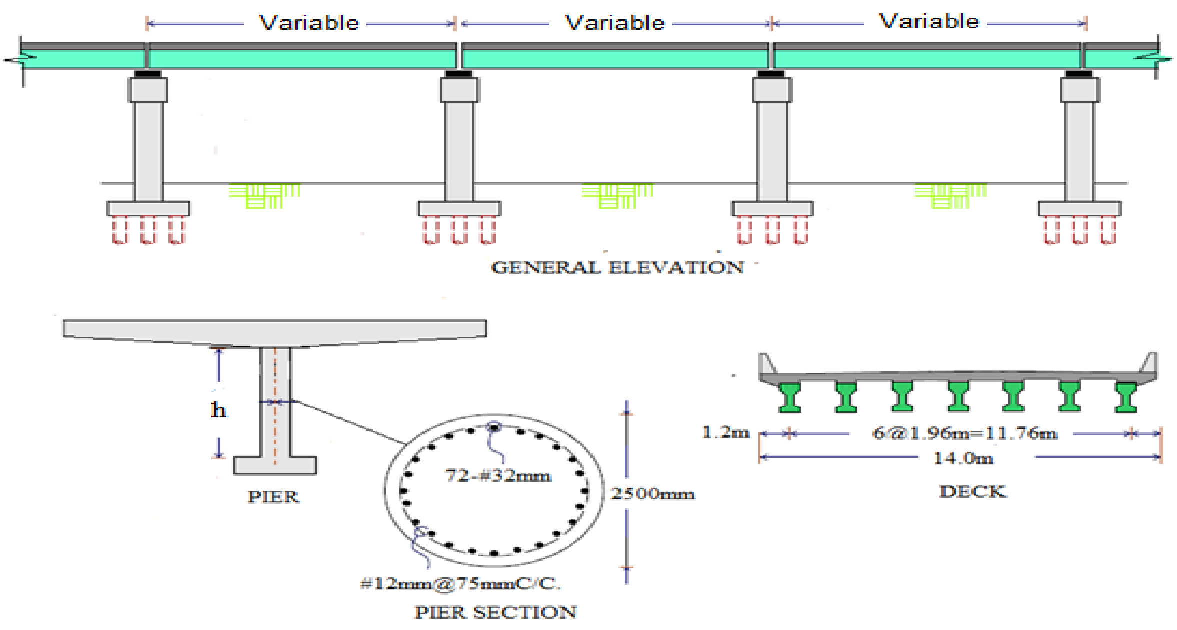


Fig.1 Geometrical model of Bahaddarhat highway bridge

Analytical Model of the Bridge Bent

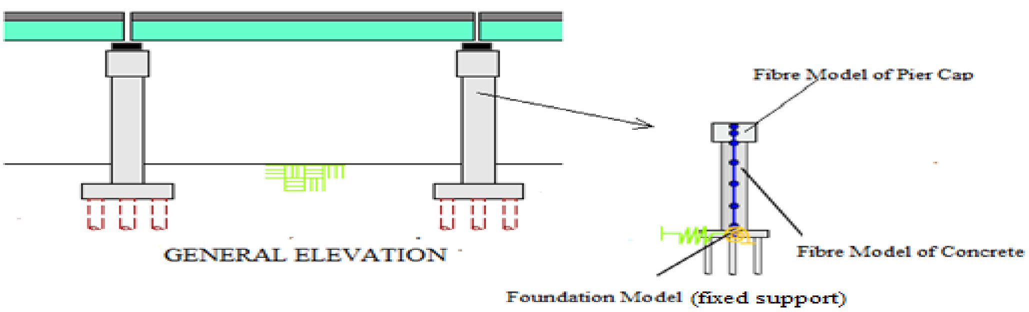
The analytical model of a tributary deck along with a pier (pier-girder system) is shown in Fig.2. This simplification holds true only when the bridge superstructure is assumed to be rigid in its own plane which shows no significant structural effects on the seismic performance of the bridge system when subjected to earthquake ground acceleration in longitudinal direction (Ghobarah et at., 1988). The pier-girder system is approximated as a continuous 2-D finite element frame using the nonlinear analysis program (SeismoStruct, 2012). Finite element model with frame elements is used to estimate the pier-girder system with a finite number of degrees of freedom. The superstructure & substructure of the system are modelled as a lumped mass system divided into a number of small discrete segments. The mass of each segment is assumed to be distributed between two adjacent nodes.

Table 1: Geometric Dimensions

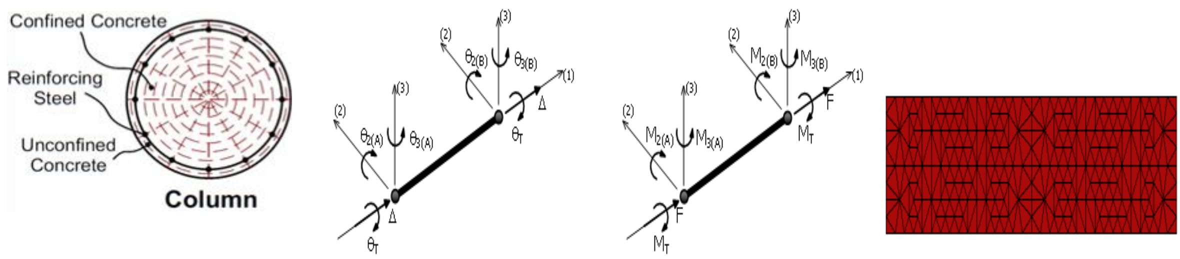
Pier No.	Pier Height (m)	Dia. (m)	Longitudinal Reinforcement
1	3.653	2.5	72 @ D32
2	4.917	2.5	72 @ D32
3	5.857	2.5	72 @ D32
4	6.557	2.5	72 @ D32
5 to 21	7.29	2.5	72 @ D32
22	7.057	2.5	72 @ D32
23	5.417	2.5	72 @ D32
24	4.153	2.5	72 @ D32

Table 2: Material Properties

Material Name	Description of Material Properties
Confined Concrete	Compressive strength = 35 MPa Tensile strength = 3 MPa Strain at Peak Stress = 0.0025 (mm/mm) Confinement factor = 1.2 Specific weight = 2.4 E-05 N/mm3
Steel	Modulus of elasticity = 200000 MPa Yield strength = 485 MPa Strain hardening parameter = 0.0075 Fracture/ buckling strain = 0.06 Specific weight = 7.8 E-05 N/mm3
Unconfined Concrete	Compressive strength = 35 MPa Tensile strength = 3 MPa Strain at Peak Stress = 0.0025 (mm/mm) Confinement factor = 1.0 Specific weight = 2.4 E-05 N/mm3



(a)



(b) (c) (d)

Fig.2 Analytical model of (a) the bridge bent, (b) fibre section of reinforced pier/ column, (c) beam element for pier modelling and (d) fibre section of reinforced pier-cap

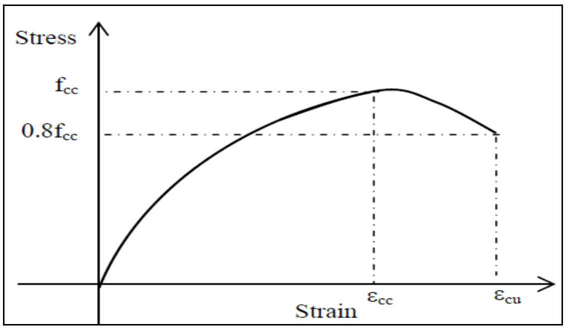
The superstructure consisting of RC decks and post-tensioned prestressed concrete girders is modelled using linear beam-column elements so that the superstructure remains elastic under the seismic loads applied in the longitudinal direction. The body of the bridge pier is modelled using the fibre elements. Each fibre has a stress–strain relationship, which can be specified to represent unconfined concrete, confined concrete, and longitudinal steel reinforcement. The confinement effect of the concrete section is considered on the basis of reinforcement detailing as discussed in the preceding section. The distribution of inelastic deformation and forces is sampled by specifying cross-section slices along the length of the element. The nonlinear force-displacement behaviour of the bridge pier should be considered in seismic analysis of a bridge system, especially in a seismically active zone. In such a region, the bridge piers are expected to incur large displacements during earthquakes, which lead to the fact that the linear force-displacement behaviour of a bridge pier will result in a very uneconomic design. The foundation movement effect is neglected in the analysis.

ASSESSMENT OF BRIDGE PIERS

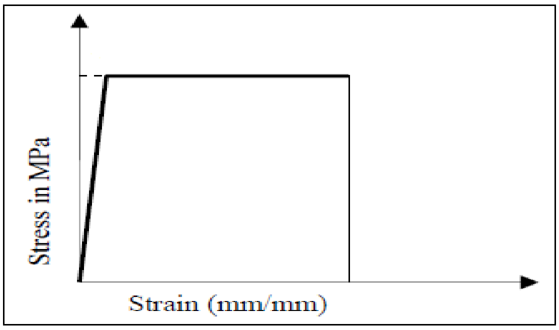
The lateral strength, ductility and mode of failure of bridge piers are computed using the method of nonlinear static analysis (i.e., pushover method) and the analytical method suggested by Japan Road Association (JRA, 2002). The sectional analysis has been conducted by professional software (Response, 2000). In addition, non-linear finite element software (SeismoStruct, 2012) is used to conduct the pushover analysis in order to derive the force-displacement relationship of a pier.

Development of Force-Displacement Relationship

The force-displacement relationship of piers can be derived from the results of moment-curvature relation at each section from top to bottom of a pier as obtained from sectional analysis of piers. Sectional properties of the piers are related to the characteristics of the materials i.e., stress-strain relationship and strength of materials. Different model of concrete are developed for seismic model (Park et al., 1985, Madas and Elnashai, 1992; Spoelstra et al., 1999). The cyclic behaviour of reinforced concrete is also critically assessed in the literature (Yankelevsky et al., 1989). A concrete model, which has been used extensively in recent years, was developed by Hoshikuma et al., (1997). The descending branch of the material law as well as the increase of strength and corresponding strain because of confined reinforcing steel is taken into account which is shown in Fig.3 (a). The bilinear model for reinforcing steel is used in the study and the constitutive model is shown in Fig.3 (b).



(a)



(b)

Fig.3 Constitutive model of materials (a) Concrete and (b) Steel

The model stress-strain curve consists of three parts i.e., an ascending branch, falling branch, and sustaining branch. The stress-strain curve can be expressed as below.

$$f_c = \begin{cases} E_c \varepsilon_c \left\{ 1 - \dfrac{1}{n\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)} \right\} & (0 \le \varepsilon_c \le \varepsilon_{cc}) \\ fcc\left\{ 1 - E_{des}\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right) \right\} & (\varepsilon_{cc} \le \varepsilon_c \le \varepsilon_{cu}) \end{cases} \tag{1}$$

Where, n is coefficient and E_{des} is deterioration rate and are given as,

$$n = E_c \varepsilon_{cc} \big/ (E_c \varepsilon_{cc} - \sigma_{cc}) \tag{2}$$

$$\sigma_{cc} = \sigma_{c0} + 3.8\alpha\rho_s\sigma_{sy} ; \varepsilon_{cc} = 0.002 + 0.033\beta \frac{\rho_s \sigma_{sy}}{\sigma_{c0}} \tag{3}$$

$$E_{des} = 11.2 \frac{\sigma_{c0}^2}{\rho_s . \sigma_{sy}} \tag{4}$$

where, σ_{c0} is design strength of concrete, σ_{sy} is the yield strength of reinforcement, α and β are shape factors and ρ_s is the volumetric ratio of tie reinforcements. The ultimate displacement d_u is defined as displacement at the gravity centre of superstructure when the concrete compression strain at out-most reinforcements reaches the following ultimate strain, ε_{cu}

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & type - I \quad earthquake \\ \varepsilon_{cc} + 0.2 \sigma_{cc} \big/ E_{des} & type - II \quad earthquake \end{cases} \tag{5}$$

where, α and β are modification factors depending on confined sectional shape: for circular β = 1.0 and α = 1.0; for square β = 0.2 and α = 0.4. To obtain the force-displacement relationship at top of the bridge pier, the pier is divided into N slices (50 slices are recommended in the code) along its height. For sectional analysis, it is mainly focused on three sections: (a) section at the top level, (b) section at one-third level from the bottom of the pier, and (c) section at the base level. This is because the configuration of the reinforcement at this level is different. Finally, the force displacement relationship at the top of the bridge pier is obtained using the moment-curvature diagrams and shear stress-strain diagram. Fig.4 shows numerical evaluation of moment curvature of piers. Steps for obtaining the force-displacement relationships are as follows:

1. The pier is divided into N slices along its height.

2. The moment-curvature diagrams for each cross-section are obtained through sectional analysis.

3. The horizontal force P is applied at the top of the pier.

4. The bending moment diagrams of the pier for the applied force P are drawn.

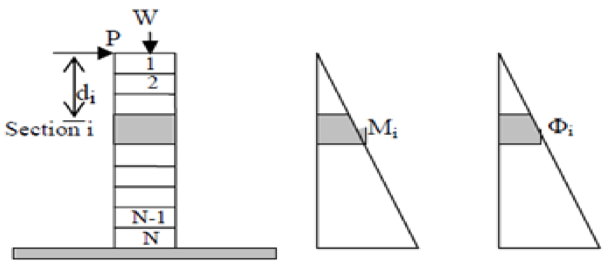
5. The curvature from bending moment and moment-curvature diagram is obtained.

6. The displacement, δ at the top of the pier is estimated using the following Equation

$$\delta = \sum_{i=1}^{N} \emptyset d_y \, d_i \tag{6}$$

where, is the curvature of the pier section i, d_y is the width of the pier cross section i and d_i is the distance from the top of the pier to centre of gravity of section i.

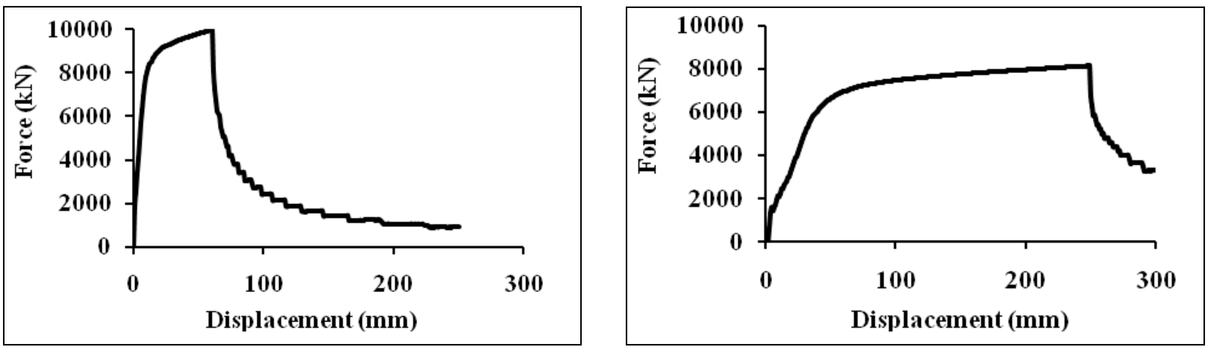
7. In a similar way, several forces P are applied and the corresponding displacement obtain. Finally, using these values, the force –displacement relationship at pier top is obtained.



Applied Load on Pier B.M.D Curvature Diagram

Fig.4 Numerical evaluation of moment curvature of piers (JRA, 2002)

The pushover analysis results for the piers are presented in the form of force-displacement relationships that are presented in Fig.5 below. Nonlinear software (SeismoStruct, 2012) is used to evaluate pushover results. Though all the piers have same sectional and material properties the shorter piers shown greater strength against applied lateral load and it is found that pier 1 have shown greatest yield capacity of 9994.867 kN whereas the tallest pier have shown yield strength of 4368.301 kN.



(a) (b)

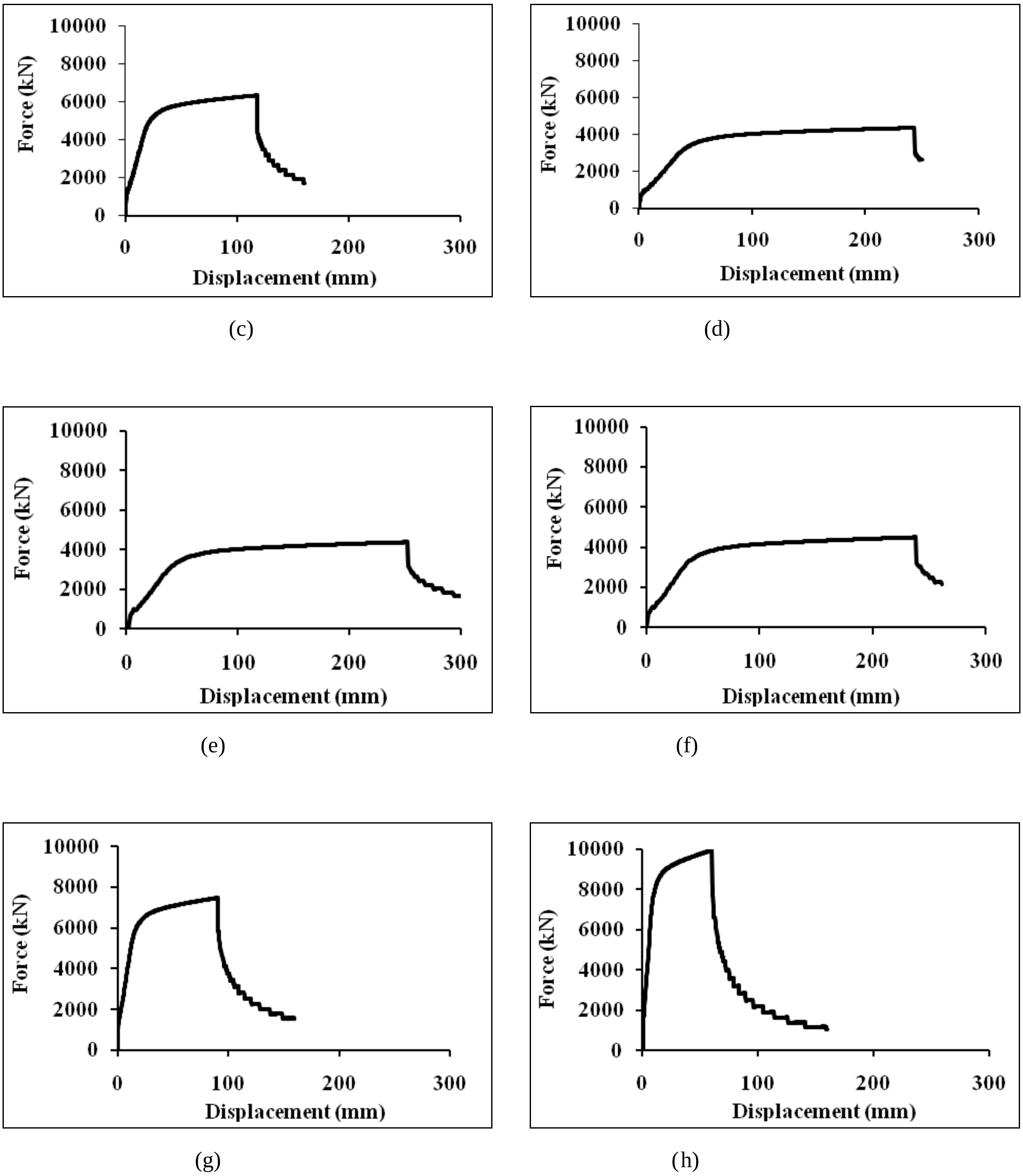


Fig.5 Force-displacement relationship of the piers (a) pier 1, (b) pier 2, (c) pier 3, (d) pier 4 (e) pier 5 to 21, (f) pier 22 (g) pier 23 and (h) pier 24.

Evaluation of Lateral Strength, Ductility and Failure Mode of Bridge Bent

Ductility and lateral strength of the bent is evaluated from both Shear and flexural capacity of the pier. Failure mode of bent is analysed according to the procedure suggested by Japan Road Association (JRA, 2002). Strength and design displacement ductility factor (μ_a) are determined depending on the failure mode of the elevated highway. Based on the flexural strength (p_u), shear strength (p_s) and shear strength under static loading (p_{so}) failure mode of a pier is decided to be one of the flexural failure, shear failure after flexural damage and shear failure as,

$$\text{Failure Mode} = \begin{cases} \text{flexural failure} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots p_u \leq p_s \\ \text{shear failure after flexural damage} \ldots \ldots \ldots \ldots . p_u \leq p_{so} \\ \text{shear failure} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . p_u \geq p_{so} \end{cases} \quad (7)$$

The lateral capacity p_a and the design displacement ductility factor μ_a are given as,

$$p_a = \begin{cases} p_u \ldots \text{flexural failure} + \text{shear failure after flexural damage} \\ p_{so} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . \text{shear failure} \end{cases} \quad (8)$$

$$\mu_a = \begin{cases} 1 + \dfrac{\delta_u - \delta_y}{\alpha . \delta_y} \\ 1 \end{cases} \quad (9)$$

where α is safety factor depending on importance of bridges and the type of ground motion (α =3.0 and 2.4 for important and ordinary bridges, respectively, under the near field ground motions, and α =1.5 and 1.2 for important and ordinary bridges, respectively, under the far field ground motions), δ_y and δ_u are yield and ultimate displacement of the bridge pier under earthquake ground motion. Using Eqn. 7 to 9, failure mode and ductility are obtained. The lateral strength, failure mode and ductility of the highway bridge are presented in Table 3.

Table 3: Lateral Strength, Ductility and Failure Mode

Pier No	Pier Height (m)	Failure Mode	Failure Criteria	Allowable Ductility	Lateral Capacity (Kip)
1	3.653	Shear Failure	$p_u \geq p_{so}$	1.0	9994.86
2	4.917	Shear Failure	$p_u \geq p_{so}$	1.0	8136.07
3	5.857	Flexural Failure	$p_u \leq p_s$	4.67	6389.56
4	6.557	Flexural Failure	$p_u \leq p_s$	4.56	4358.04
5 to 21	7.290	Flexural Failure	$p_u \leq p_s$	4.21	4368.30
22	7.057	Flexural Failure	$p_u \leq p_s$	4.30	4490.45
23	5.417	Shear Failure	$p_u \geq p_{so}$	1.0	7471.17
24	4.153	Shear Failure	$p_u \geq p_{so}$	1.0	9931.74

Safety Evaluation of the Bridge Bent

Twenty far field earthquake ground motion records were utilised to compare spectral acceleration in this study. The characteristics of the far field earthquake ground motion records are presented in Table 4 with PGA values ranging from 0.22g to 0.728g. The response spectrums of twenty far field ground motions and their average spectral acceleration are shown in Fig.6. The highest response spectrum acceleration is found to be 2.25g and the average response spectrum has seen around 1.0g.

Table 4: Earthquake Ground Motion Records

Earthquake	Name	Recording Station	PGAmax (g)	PGVmax (cm/s.)
EQ-1	Northridge	Beverly Hills – Mulhol	0.416	58.95

EQ-2	Landers	Yermo Fire Station	0.24	51.5
EQ-3	Northridge	Canyon Country-WLC	0.4	43.0
EQ-4	Landers	Coolwater	0.283	26
EQ-5	Duzce, Turkey	Bolu	0.7	56.4
EQ-6	Loma Prieta	Capitola	0.53	35
EQ-7	Hector Mine	Hector	0.3	28.6
EQ-8	Loma Prieta	Gilroy Array #3	0.56	36
EQ-9	Imperial Valley	Delta	0.2	26.0
EQ-10	Manjil, Iran	Abbar	0.51	43
EQ-11	Imperial Valley	El Centro Array #11	0.4	34.4
EQ-12	Superstition Hills	El Centro Imp. Co.	0.36	46.4
EQ-13	Kobe, Japan	Nishi-Akashi	0.5	37.3
EQ-14	Superstition Hills	Poe Road (temp)	0.45	35.8
EQ-15	Kobe, Japan	Shin-Osaka	0.2	38.0
EQ-16	Cape Mendocino	Rio Dell Overpass	0.385	43.8
EQ-17	Kocaeli, Turkey	Duzce	0.3	59.0
EQ-18	Chi-Chi, Taiwan	CHY101	0.353	70.65
EQ-19	Kocaeli, Turkey	Arcelik	0.2	17.7
EQ-20	Chi-Chi, Taiwan	TCU045	0.474	36.7

Fig.7 shows different percentiles of far field earthquake ground motion records. The spectral accelerations of 0.15g, 0.28g and 0.36g are plotted according to BNBC (2006). The percentile of far field ground motions are illustrated for comparison. A percentile of spectrum acceleration is a measure used in statistics indicating the value below which a given percentage of observations in a group of observations fall. The 25th percentile is the value below which 25 percent of the observations may be found. The 25th percentile is also known as the first quartile, the 50th percentile as the median or second quartile, and the 75th percentile as the third quartile.

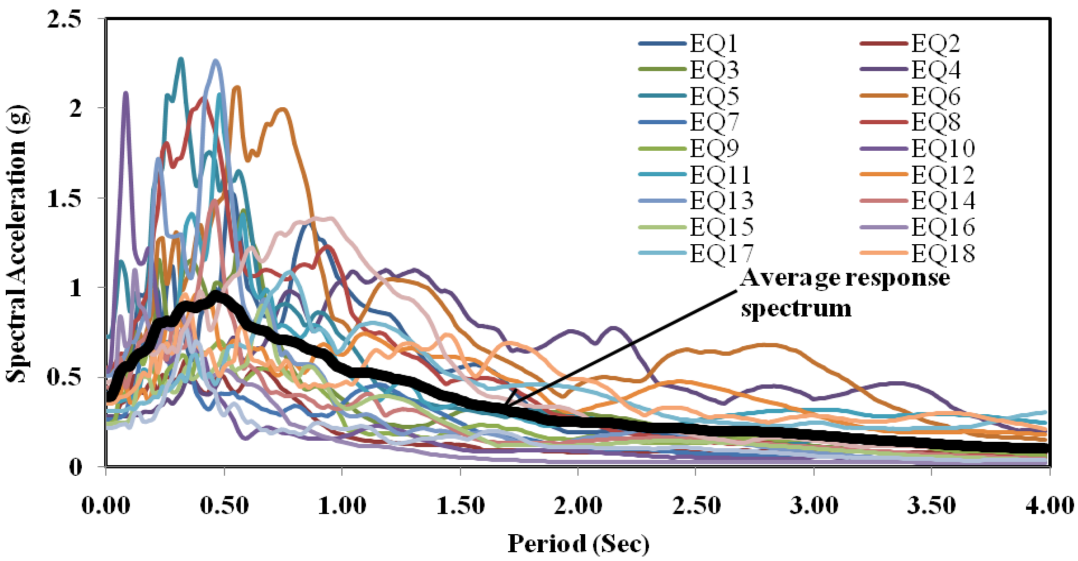


Fig.6 Response spectral acceleration of earthquakes

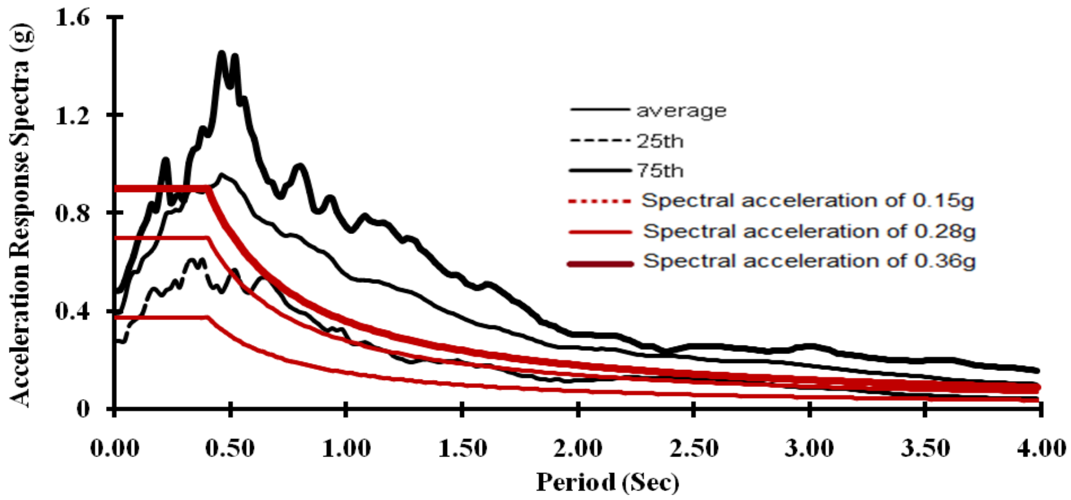


Fig.7 Spectral acceleration (percentile and BNBC response spectrum)

Safety of the pier is evaluated by comparing the lateral force demand to the lateral force capacity. Eqn. 10 is used to determine lateral force demand, p_a for a particular spectral acceleration,

$$p_a = \left(\frac{WS_a}{gR}\right) \tag{10}$$

where, S_a is the spectral acceleration, W is the total dead load, g is the acceleration due to gravity and R is the response modification factor. The lateral force capacity of the pier is evaluated by conducting nonlinear static analysis (i.e., push over analysis) as discussed in preceding Section. The response modification factor, R can be found from Eqn. 11,

$$R = \sqrt{2\mu_a - 1} \tag{11}$$

The safety of bridge piers against 0.15g spectral acceleration are tabulated in Table 5. It is seen from the results that all the piers are in **"Safe"** stage. The results of safety analyses against 0.28g and 0.36g spectral accelerations are shown in Table 6 and Table 7 respectively. The bridge piers are in **"Not Safe"** state for these two response spectral accelerations.

Table 5: Safety evaluation of piers for 0.15g response spectral acceleration

Pier No	Allowable Ductility	Response Modification Factor, R	Spectral Acceleration, S_a (m/s^2)	Equivalent Response Acceleration, S_a/R	Lateral Force Demand, P_a (KN)	Lateral Capacity, P_u (KN)	Safety Status
1	1.0	1	3.68	3.68	5625	9994.8	Safe
2	1.0	1	3.68	3.68	7500	8136.0	Safe
3	1.0	2.87	3.68	1.28	3895.5	6328.1	Safe
4	4.56	2.85	3.68	1.29	3947.3	4358.0	Safe
5 to 21	4.21	2.72	3.68	1.35	4136.0	4368.3	Safe
22	4.30	2.76	3.68	1.33	4076.0	5976.9	Safe
23	1.0	1	3.68	3.68	7500	7945.3	Safe
24	1.0	1	3.68	3.68	5625	9994.8	Safe

Table 6: Safety evaluation of piers for 0.28g response spectral acceleration

Pier No	Allowable Ductility	Response Modification Factor, R	Spectral Acceleration, S_a (m/s^2)	Equivalent Response Acceleration, S_a/R	Lateral Force Demand, P_a (KN)	Lateral Capacity, P_u (KN)	Safety Status
1	1.0	1	6.86	6.86	10500	9994.8	Not Safe
2	1.0	1	6.86	6.86	14000	8136.0	Not Safe
3	1.0	2.87	6.86	2.37	7271.7	6328.1	Not Safe
4	4.56	2.85	6.86	2.40	7368.4	4358.0	Not Safe
5 to 21	4.21	2.72	6.86	2.52	7720.5	4368.1	Not Safe
22	4.30	2.76	6.86	2.48	7608.6	5976.9	Not Safe
23	1.0	1	6.86	6.86	14000	7945.3	Not Safe
24	1.0	1	6.86	6.86	10500	9994.8	Not Safe

Table 7: Safety Evaluation of piers for 0.36g response spectral acceleration

Pier No	Allowable Ductility	Response Modification Factor, R	Spectral Acceleration, S_a (m/s^2)	Equivalent Response Acceleration, S_a/R	Lateral Force Demand, P_a (KN)	Lateral Capacity, P_u (KN)	Safety Status
1	1.0	1	9.32	9.32	14250	9994.8	Not Safe
2	1.0	1	9.32	9.32	19000	8136.0	Not Safe
3	1.0	2.87	9.32	3.23	9868.7	6328.1	Not Safe
4	4.56	2.85	9.32	3.27	10000	4358.0	Not Safe
5 to 21	4.21	2.72	9.32	3.43	10477.9	4368.3	Not Safe
22	4.30	2.76	9.32	3.38	10326.0	5976.9	Not Safe
23	1.0	1	9.32	9.32	19000	7945.3	Not Safe
24	1.0	1	9.32	9.32	14250	9994.8	Not Safe

CONCLUDING REMARKS

Lateral load resistance of the bridge piers was evaluated based on the force-displacement relations as obtained from the Push-over analysis results. Subsequently, the failure mode and allowable ductility of the bridge piers had been obtained following the guidelines of JRA (2002), which were then employed in the safety evaluation of the bridge piers. The Ductility Method (JRA, 2002) has been used in safety evaluation of the bridge piers. In this case, the design acceleration spectrum as recommended in BNBC (2006) was employed in the analysis. Three spectral acceleration responses corresponding to earthquakes of 0.15g, 0.28g and 0.36g PGA were used in the Ductility Method. From the results it has been found that the bridge bents of length less than 5.22 m are susceptible to shear failure whereas the bents of length more than 5.22 m are more likely to fail by flexure. Moreover, the lateral capacity of the bridge bent is quite capable of withstanding the lateral forces due to the earthquake of PGA 0.15g. However, for the earthquakes of PGA 0.28g and 0.36g, the bridge bent appears to be very weak in supporting the lateral forces

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AN IMPROVED RHEOLOGY MODEL OF HIGH DAMPING RUBBER BEARING FOR SEISMIC ANALYSIS

A.K.M. T. A. Khan[1] & M. A. R. Bhuiyan[2]

[1] *Department of Disaster and Environmental Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.*
[2] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.*
**Corresponding Author, <thohidul.ce@gmail.com>*

ABSTRACT

This study is devoted towards developing an improved rheology model of high damping rubber bearings (HDRBs) for seismic analysis of bridges. In this regard, the mechanical behaviour of HDRB was investigated through a set of mechanical tests under horizontal shear deformation with a constant vertical compressive load. The tests as conducted -with the existence of the Mullins' softening behaviour, strain and strain-rate dependent behaviour, and strain hardening features at high strain levels are discussed. Having motivated by the test results an elasto-visco-plastic rheology model was proposed to characterize the mechanical behaviour of the HDRBs for seismic analysis. The proposed rheology model was derived from the Maxwell model by adding a nonlinear elastic element and an elasto-plastic element in parallel, in addition to the dashpot element. A nonlinear viscosity law of the dashpot element was deduced from the experimental results and was subsequently incorporated into the rheology model to reproduce the nonlinear rate dependent viscosity of the HDRBs. Numerical simulation of the test results demonstrated the adequacy of the proposed model. The capability of the proposed model was further checked by conducting nonlinear dynamic analysis of a seismically isolated bridge structures. The seismic responses of the bridge, thus obtained using the proposed model of the HDRBs, were compared with those obtained by the available rheology model.

Keywords: Improved Rheology Model, High Damping Rubber Bearing, Seismic Isolation, Bridge.

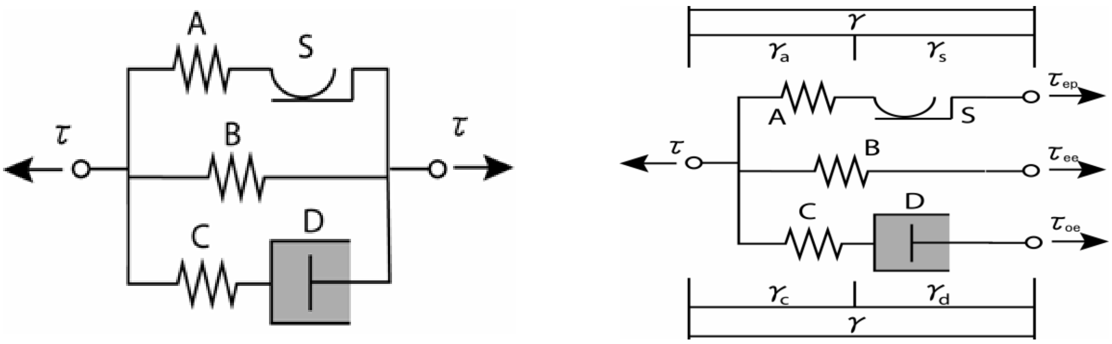
INTRODUCTION

Bridges stands as the economic benchmark for countrywide transportation development. It strongly supports the smooth movement of goods and lives by establishing the links between cities, urban and semi urban areas. A large number of bridge structures collapsed in recently occurred destructive earthquakes. Particularly, the Northridge (1994) and Kobe (1995) earthquake have exposed inadequacy of the design of existing bridge structures, which push on engineers to rethink widely on how to design bridge structures against earthquakes. Natural Rubber Bearing (RB), Lead Rubber Bearing (LRB), High Damping Rubber Bearing (HDRB) are worldwide used different isolation devices in bridges (Bhuiyan and Okui, 2012). Due to the capability of loads while sustaining large movements with little or no maintenance requirement, very high extensibility and compressive strength together with fatigue, abrasion and corrosion resistant characteristics, the high damping rubber bearings (HDRBs) have been found more and more applications in recent years as seismic isolation devices in bridges (Bhuiyan, 2009; Khan, 2014). The mechanical behaviour of the HDRBs is largely dominated by the nonlinear strain rate dependence in addition to the strain history and axial load. A good understanding of the mechanical behaviour of the HDRBs, in terms of strain-rate dependent hysteresis along with the strain history dependent elasto-plastic behaviour under the conditions of interest of seismic applications, is necessary towards development of a rational design of seismic isolation system. The currently practiced rheology model was developed based on 175% maximum strain level (JRA, 2002) which may not be adequately employed for describing the above mentioned mechanical behaviour of the HDRBs in high strain region (250% maximum strain level). In light of the above thinking, an improved rheology model and parameter identification scheme of

HDRBs was developed. Moreover, a solution algorithm was proposed for nonlinear dynamic analysis of bridge structures to assess various seismic performances and compare with results obtained using the original rheology model developed by Bhuiyan (2009)

ORIGINAL RHEOLOGY MODEL

The experimental investigations conducted by several authors (Bhuiyan, 2009; Bhuiyan et al., 2009; Hwang et al., 2002; Imai et al., 2010; Miehe and Keck, 2000) have revealed four different fundamental properties, which together characterize the typical overall response of laminated rubber bearings: (i) a dominating elastic ground stress response, which is characterized by large elastic strains (ii) a finite elasto-plastic response associated with relaxed equilibrium states (iii) a finite strain-rate dependent viscosity induced overstress, which is portrayed by relaxation tests, and finally (iv) a damage response within the first cycles, which induces considerable stress softening in the subsequent cycles. Considering the first three properties and motivated by the experimental observations, a visco-elastic-plastic rheology model for the HDRB and other bearing types was developed (Bhuiyan, 2009).



(a) (b)

Fig.1 Structural configuration of the rheology model (after Bhuiyan and Okui, 2012)

The model is the extended version of the Maxwell's model by adding two branches: one branch is the nonlinear elastic spring element and the other one is the elasto-plastic spring–slider elements (Fig.1). The elasto-plastic and the nonlinear elastic responses, which are represented in the top two branches of the model (Fig.1), constitute the rate-independent (equilibrium) hysteresis. The equilibrium hysteresis is identified from the relaxed equilibrium responses of the multi-step relaxations (MSR) of the bearings. The remaining part of the model is the rate-dependent hysteresis, which is identified from the simple relaxation (SR) and cyclic shear (CS) loading tests. The total stress response is phenomenologically decomposed into three components, which has been illustrated in Fig.1 (b), as:

$$\tau = \tau_{ep}(\gamma_a) + \tau_{ee}(\gamma) + \tau_{oe}(\gamma_c), \tag{1a}$$

$$\tau_{ep} = C_1 \gamma_a \ , \quad \text{with} \ \begin{cases} \dot{\gamma}_s \neq 0 & \text{for} \ \left|\tau_{ep}\right| = \tau_{cr} \\ \dot{\gamma}_s = 0 & \text{for} \ \left|\tau_{ep}\right| < \tau_{cr} \end{cases} \ , \tag{1b}$$

$$\tau_{ee} = C_2 \gamma + C_3 |\gamma|^m \operatorname{sgn}(\gamma) \ , \tag{1c}$$

$$\tau_{oe} = C_4 \gamma_c \ , \text{with} \ \ \tau_{oe} = A \left|\frac{\dot{\gamma}_d}{\dot{\gamma}_o}\right|^n \operatorname{sgn}(\dot{\gamma}_d) \ , \tag{1d}$$

$$\text{and, } A = \frac{1}{2}\left(A_l \exp(q|\gamma|) + A_u\right) + \frac{1}{2}\left(A_l \exp(q|\gamma|) - A_u\right)\tanh(\xi\tau_{oe}\gamma_d), \tag{1e}$$

where, τ_{ep} is the stress in the first branch composed of a spring (Element A) and a slider (Element S); τ_{ee} denotes the stress in the second branch with a spring (Eement B); τ_{oe} represents the stress in the third branch comprising a spring (Element C) and a dashpot (Element D). The first and second branches represent the rate-independent elasto-plastic behavior, while the third branch introduces the rate-dependent viscosity behaviour (Bhuiyan, 2009).

IMPROVED RHEOLOGY MODEL

Considering the first three aforementioned properties, a strain-rate dependent constitutive model (i.e., Improved Rheology Model) for the HDRBs was developed and verified for sinusoidal excitations. Eq. (2a) to Eq. (2d) provides the explicit expressions for the average shear stress t and strain g of the bearings.

$$\tau = \tau_{ep}(\gamma_a) + \tau_{ee}(\gamma) + \tau_{oe}(\gamma_c), \tag{2a}$$

$$\tau_{ep} = C_1\gamma_a, \quad \text{with} \begin{cases} \dot{\gamma}_s \neq 0 & \text{for } |\tau_{ep}| = \tau_{cr} \\ \dot{\gamma}_s = 0 & \text{for } |\tau_{ep}| < \tau_{cr} \end{cases}, \quad \text{and } \tau_{cr} = S_1 + S_2|\gamma_{\max}|, \tag{2b}$$

$$\tau_{ee} = C_2\gamma + C_3|\gamma|^m \operatorname{sgn}(\gamma), \tag{2c}$$

$$\tau_{oe} = C_4\gamma_c, \text{ with } \tau_{oe} = A\left|\frac{\dot{\gamma}_d}{\dot{\gamma}_o}\right|^n \operatorname{sgn}(\dot{\gamma}_d), \tag{2d}$$

$$\text{and, } A = \frac{1}{2}\left(A_l \exp(q|\gamma|) + A_u\right) + \frac{1}{2}\left(A_l \exp(q|\gamma|) - A_u\right)\tanh(\xi\tau_{oe}\gamma_d), \tag{2e}$$

where, C_i (i = 1 to 4), S_i (i = 1 to 2), τ_{cr}, m, A_l, A_u ,q n and ξ are the model parameters determined from a number of experiments and are listed in Table 1 for HDR1, HDR2 and HDR3 respectively. Here, all the parameters express the similar meaning as like the original rheology model (Bhuiyan, 2009) except the critical shear stress τ_{cr} possess the algebraic summation of S_1 and S_2 (Khan, 2014).

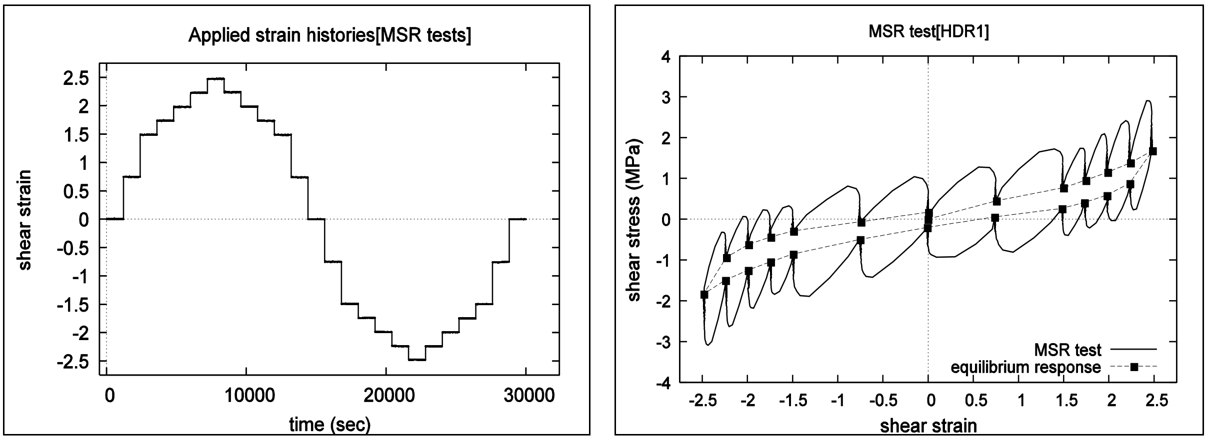
Table 1: Parameters for HDR1, HDR2 and HDR3

Parameter	Value (HDR1)	Value (HDR1)	Value (HDR1)
C_1 (MPa)	2.50	2.50	2.50
C_2 (MPa)	0.21	0.48	0.397
C_3 (MPa)	0.06121	0.00862	0.05301
C_4 (MPa)	2.50	3.25	2.80
m	2.94	5.17	3.11
A_l (MPa)	0.30	0.35	0.40
A_u (MPa)	0.20	0.27	0.24
q	0.532	0.344	0.353

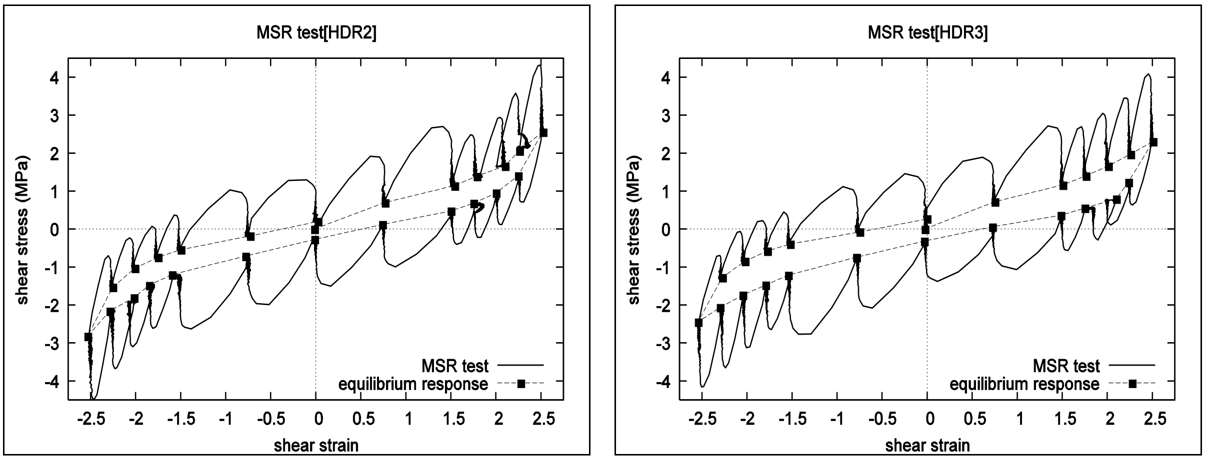
n	0.205	0.224	0.213
S_1 (MPa)	0.076	0.077	0.056
S_2 (MPa)	0.073	0.097	0.133
ξ	1.221	1.252	1.242

Experimental Characterization of High Damping Rubber Bearings

To characterizing the strain hardening features along with dependence of the equilibrium hysteresis on loading history of the bearings, multi-step relaxation (MSR) tests were carried out at maximum strain level of 2.50. The shear strain history applied in MSR test at 2.50 maximum strain level is presented in Fig.2 (a), where a number of relaxation periods of 20 min during which the applied strain is held constant are inserted in loading and unloading at a constant strain rate of 5.5/s. Fig.2 (b) to (d) present the results obtained in testing HDR1, HDR2 and HDR3 due to the stress history of MSR test with maximum strain level of 2.50. The equilibrium hysteresis effect is observed in the MSR test; however, the magnitudes were found to increase when increasing strain level with increased supply of energy. The results indicate strong hardening features to be present at higher strain levels. Moreover, a strong dependence of the equilibrium hysteresis on the past maximum strain level was also appeared in the test procedure. In addition, the equilibrium response was also found to be strongly dependent on the current strain values in all bearings. Based on this high strain level (i.e., 250%), it might be appealing to develop an improved rheology model for betterment in the understanding of seismic performance analysis of base-isolated bridge structures.



(a) (b)

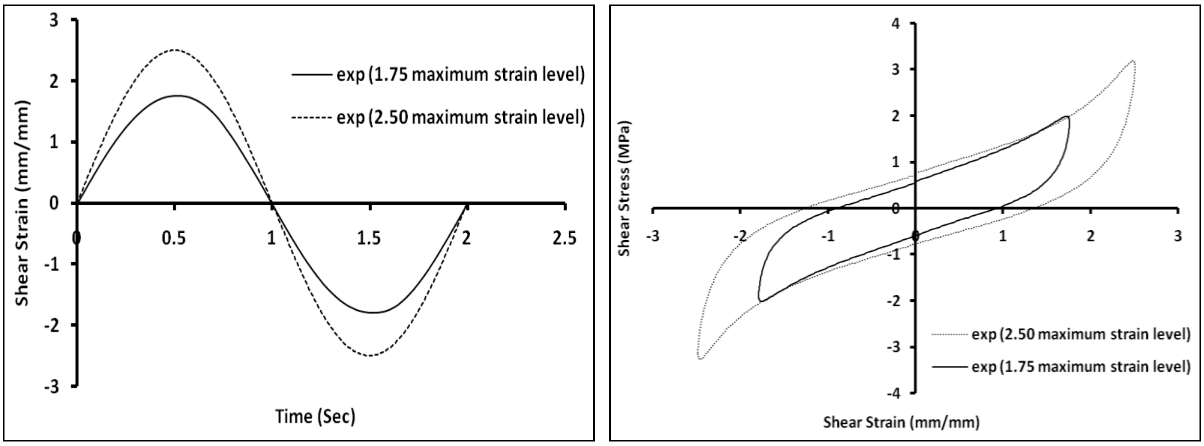


(c) (d)

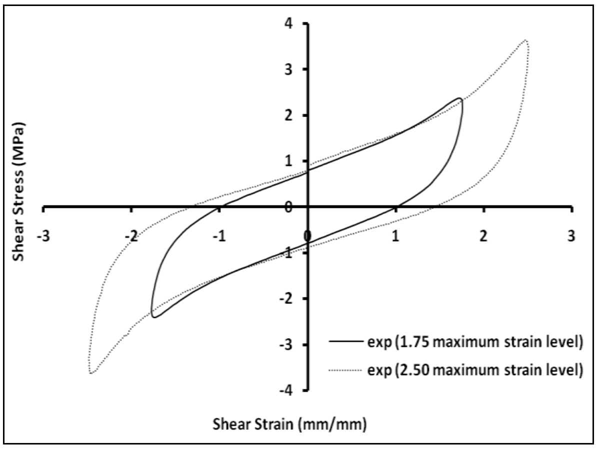
Fig.2 Applied strain histories and elasto-plastic response obtained from MSR test results of (b) HDR1, (c) HDR2 and (d) HDR3 at 2.50 maximum strain level

Mechanical Behaviour under Sinusoidal Excitation

A test scheme has been carried out in association with experimental shear stress-shear strain relationship for 1.75 and 2.50 maximum strain levels. Firstly, the applied strain histories are demonstrated (Fig.3 (a)). In order to remove the Mullin's softening effect from the shear stress – strain response of the bearing the 4th cycle stress responses are presented in the Figures. Fig.3 (b) to (d) presents the shear stress-strain relationship under sinusoidal excitations using the experimental data of HDR1, HDR2 and HDR3 respectively for 1.75 and 2.50 maximum strain level. From Fig.3 (b), at the upper limb, the first loading path of HDR1 for both 1.75 and 2.50 maximum strain level shows almost similar trends under sinusoidal excitations. But, the first unloading path of HDR1 for both 1.75 and 2.50 maximum strain level appears some differences under sinusoidal excitations. At the lower limb, the second loading path of HDR1 for both 1.75 and 2.50 maximum strain level shows almost similar trends under sinusoidal excitations. Consequently, the second unloading path of HDR1 for both 1.75 and 2.50 maximum strain level appears some differences under sinusoidal excitations. As like HDR1, nearly similar trends have been observed for HDR2 (Fig.3 (c)) and HDR3 (Fig.3 (d)).



(a) (b)



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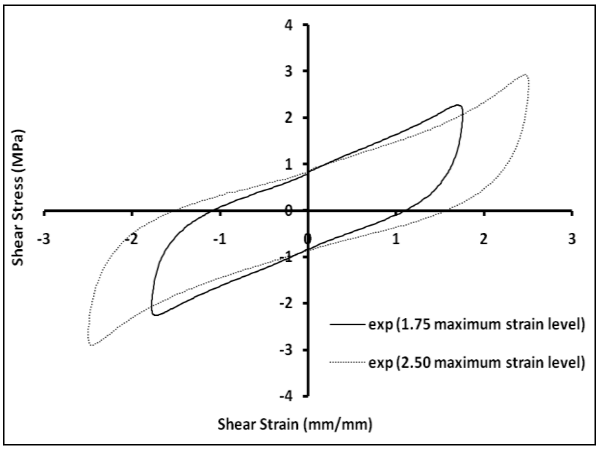
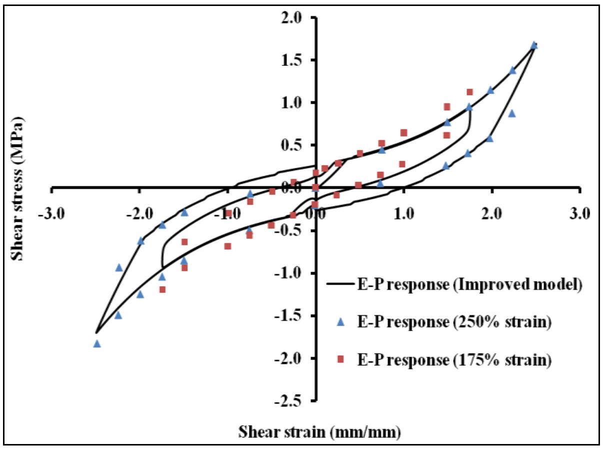


(d)

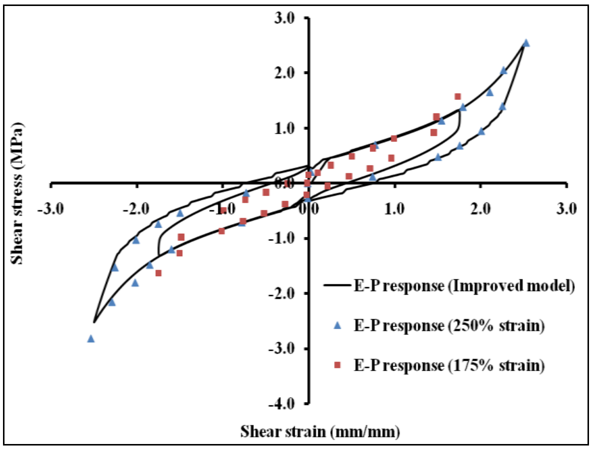
Fig.3 Applied strain histories and shear stress-shear strain relationship under sinusoidal excitations using the experimental data of (b) HDR1, (c) HDR2 and (d) HDR3 for 1.75 and 2.50 maximum strain level

Modelling of Equilibrium Hysteresis

From the MSR test data of the bearings (Bhuiyan, 2009) an equilibrium hysteresis loop with strain hardening is visible in each bearing. The equilibrium hysteresis is shown to be dependent on the maximum strain level used in the MSR test, as presented in Fig.4. The effect of maximum strain magnitudes considered in the MSR tests is significantly visible in Fig.4 especially in unloading part of the responses. However, the loading part of the responses shows no significant effect on the maximum strain level of the MSR test, i.e. the equilibrium response of the bearings is governed by current magnitudes of strain, not on the history of strain magnitudes (Khan, 2014).



(a)



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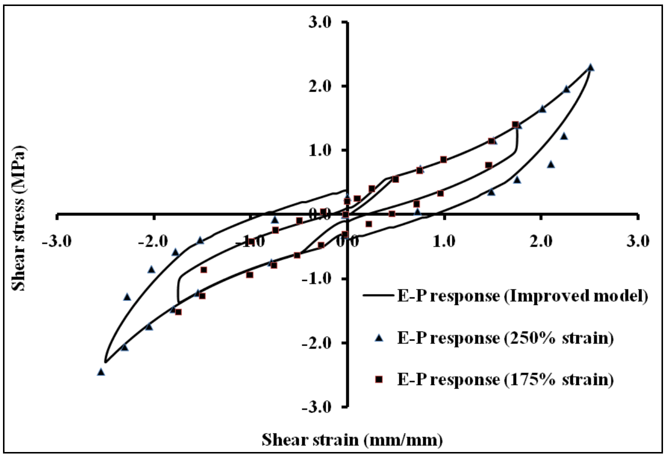


(c)

Fig.4 Equilibrium hysteresis and identification of equilibrium response parameters for (a) HDR1, (b) HDR2 and (c) HDR3; the experimental results are obtained from the MSR tests in asymptotic sense and the model results are determined using $\tau = \tau_{ee} + \tau_{ep} + \tau_{oe}$ with parameters given in Table 1

This equilibrium hysteresis loop can be suitably represented by combining the ideal elasto-plastic response and the nonlinear elastic response as motivated by the experiments (Fig.4). In order to determine the equilibrium response parameters as presented in Eq. (1b and 1c), the equilibrium hysteresis loops as obtained from the MSR test have been considered. The equilibrium hysteresis loops of the bearings considered in the study are presented in Fig.3. The critical shear, τ_{cr} is determined by using the equilibrium hysteresis loop. The difference between loading and unloading stresses in the equilibrium hysteresis loop at each strain level corresponds to $2\tau_{cr}$. Accordingly, τ_{cr} can be determined from the half of the arithmetic average values of the stress differences. The variation of τ_{cr} with the maximum strain level of MSR test is represented by Eq. (2b). The parameters S_1 and S_2 can be identified using a standard least square method. The parameter C_1 corresponding to the initial stiffness can then be determined by fitting the initial part as well as the switching parts from loading and unloading in the equilibrium hysteresis loop (see, for example, Fig.4). Finally, the parameters for the nonlinear spring (Element B) are identified. The subtraction of the stress τ_{ep} of Eq. (2b) from the equilibrium stress response obtained from the MSR test gives the stress τ_{ee} corresponding to Eq. (2c). Parameters C_2, C_3, and m are determined using a standard least square method. The equilibrium response parameters C_2, C_3, S_1, S_2 and m for the bearings are given in Table 1. The equilibrium responses obtained using the proposed model and the identified parameters are presented in Fig.4. The solid line in each figure shows the equilibrium responses obtained by the improved rheology model.

SIMULATION FOR SINUSOIDAL EXCITATIONS

Simulation for sinusoidal excitation and comparing with experimental results (i.e., basic test) offers some potential benefits to check the adequacy of the proposed rheology model. Fig.5 present strain histories of the sinusoidal excitations were applied for numerical simulation.

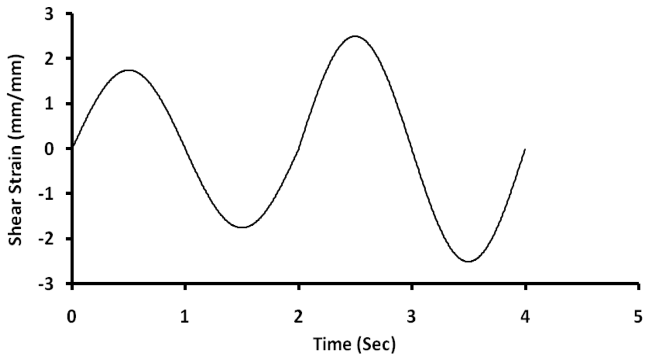
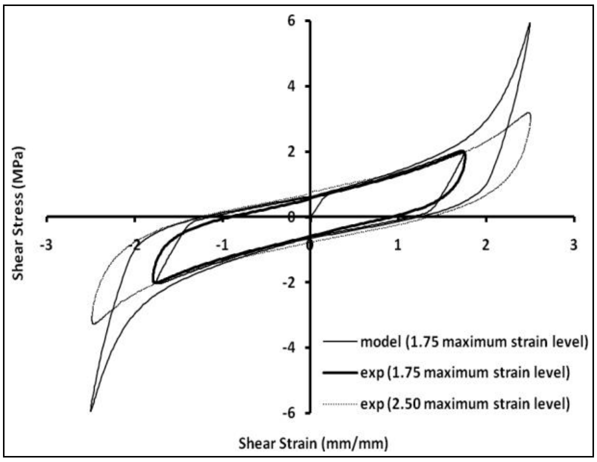
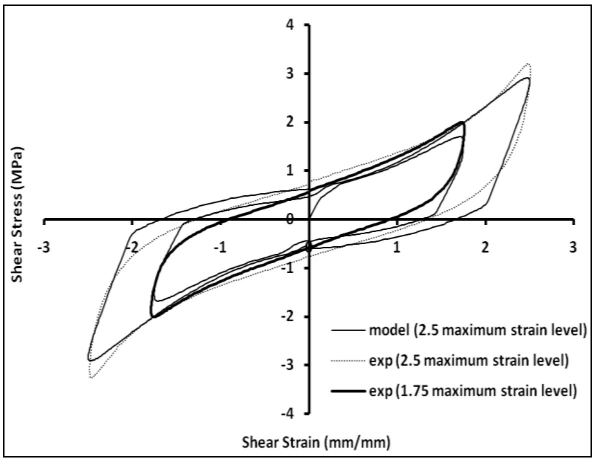


Fig.5 Sinusoidal Strain histories for numerical simulation



(a)



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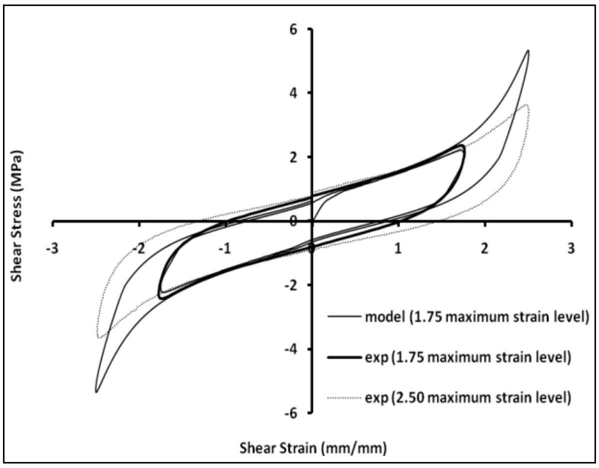

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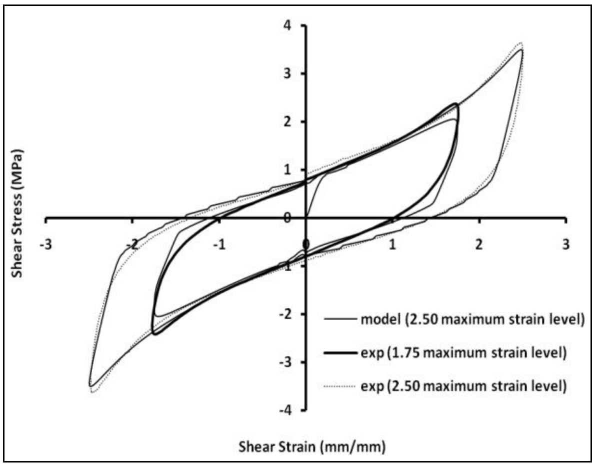

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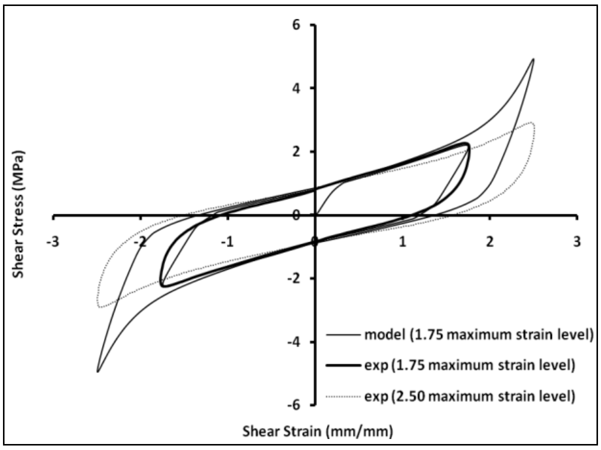

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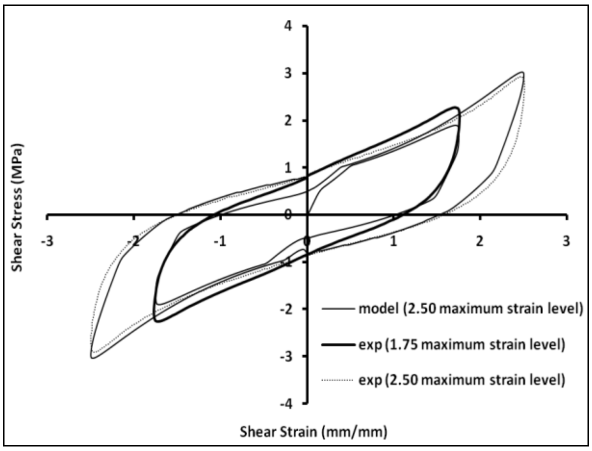


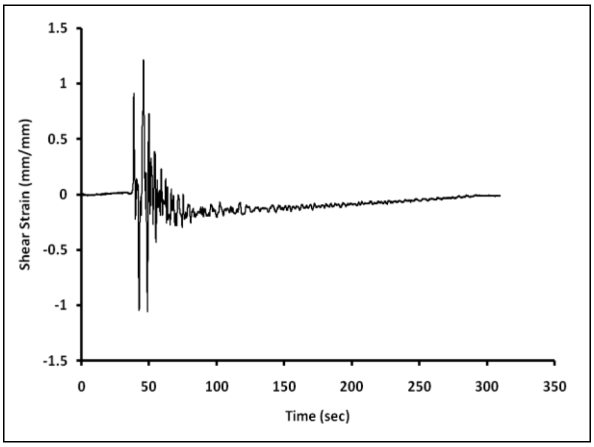
(f)

Fig.6 Numerical simulation for sinusoidal excitations using the original and Improved rheology model for HDR1 (a & b), HDR2 (c & d), and HDR3 (e & f); for clarity the results were compared with experimental outcomes at 1.75 and 2.50 maximum shear strain levels

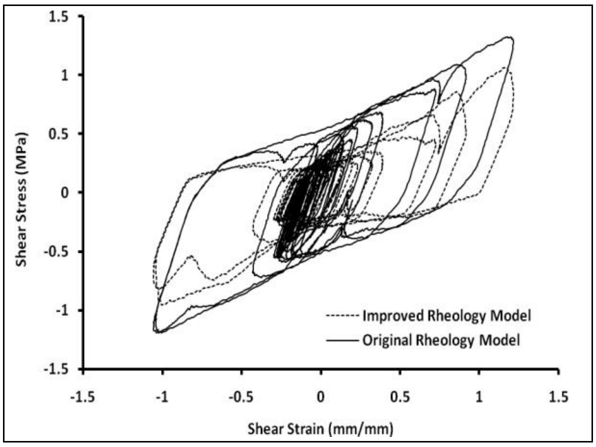
Fig.6 (a) shows the numerical simulation for sinusoidal excitation using the original rheology model for HDR1 under 1.75 maximum shear strain level and compared with experimental results of 1.75 and 2.50 maximum shear strain levels. In comparison with experimental model, the original rheology model was not adequately agreed for high damping rubber bearing (HDR1). On the other hand, Fig.6 (b) shows the numerical simulation for sinusoidal excitation using the improved rheology model for HDR1 under 2.50 maximum shear strain level and compared with experimental results of 1.75 and 2.50 maximum shear strain levels. The improved rheology model was satisfactorily matched with the experimental outcomes of high damping rubber bearing (HDR1). As like HDR1, almost similar trends have been observed for high damping rubber bearing specimens HDR2 (Fig. 6 (c & d)) and HDR3 (Fig. 6 (e & f)) respectively (Khan, 2014).

RESPONSE PREDICTION FOR RANDOM DISPLACEMENT HISTORY

After diagnosis the adequacy of the proposed rheology model, prediction of seismic responses for a synthesized earthquake ground motion make it more reliable for further experimental implementation. A synthesized earthquake excitation was considered (Fig.7 (a)). Fig.7 (b) to (d) postulates the shear stress-shear strain relationship of high damping rubber bearing (HDR1, HDR2 and HDR3) for improved and original rheology model (Bhuiyan, 2009) when subjected to the synthesized seismic excitation. A test scheme has been carried out in association with numerical stress-strain relationship for improved rhology model (2.50 maximum strain level) and original rheology model (1.75 maximum strain level) for comparison. The stress-strain relationship of HDRBs for improved and original rheology model may obtained by using the Eq. (2b) to (2d). A nearly similar graphical trend was observed by high damping rubber bearings (HDR1, HDR2 and HDR3) for both improved and original rheology model. Fig.7 (b) to (d) portrayed a clear differentiation in between improved and original rheology model. The improved rheology model becomes more spontaneous to measuring the actual stress-strain relationship than the original rheology model and is more enthusiastic for high strain level. From the previous studies, in some extent of high strain level, it has been revealed that, original rheology model may pursue an overestimated route at shear stress-strain relationship. But, the proposed improved rheology model may successfully overcome this flaw and tend to produce the stress-strain relationship that is satisfactorily nearest to the optimal outcomes (Khan, 2014).



(a)



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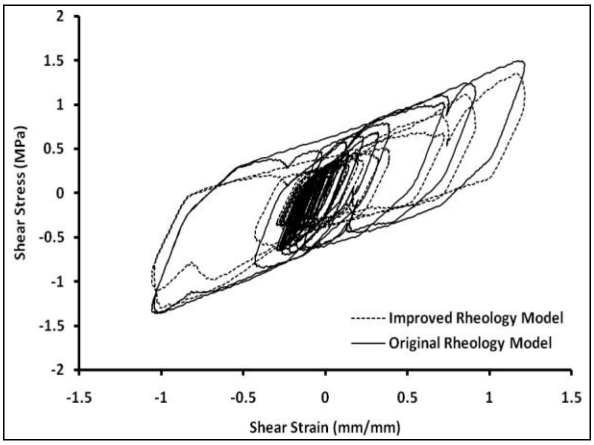

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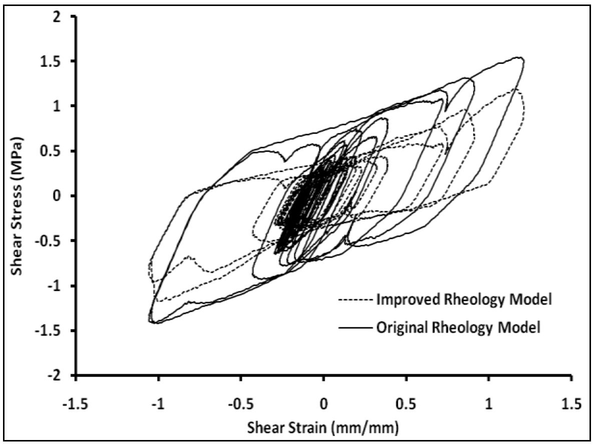

(d)

Fig.7 Applied strain history for a synthesized earthquake and shear stress-strain relationship for improved and original rheology model due to the synthesized earthquake ground motion applied on (b) HDR1, (c) HDR2 and (d) HDR3

CONCLUDING REMARKS

The mechanical behaviour of three high damping rubber bearings as obtained in the multi-step relaxation and cyclic shear tests were investigated and employed in the development the improved rheology model. The motivation of the modification in the rheology structure of the model came from the multi-step relaxation experiments of the high damping rubber bearings which explores that the equilibrium hysteresis of the bearings not only depends on the current shear deformation but also on the past shear deformation experienced by the bearings. Motivated by the experimental results of the bearings, the current rheology model has been modified by incorporating the effect of past shear deformation. The adequacy of the improved rheology model of the bearings has been verified by sinusoidal excitations. Finally, the prediction capability of the improved rheology model of the bearings has been demonstrated for a synthesized earthquake.

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APPLICATION OF FRP MATERIALS FOR STRENGTHENING OF RC STRUCTURAL MEMBERS

F. H. Chowdhury[1*] & G. M. S. Islam[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong – 4349, Bangladesh.*
**Corresponding Author, <fhchy@yahoo.com>*

ABSTRACT

Strengthening of reinforced concrete structures using fiber-reinforced polymer (FRP) is an advanced technology. Repairing of a RC structure is often preferred than demolishing considering economy and environment. The use of FRP for strengthening of concrete structures is a popular method at present and it has gained attention during last two decades. The main types of FRP materials include CFRP, AFRP and GFRP. Based on their properties and shapes, numerous strengthening systems are introduced with respect to their suitability in various applications. This paper renders a succinct review of the properties of commercially available FRP materials and their suitable applications for strengthening of RC structural members. It also includes comparison among performance of FRP materials while applying for same purpose.

Keywords: Strengthening, Fiber Reinforced Polymer, CFRP, GFRP, RC structure.

INTRODUCTION

The application of fiber reinforced polymer (FRP) composites for strengthening of concrete structures was first introduced as an alternative of steel plate bonding for strengthening of beam at the Swiss Federal Laboratory for Materials Testing and Research (EMPA) but test of RC beam strengthened using CFRP plates started at 1984 (Teng *et al*., 2003). Strengthening of concrete structures using externally bonded fiber reinforced polymer composites has proven its efficiency through many laboratory tests and field applications implemented in the last two decades. This technology may be needed when it require to carry additional loads or to meet new standards. Strengthening of existing concrete structures may require carrying higher design loads, improvement for strength loss due to deterioration, poor initial design and construction deficiencies or due to lack of maintenance (FIB Bulletin 14, 2001). Altering the purpose of use (e.g. ready-made garments industry) also requires strengthening of exiting of current structures. This concern rose after the recent collapse of RANA PLAZA at Bangladesh (The Daily Star, April 25, 2013). Environmentally induced degradation or earthquakes can also be the reason. Comparing to other solution to strengthen reinforced structures which are either in poor state or to withstand greater loads than initially designed, fiber reinforced polymer is effective, reasonable and time saving alternative (Dumas, 2012)

MATERIAL FOR STRENGTHENING

The most widely used FRP composites for external bonding is glass-fiber-reinforced polymer (GFRP) composites, carbon-fiber-reinforced polymer (CFRP) composites, and aramid fiber reinforced polymer (AFRP) composites. GFRP, CFRP and AFRP composites have all been used in practical applications and in research, but the first two appear to have been much more widely used in strengthening works. CFRP composites have superior properties to GFRP composites, but also significantly costlier (Teng. *et al.*, 2003). Researches have been carried out using CFRP and GFRP mostly in the past years. This is may be due to the low density and relatively low compressive yield strength of AFRP whereas GFRP is very much suitable when compared it comes to compressive strength, density and shear properties. Cost, and in some cases service temperature or durability factors, restrict the use of AFRP to specific applications (ACI 440-96). Table 1 shows comparison of properties between the FRP composites. In terms of structural use, one property is commonly important to all three types of FRPs is that their stress–strain behaviour is linearly elastic until rupture. The figure below shows a comparison with steel, typical stress-strain diagrams for unidirectional composites under short-term monotonic loading.

Table 1. Comparison between physical properties of various FRP

Unidirectional advanced composite materials	**Fibre content, % by weight**	**Density, kg/m^3**	**Longitudinal tensile modulus, GPa**	**Tensile strength, MPa**	**Cost, \$/m^2**
Carbon/epoxy CFRP laminate	65-75	1600-1900	120-250	1200-2250	10-500
Glass fibre/polyester GFRP laminate	50-80	1600-2000	20-55	400-1800	1-5
Aramid/epoxy , AFRP laminate	60-70	1200-1400	40-125	1000-1800	10-50

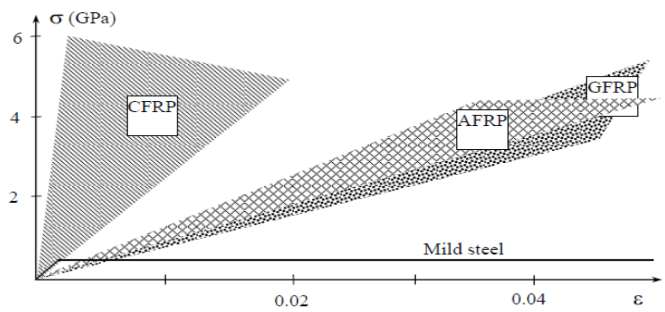

Fig. 1 Uniaxial tension stress-strain diagrams for different unidirectional FRPs and steel (FIB, 2001)

METHODS OF APPLICATION

Mainly two methods are used for external bonding of FRP composites have been widely used in strengthening RC structures. These include 1. wet lay-up system; and 2. precured system.

Wet lay-up system

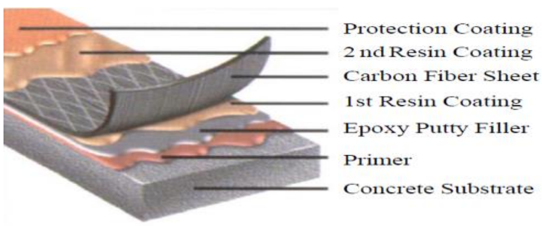

Fig. 2 Step by step installation-wet layup system (Saito and Kobayashi, 2000)

The wet lay-up method is the in situ application of epoxy resin to dry unidirectional or multidirectional fiber sheets or fabrics along with the compatible primer and putty, bonds the FRP sheets to the concrete surface.

Precured system

Precured FRP systems consist of a wide variety of composite shapes (such as laminate) manufactured off site. Typically, an adhesive, along with the primer and putty, is used to bond the precured shapes to the concrete surface. For installation procedures manufacturer guidelines should be followed.

STRENGTHENING OF RC STRUCTURES

Different types of external bonding of FRP have been applied over the years considering the type of strengthening needed in the structural members of a RC structure. The strengthening techniques for different structural members are described in the following section.

Beam Strengthening

Flexural strengthening of reinforced concrete beams can be done by external bonding of FRP composites using unidirectional sheets and laminates. The bond between FRP and concrete surface has to be ensured to improve flexural strength and stiffness so that premature debonding failure can be avoided. Various experimental and analytical studies (Norris *et al*., 1997; Rahimi and Hutchinson, 2001; Esfahani *et al.,* 2007;; Attari *et al.,* 2012) have been carried out all over the world on flexural strengthening of concrete beams using FRP composites. Purpose of these studies was to assess the efficiency of flexural performance of concrete, inspect the effect of various parameters on probable failure modes and also to compare the strengthening difference between different types of FRP composites. Flexural Strengthening using CFRP and GFRP plates have demonstrated that gain of ultimate strength is 140% and 106% respectively (Rahimi and Hutchinson, 2001), although the thickness and the area of the GFRP plate was 1.5 times greater than CFRP plates. Using CFRP and GFRP sheets in beam strengthening has shown an increase of 114% and 118% respectively (Attari *et al.* 2012). GFRP sheet was used in 2 layers whereas CFRP was applied only one layer.

Fig. 3 GFRP fabric to strengthen deteriorated reinforced concrete T-beam

Fig. 4 Flexural Strengthening of Beam using CFRP sheet

Application of anchoring system can enhance the strength and ductility of the beam by confining the concrete. Shear enhancement of the beam can also be applied either by applying vertical U (0/90 deg) or inclined L FRP strips (45/135 deg). Experimental studies has shown that shear enhancement of RC beam were 37.28 % and 138.41% respectively (Jayaprakash *et al.*, 2008). Therefore, specimens with inclined L-CFRP strips (45/135 deg) attained a shear enhancement of 42% greater in comparison of specimen with vertical U-Strips (0/90 deg) (Jayaprakash *et al* .2008). On the other hand inclined L-GFRP strips (45/135 deg) showed 50% enhancement of shear strength (Sundarraja and Rajamohan, 2009).

Column strengthening

Columns are the most important load carrying component of a building. Therefore, columns are considered as the most susceptible part of a typical RC structure. Since the beams permit more effective energy dissipation, it is very vital strengthening the columns to create a *strong column and weak beam system*, so that plastic hinges are formed in the beams. Considering earthquakes columns should be designed correctly to avoid a soft story failure of a RC structure. A column can fail in three ways after an earthquake Such as a) shear failure; b) plastic hinge failure; and c) lap splice failure.

Researches in have shown that, shear strength of an upgrade column can be increased proportionally by increasing the thickness or layers of FRP jacket (Kumutha *et al.* 2007). Experimental studies have shown that properly strengthened RC column can update shear strength to some extent that brittle failure can be altered into inelastic flexural deformation as well as increase ductility. Energy dissipation capacity and ductility of a column can be improved using a single layer of CFRP sheet (Kumutha *et al.* 2007). Insufficient length of lap splice can cause breaking of the bond during seismic activity and lead to lap splice failure.

Harajli *et al*. (2006) summarized the studies on the efficiency of FRP in retrofitting of circular, square and rectangular reinforced concrete columns. Noteworthy development in axial strength and ductility is resulted when rectangular column is confined with FRP composites (Haralji *et al*. 2006). For square column sections without longitudinal reinforcement (plain concrete) the increase in axial strength was found to be 154, 213, and 230% for one, two, or three layers of CFRP wraps, respectively. For square steel reinforced concrete columns, the increase in axial strength was 188, 255 and 310% with one, two or three layers of CFRP wraps, respectively (Haralji et al. 2006). For square and rectangular steel reinforced concrete columns, the increase in axial strength resulting from FRP retrofitting scheme is 4.05 &16.22% and 2.93 & 22.67% with one and two layers of GFRP wraps (Kumutha *et al.* 2007). However, experiments showed that FRP confinement becomes less significant as the aspect ratio of the column section increases.

Fig. 5 Column confinement using carbon fiber sheet

Fig. 6 Column strengthening using glass fiber sheet

Slab Strengthening

Importance of slab cannot be overstated. It transfers load to the adjoining beam and then to column. Failure of slab is mainly flexure in the middle and shear failure is seen in the corner or in the joint. Experimental and analytical studies (Mosallam and Mosalam, 2003; Soudki et al. 2012; and Anil et al. 2013) have shown that load carrying capacity of both one way and two way slab can be increased more than its initial capacity, using FRP systems. Mosallam and Mosalam, (2003) reported that CFRP laminates results an appreciable upgrade of structural capacity of the unreinforced and reinforced slab up to 500% and 200% respectively.

Fig. 7 Slab strengthening using CFRP laminates

Fig. 8 Strengthening of Beam-Column joint using CFRP

GFRP laminates were also used in two particular specimens, with increase of layers, 1.5 times with respect to CFRP laminates. GFRP system also showed debonding failure, which was not present in CFRP system. Low modulus and its requirement of several layers to fulfil creep rupture condition prevent its use. Another study by Anil *et al*. (2013) showed that use of CFRP strips increased in load carrying capacity up to 1.16 and 1.48 times for the shear and flexural failure respectively.

Strengthening beam-column joints and slab-column joints

Beam column joint retrofitting is another essential phase of strengthening a RC structure. This is needed due to inadequate seismic performance of a RC structure. Confinement and wrapping of RC concrete columns using FRP material helps to promote the formation of plastic hinges in the beam area and thus creates a more acceptable ductility and energy dissipation during an earthquake. Experimental study showed that (El-Amoury and Ghobarah, 2002) use of GFRP sheet in beam-column joint specimen strengthening showed 3 and 6 times more energy dissipation than un-strengthened specimen. Experiments showed that CFRP sheets significantly reduced slab deflection and while increasing both the yielding load and the ultimate load carrying capacity of the slab–column connection. Results showed that, after strengthening, experimental ultimate load were increased by 33%, 40%, and 67% for specimens with central, eccentric and edge columns respectively. However specimen ductility was reduced about 20% for central column where specimen with edge or eccentric column showed 40% of reduction (El-Enein *et al.* 2014). Application of bidirectional fabrics brings more suitability when applied in the joints and connections.

CONCLUSIONS

This paper renders a succinct review of the properties of commercially available FRP materials and their suitable applications for strengthening of RC structural members based on the existing research being carried. Experimental and analytical studies on flexural and shear strengthening of beams, flexural strengthening of slabs, and strengthening of columns subject to both static and seismic loads have been covered. Since the introduction of CFRP materials, its use has been increasing rapidly. Different forms of FRP materials are also being introduced. However, use of CFRP is quite extensive than the other two. GFRP has gained less interest due to its low tensile modulus, low humidity and requirement of several layers where CFRP can meet these need. Use of AFRP is reduced due to its low compressive strength, also difficulties in cutting and machining compared to CFRP and GFRP. Although CFRP has all its superiority over other two FRP composites, it requires high energy during production and price can also be an issue sometime. Based on the researches and studies it can be concluded that GFRP can replace in some application by increasing its layer and proper application. Design guidelines and recommendation should be more precise regarding the use of GFRP in that case. In the developing and under developed country use of FRP system is not quite popular yet. If regaining higher strength of a RC structure is not but meeting its initial strength is the main objective, GFRP can be used by following proper instructions. Several layers of GFRP can ensure better safety. However, structures which require additional strength to withstand additional loading CFRP composites should be chosen. Overall, these strengthening techniques can be a potential method for saving existing ready-made garments industry buildings which does not meet the loading criteria specified by national/international building codes.

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VIBRATION CONTROL OF STRUCTURES WITH MULTIPLE TUNED MASS DAMPERS FOR DIFFERENT EARTHQUAKE CONSIDERING DIFFERENT TYPES OF SOIL

R. Debbarma[1*] & S. Hazari[1]

[1] *Department of Civil Engineering, National Institute of Technology, Agartala, Tripua, India*
**Corresponding Author, <ramadebbarma@gmail.com>*

ABSTRACT

The effectiveness of multiple tuned mass dampers (MTMD) to mitigate the vibration of structures under different earthquake peak ground acceleration considering different types of soil is investigated in this paper. In such cases, the stochastic earthquake response quantities with MTMD, single tuned mass damper (STMD) and without dampers are obtained in random vibration framework using state space formulation to study the possible improvement of performance of MTMD system with STMD. The parametric study is conducted on the effectiveness and robustness of MTMDs for different earthquakes considering different types of soil in comparison with STMD. It is investigated that the MTMDs (when 5 Nos.) configuration is more effective to controlling the seismic motion of the primary system. Numerical study is performed to evaluate the effectiveness of MTMDs and system performance.

Keywords: multiple tuned mass dampers, different peak ground acceleration, different types of soil, parametric study, vibration control.

INTRODUCTION

In recent years, the construction of lightly damped, flexible tall building by using high strength materials in regions of seismic risk has created concern in the structural engineering community. In recognition of the serviceability issues, structural engineering researchers have created artificial passive vibration control devices. Tuned mass damper is the oldest passive vibration control device. In dynamic vibration control of structures, the tuned mass damper (TMD) has been installed as an effective passive control device to mitigate the structural vibration. A TMD is a passive vibration control device consisting of a mass, damping, and a spring; it is attached to a main building structure to reduce any undesirable vibrations induced by earthquake loads. The natural frequency of the TMD is tuned in resonance with the fundamental mode of the building structure, so that the huge amount of the structural vibrating energy is transferred to the TMD and dissipated by the damping as the building structure is subjected to earthquake loads. Multiple tuned mass dampers are more successful passive vibration control system. In these systems, MTMDs are tuned to several modes of structure vibration. In this present study, the rms displacement of primary system is obtained with various mass ratios by using MTMD for different earthquake peak ground acceleration considering different types of soil (hard, medium and soft) compared with STMD. Bergman et al. (1989, 1991) investigated the performance of MTMDS, spatially distributed in a primary structure. The application of MTMD for single degree system have been studied by (Xu and Igusa,1992, Igusa and Xu,1994). It has been demonstrated that MTMD with distributed natural frequencies are more effective than a single TMD. The effectiveness and robustness of MTMD under dynamic load were studied by Yamaguchi and Harnpornchai (1993), Abe and Fujino(1994), Kareem and Kline (1995), Jangid (1995),(Lewandowski and Grzymislawska,2009). The present paper deals the effectiveness and robustness of MTMD to controlling the vibration of structure under random earthquake loads. A parametric study is conducted to investigate the performance of MTMD in comparison with single TMD using state space formulation. A numerical example is taken to evaluate the effectiveness of MTMDs (3 Nos. and 5 Nos.) considering several parameters under random earthquake loadings

THEORITICAL FORMULATION

Description of MTMD system:

The aim of designing MTMD is to tune damper parameters to the fundamental mode of vibration. It means that the natural damper frequency (or a group of dampers) ω_d must be close to the natural frequency of fundamental vibration mode of structure $(\omega_d \approx \omega_s)$. Moreover, the damping coefficient of the damper must be appropriately chosen by (Zuo and Nayfeh, 2005) and c_j is obtained using equations developed by Den Hartog (1956) for the SDOF damper.

The optimum parameters of such a damper (or group of MTMD) can be obtained from the formulae given in a paper (Warburton 1982). The optimal frequency ratio is determined from:

$$\frac{\omega_d^2}{\omega_s^2} = \frac{2+\mu}{2(1+\mu)^2} \tag{1}$$

Where, $$\mu = \frac{\sum_{j=1}^{n} m_j}{m_s},\ \omega_s^2 = \frac{k_s}{m_s},\ \omega_d^2 = \frac{k_d}{m_d}, m_d = \sum_{j=1}^{n} m_j \tag{2}$$

The Equation Of Motion Of Structure And MTMD System:

The equation of motion of a sdof system attached with MTMD (as shown in Fig.1) can be expressed as,

$$\tilde{\mathbf{M}}\ddot{\mathbf{Y}} + \tilde{\mathbf{C}}\dot{\mathbf{Y}} + \tilde{\mathbf{K}}\mathbf{Y} = -\tilde{\mathbf{M}}\bar{\mathbf{r}}\ddot{z}_b \tag{3}$$

Where, $\mathbf{Y} = [x_s, x_1, x_2, ..., x_n]^T$ is the relative displacement vector, and $\bar{\mathbf{r}} = [0 \quad \mathbf{I}]^T$, where $\mathbf{I}$ is an nx1 unit vector. $\tilde{M}$, $\tilde{C}$ and $\tilde{K}$ represent the mass, damping and stiffness matrix of the combined system.

$$\tilde{M} = \begin{bmatrix} M_s & 0 \\ 0 & m \end{bmatrix} \tag{4}$$

Where, M_s is the mass of the structure and m is the matrix of dampers.

$$m = \text{diag}[m_1, \quad m_2, \quad m_3, \quad m_4, \quad m_5 \quad .... \quad m_n]$$

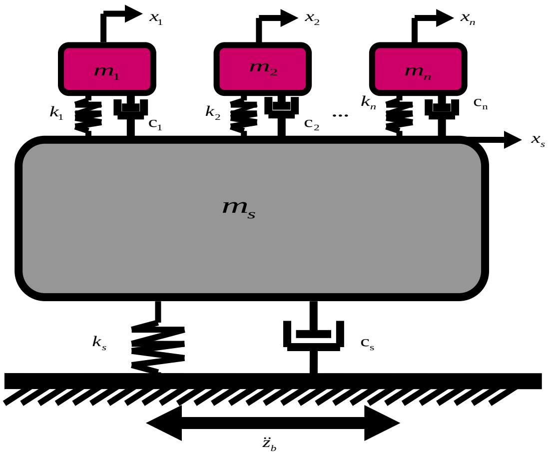


Fig.1 Structure-MTMD system

The stiffness matrix $\tilde{K}$ of the considered system can be written in the block form below,

$$\tilde{K} = \begin{bmatrix} K_s + k_d & k^* \\ k^{*T} & k \end{bmatrix} \quad (5)$$

Where, K_s is the stiffness of structure.

$$k_d = \sum_{j=1}^{n} k_j \quad , \quad k^* = \begin{bmatrix} -k_1 & -k_2 & -k_3 & -k_4 & -k_5 & \dots & -k_n \end{bmatrix}$$

$$k = \mathrm{diag}\begin{bmatrix} k_1, & k_2, & k_3, & k_4, & k_5, & \dots & k_n \end{bmatrix}$$

The damping matrix of the system $\tilde{C}$ is in a form similar to that of the stiffness matrix $\tilde{K}$. The specific blocks of this matrix are shown below:

$$\tilde{C} = \begin{bmatrix} C_s + c_d & c^* \\ c^{*T} & c \end{bmatrix} \quad (6)$$

Where, C_S is the damping of the structure.

Where, $c_d = \sum_{j=1}^{n} c_j \qquad c^* = \begin{bmatrix} -c_1 & -c_2 & -c_3 & -c_4 & -c_5 & \dots & -c_n \end{bmatrix}$

$$\frac{c_j}{2\sqrt{m_j k_j}} = \sqrt{\frac{3\mu_j}{8(1+\mu_j)}}, \quad \xi j = \frac{c_j}{2m_j\omega_j}, \quad c = \mathrm{diag}[c_1, \quad c_2, \quad c_3, \quad c_4, \quad c_5, \quad \ldots \quad c_n]$$

Introducing the state space vector, $\mathbf{Y}_s = (x_s, x_1, x_2, \ldots. x_n \dot{x}_s, \dot{x}_1, \dot{x}_2 \ldots. \dot{x}_n)^T$, Equation (3) can be written as,

$$\dot{\mathbf{Y}}_s = \mathbf{A_s Y}_s + \tilde{\mathbf{r}}\,\ddot{z}_b \text{ where, } \mathbf{A}_s = \begin{bmatrix} 0 & \mathrm{I} \\ \mathbf{H}_k & \mathbf{H}_c \end{bmatrix} \tag{7}$$

Where, $\mathbf{H}_k = \mathbf{M}^{-1}\mathbf{K}$, $\mathbf{H}_c = \mathbf{M}^{-1}\mathbf{C}$

In which $\tilde{\mathbf{r}} = [0, \mathbf{I}]^T$ with **I** and 0 is the (n+1)x(n+1) unit and null matrices, respectively

Response Covariance Analysis:

The structure -MTMD system as shown in Fig.1 is subjected to stochastic load due to the random seismic acceleration that excites the primary structure at base. A widely adopted stationary model of $\ddot{z}_b(t)$ is obtained by filtering a white noise process acting at the bed rock through a linear filter which represents the surface ground. This is the well-known Kanai– Tajimi stochastic process [Tajimi 1960] which is able to characterize the input frequency content for a wide range of practical situations. The process of excitation at the base can be described as:

$$\ddot{x}_f(t) + 2\xi_f\omega_f\dot{x}_f + \omega_f^2 x_f = -\omega(t) \text{ and } \ddot{z}_b(t) = \ddot{x}_f(t) + \omega(t) = 2\xi_f\omega_f\dot{x}_f + \omega_f^2 x_f \tag{8}$$

Where, $\omega(t)$ is a stationary Gaussian zero mean white noise process, representing the excitation at the bed rock, ω_f is the base filter frequency and ξ_f is the filter or ground damping. Defining the global state space vector is defined as: $Z = (x_s, x_1, x_2, \ldots. x_n, x_f \dot{x}_s, \dot{x}_1, \dot{x}_2, \ldots. \dot{x}_n, \dot{x}_f)^T$, Eqn. (7) and (8) leads to an algebraic matrix equation of order six i.e. the so called Lyapunov equation (Lutes and Sarkani 1997):

$$\mathbf{AR} + \mathbf{RA}^T + \mathbf{B} = 0 \tag{9}$$

The details of the state space matrix **A** and **B** in Eqn. (9) are as below:

$$[\mathbf{A}] = \begin{bmatrix} 0 & \mathbf{I} \\ \bar{\mathbf{H}}_\mathbf{k} & \bar{\mathbf{H}}_\mathbf{c} \end{bmatrix} \tag{10}$$

Where,

$$\bar{\mathbf{H}}_{\mathbf{k}} = \begin{bmatrix} -\mathbf{M}^{-1}\mathbf{K} & \begin{matrix} \vdots & \omega_f^2 \\ \vdots & \omega_f^2 \\ \vdots & \cdot \\ \vdots & \omega_f^2 \end{matrix} \\ \cdots\ \cdots\ \cdots\ \cdots\ \cdots & \cdots \\ 0\ \ 0\ \ \ldots\ \ 0 & \vdots\ \ -\omega_f^2 \end{bmatrix} \quad \bar{\mathbf{H}}_{\mathbf{c}} = \begin{bmatrix} -\mathbf{M}^{-1}\mathbf{C} & \begin{matrix} \vdots & 2\xi_f\omega_f \\ \vdots & 2\xi_f\omega_f \\ \vdots & \cdot \\ \vdots & 2\xi_f\omega_f \end{matrix} \\ \cdots\ \cdots\ \cdots\ \cdots\ \cdots & \cdots \\ 0\ \ 0\ \ 0\ \ 0 & \vdots\ \ -2\xi_f\omega_f \end{bmatrix} \quad \mathbf{B} = \begin{bmatrix} 0 & \cdots & \cdots & \cdots & \cdots & 0 \\ \vdots & \cdots & \cdots & \cdots & \cdots & \vdots \\ \vdots & \cdots & \cdots & \cdots & \cdots & \vdots \\ \vdots & \cdots & \cdots & \cdots & \cdots & \vdots \\ \vdots & \cdots & \cdots & \cdots & \cdots & \vdots \\ 0 & \cdots & \cdots & \cdots & \cdots & 2\pi S_o \end{bmatrix} \quad (11)$$

The space state covariance matrix $\mathbf{R}$ is obtained as the solution of the Lyapunov equation. The state space covariance matrix is represented by the sub-matrices $R_{zz}, R_{z\dot{z}}, R_{\dot{z}z}$ and $R_{\dot{z}\dot{z}}$. The root mean square (rms) displacement of liquid and the primary system can be then obtained as:

$$\sigma_{x_S} = \sqrt{R_{ZZ}(1,1)} \text{ and } \sigma_i = \sqrt{R_{ZZ}(i,i)}, \text{ where i=2 to n}$$

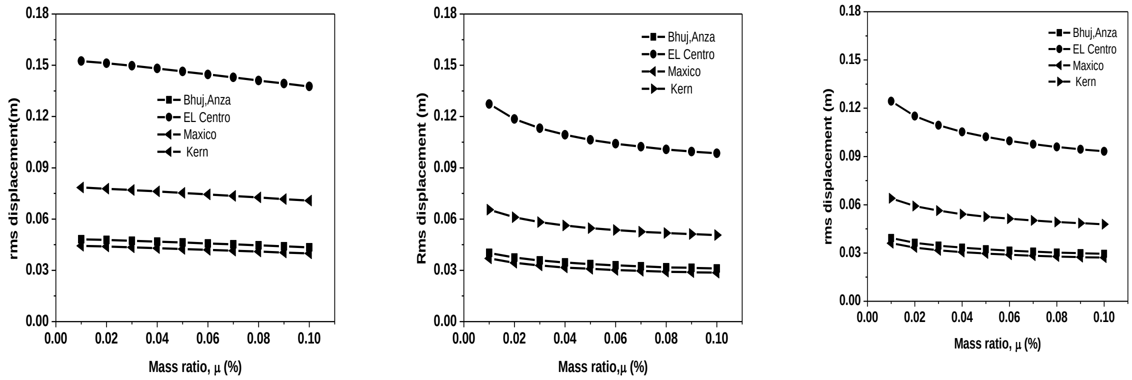
NUMERICAL STUDY

A single degree of freedom primary system with an attached MTMD as shown in Fig.1 subjected to stochastic earthquake excitation is undertaken to study the effectiveness and robustness of the MTMD considering uniform and non uniform mass distribution system. The primary system has the following mass and stiffness values: m_s=2.5×10^6 kg; k_1=1.0×10^7N/m. Unless mentioned otherwise, following nominal values are assumed for various parameters: structural damping, $\xi_s = 3\%$, mass ratio, μ=5%. The PSD of the white noise process at bed rock, S_0 is related to the standard deviation $\ddot{\sigma}_z$ of ground acceleration (Crandall and Mark) by: $S_0 = \dfrac{2\xi_f \sigma_{\ddot{z}_b}^2}{\pi\left(1+4\xi_f^2\right)\omega_f}$. For numerical study, the peak ground acceleration is taken as, for different earthquake as shown in Table 1, where 'g' is the acceleration due to gravity. It is assumed that $PGA = 3\ddot{\sigma}_{z_b}$. The mean value of ground motion filtered frequency and damping are taken for hard $(\omega_f = 2.5 \text{ Hz}, \xi_f = 0.6)$, medium $(\omega_f = 1.5 \text{ Hz}, \xi_f = 0.4)$ and soft $(\omega_f = 0.5 \text{ Hz}, \xi_f = 0.2)$ soil (Banerji et al., 2000).

Table 1: The peak ground acceleration (pga) values are given for different earthquake.

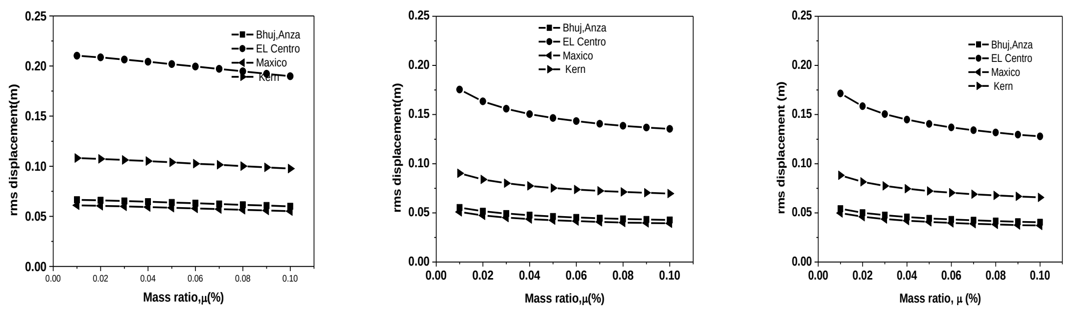
Earthquake	Year	Peak ground acceleration(pga)
Bhuj, India	2001	0.11g
Anza, USA	1980	0.11g
Imperial Valley	1940	0.348g
Mexico City	1985	0.101g
Kern Country	1952	0.179g

The variations of the rms displacement of the primary structure with increasing mass ratio for different earthquake considering hard soil for $\xi_s = 3\%$ structural damping by using STMD, MTMD (3Nos.) and MTMD (5 Nos.) are shown in Fig.2. It is seen that rms displacement reduces with increasing mass ratio. Also, it is observed that the reduction of rms displacement is more for considering MTMD with compared to STMD system. Further, it seen that the rms displacement decreases when Nos. of MTMD is more. It is also observed that the tuning ratio decreases with increasing mass ratio and damping ratio of damper increases with increasing mass ratio. The similar trends are shown in Figs.3 and 4 considering medium and soft soil.



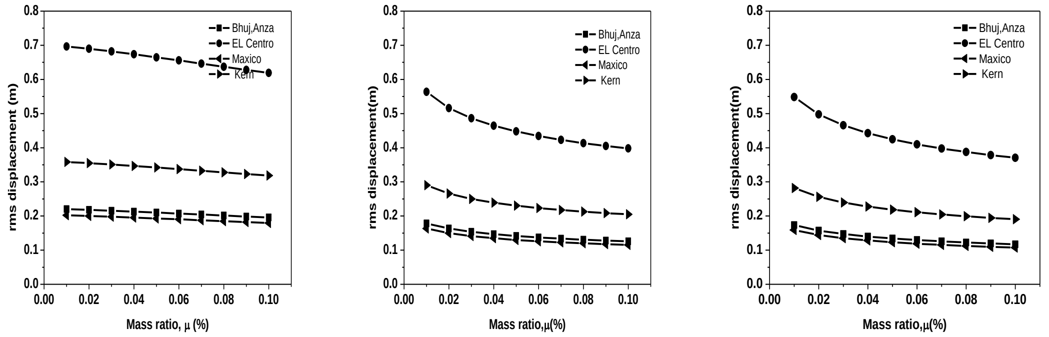
(a) STMD (b) MTMD (3 Nos) (c) MTMD (5 Nos)

Fig.2: Variation of rms displacement of structure with mass ratio for different earthquake considering hard soil by using (a) STMD , (b) MTMD (3 Nos) and (c) MTMD (5 Nos) for $\xi_s = 3\%$



(a) STMD (b) MTMD (3 Nos) (c) MTMD (5 Nos)

Fig.3: Variation of rms displacement of structure with mass ratio for different earthquake considering medium soil by using (a) STMD , (b) MTMD (3 Nos) and (c) MTMD (5 Nos) for $\xi_s = 3\%$



(a) STMD (b) MTMD (3 Nos) (c) MTMD (5 Nos)

Fig.4: Variation of rms displacement of structure with mass ratio for different earthquake considering soft soil by using (a) STMD , (b) MTMD (3 Nos) and (c) MTMD (5 Nos) for $\xi_s = 3\%$

CONCLUSIONS

The performance of MTMDs for controlling the response of a SDOF system is investigated in this paper. The parametric study is conducted to evaluate the influence of several parameters (mass ratio, damping ratio of structure) on the effectiveness and robustness of MTMDs in comparison with STMD. It is observed that rms displacement reduces with increasing mass ratio for MTMD and STMD cases. But, the effectiveness and robustness of MTMD is more in comparison with STMD. It is also seen that tuning ratio decreases with increasing mass ratio and damping ratio of damper increases with increasing the same. Further, it is observed that the rms displacement of primary structure reduces more when no. of MTMD is more.

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MANUFACTURING PROCESSES OF CARBON FIBER REINFORCED POLYMER AND ITS BENEFICIAL APPROACH TO RETROFITTING AND STRENGTHENING OF STRUCTURAL ELEMENTS

M.A. Huq Chowdhury[1*]

[1] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong – 4349, Bangladesh.*
**Corresponding Author, <ayan_cuet09@yahoo.com>*

ABSTRACT

Bangladesh as an underdeveloped country is facing major catastrophe in infrastructure development. Most of the infrastructures are found old and are still in use. The structural members of these infrastructures possess deterioted concrete with extensive cracks and damages make unfit for use in any purpose. A lot of these buildings are being used as residential buildings, schools, colleges, universities, garments factories- and, hospitals etc.. According to seismic zoning map prepared by Bangladesh University of Engineering and Technology, 43 % area in Bangladesh is rated as high risk, while 41% is designated as moderate and 16% as low risk zone. However, complete earthquake monitoring facilities are not available in our country. Strengthening and retrofitting of structural members with carbon fiber reinforced polymer (CFRP) is a common practice as it is less costly than re-establishing the whole structure. In addition, there are no CFRP manufacturing plants in Bangladesh to implement this repairing technique. This paper discusses the simple manufacturing procedures of CFRP and its field applications where its usage is encouragingly notable.

Keywords: CFRP, Manufacturing, Strengthening

INTRODUCTION

Deterioration of concrete structures inaugurates due to adverse condition of the environment where it is exposed to weather, water and chemicals for a long time. Other factors which weaken a concrete structure include primary design errors, additional loads, faulty construction procedure, accidental damages and environmental calamities etc. Consequently strengthening and repairing of these structures become essential to ensure safe usage. Concrete jacketing, addition of steel reinforcement are some of the common techniques, usually followed to improve the degraded concrete members. Concrete jacketing is done by removing the detached concrete to clear away the member surface and filling with new concrete of low shrinkage property with additional ties and reinforcements. This jacketing system has the limitation of minimum thickness 100mm, increasing amount of dead mass, reduction in usable space and being slow and time consuming process (Rai, Indolia, 2011). Jacketing by steel plates using epoxy as a bonding material is another effective process of strengthening. But it has the major disadvantage of corrosion. Moreover, the availability of proper plate length, need of joints, difficulty in handling, need for scaffolding; cost and mechanical efficiency are matters of concern. In recent years, CFRP is extensively used in rehabilitation process of concrete structures. Modulus of elasticity of CFRP is approximately 70 GPa and it possesses a tensile strength of about 100-145 ksi (Ehsani, 2005). High strength to weight ratio, corrosion resistance, easy transportation, high fatigue resistance, availability of any shape are some of the salient advantages of CFRP, makes it a good strengthening material to overcome the limitations of other retrofitting techniques. This paper focuses on the simple manufacturing techniques of CFRP and the strengthening of concrete structural members with CFRP materials. This will be a lead to the manufacturers of Bangladesh to understand the overwhelming market of CFRP and a guide to civil engineers to adopt an effective and economical way of developing and strengthening civil structures.

CARBON FIBER REINFORCED POLYMER (CFRP)

Carbon fiber reinforced polymer composite is a heterogeneous substance formed by combination of two or more materials where materials retain their own characteristics. It has anisotropic properties and it consists mainly of two materials fiber and matrix which are bonded together to deliver the final product (Ishida, Kumar, 1985). The fiber used is primarily carbon fiber which contributes to the necessity of strength and stiffness. And the matrix used is a polymer compound which provides the CFRP composite rigidity and environmental protection.

CARBON FIBER

Carbon fiber is long filament type high performance fiber comes with a diameter of about 5 to 8µm (Mazumdar, 2002). There fabrication process includes pyrolysis and crystallization of precursor above 2000 ºC. Carbon crystals are aligned along the length of the fiber and woven into a fabric. Rayon precursor, polyacrilonitrile (PAN) precursor, pitch precursor are three types, used in the manufacturing process of carbon fiber. Low yielding of the initial fiber mass (25%) is the main disadvantage of rayon precursor (Illston, Domone, 2002). Pitch precursor yields high carbon at lower cost but the fiber manufactured from it has less uniformity. PAN precursor is universally used for manufacturing carbon fiber commercially. About 50% of the initial fiber mass is gained from this precursor. Carbon fiber reinforced polymer has a promising service life than aramid and glass fiber (Shenoi et al., 2002).

MATRIX

Matrix is a synthetic polymer composite, composed of many smaller units called monomers. Matrix with low modulus and elongation higher than the fiber is used to transfer large amount of load to the fiber. Thermoplastic and thermosetting polymers are two major types used as matrix material. Thermoplastic polymers are tougher than thermosets, ductile and flexible in nature and possess lower strength and stiffness. They can be reformed and reshaped by heating and cooling process. There creep resistance is poor at high temperature and they get easily affected by solvents. Polypropylene(PP), nylon 66, polyphenylene sulphide(PPS), polyetheretherketine(PEEK) are some of the examples of thermoplastics. Thermosetting polymers are hardened through the process of condensation and polymerization. Once they are hardened, cannot be reformed or reshaped which make them brittle in nature. They have high dimensional stability than thermoplastics. They occupy high rigidity, thermal, electrical and solvent resistance, and load bearing capability at high temperature. Polyester, vinylester, phenolics, polyimides are commonly used as thermosets (Illstone, Domone, 2002).

MANUFACTURING PROCESSES OF CFRP

HAND LAY-UP AND SPRAY-UP MOLDING

Non-automatic processes of CFRP manufacturing include hand lay-up and spray-up. They were followed at the earliest time of manufacturing. In this process the reinforcement fiber is placed in the resin applied to the mold and a hand-operated metal roller is used to inseminate the fiber with resin. Careful observation to the removal of entrapped air should be given. The impregnation is continued till the desired thickness is achieved. Dimensional stability of the fibers in the long direction can be maintained comfortably in this process. It is also suitable for manufacturing products with variable shapes and sizes. Excessive time and labor requirement, lack of quality control and environment pollution through the discharge of styrene are some of the disadvantages of this method (Ishida, Kumar, 1985).

Spray-up process implicates the usage of a spray gun to blow the liquid resin into the mold. The application method is fast with minimum time and labor requirement. The fibers chopped in the spray gun at a fixed size of 10 to 40 mm, are applied through separate nozzles so as the catalyst. 20 to 40 %

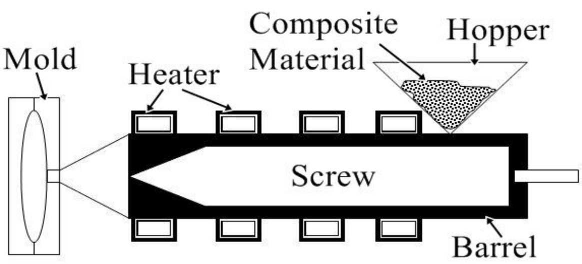
of the total weight is provided as fiber reinforcement (Mazumdar, 2002). The resin should maintain liquidity at room temperature for proper handling. Before the solidification the laminate is rolled to remove any air trapped in the resin fiber matrix. The product is cured at room temperature. It is hard to control thickness, dimensional stability and smooth surface on both sides of the product.

COMPRESSION MOLDING

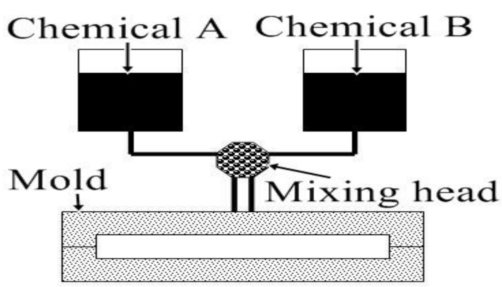
In compression molding, the resin and fiber mixture compound is stored in a preheated mold [usually 140ºC] which is placed between compressor platens. The mold is divided into two halves, one is fixed with the bottom platen and the other half is movable, connected with the upper platen. Rectangular size SMC materials are placed in the bottom fixed mold cavity which are also called charge. The mold operational speed is maintained at 40 mm/sec for sheet molding compound (SMC) (Mazumdar, 2002). 60 to 70% of the surface area of the bottom fixed mold is covered with SMC material. The uncovered area fills up as the charge flow continues (Mallick, 1988). Continuation of flow of SMC material in the mold cavity ensures elimination of air bubbles. The curing process is initiated at higher temperature after the mold is completely filled up. At the end of curing the final product is removed from the mold.

INJECTION COMPRESSION MOLDING

Basically two types of injection molding process is followed – screw type injection molding where the fiber resin mixture is carried by the rotation of a screw and plunger type injection molding where a plunger is used to transfer the mixture towards the mold. The mixture compound is supplied into a hopper through which it goes into the heated barrel. The viscosity of the material lessens as it passes through the barrel with the aid of a rotational screw. After reaching the end of the barrel high pressure (2000 psi to 30000psi) is applied to inject the mixture into the mold (Rosato et al., 2000). To prevent the backflow of the mixture a holding pressure (50% of the injection pressure) is also applied depending on the wall thickness of the product (Bayer, 2013). After the curing of the product the mold is opened and the final product is obtained. To maintain good mechanical properties plunger type injection molding process is accepted as the screw chops off the fibers. Production rate of this process is relatively higher than the other processes.



(a) Injection compression molding



(b) Reaction injection molding

Figure 1 Schematic diagram of Injection molding and Reaction injection molding process (Mazumdar, 2002)

REACTION INJECTION MOLDING

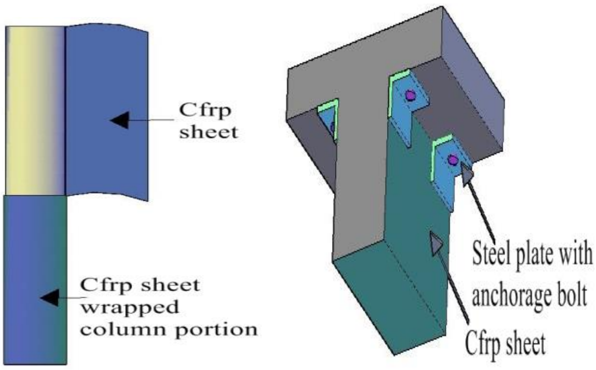
In reaction injection molding (RIM) process two monomeric or oligomeric chemicals are placed in two separate tanks and mixed by injecting with high pressure in a mixing chamber. These highly reactive chemicals collide with a pressure of 10 to 40 MPa and form the polymer matrix (Mazumdar, 2002). The injection pressure of the resin mixture formed, into the mold is far less than the mixing pressure. The Chemicals used are of low viscosity (< 200 centipoise) in room temperature which does not affect the fiber preform placed in the mold (Chanda, Roy, 2007). To remove the molded product from the metal mold, a release agent is also added to the resin mixture. When short fibers are used with the resin mixture as reinforcement the process is called reinforced RIM process. And when long or woven fiber reinforcement is placed in the mold beforehand as reinforcement and resin mixture is injected then the process is called Structural RIM proces

STRENGTHENING

The most commonly used application process of cfrp sheet on concrete structure is wet lay-up system. Usually three types of resin materials are used to confine the concrete: primer, putty and saturant.

In case of beams and columns, surface preparation is needed for efficient confinement of cfrp sheet. Uneven surfaces, loose materials are grinded and sharp corners are given round shapes with a radius of 20 mm (Dhir, Henderson, 1999). To reduce the presence of cracks, surface pores, and large voids and to manage a continuous smooth surface, a bulk adhesive or putty is applied using a trowel whose working time is 40 minutes at 25ºC. A layer of low viscosity polymer primer is applied using a brush or roller. The primer works like a binding substrate to both resin and concrete surface with a working time of 20 minutes at 25 ºC. Then the cfrp sheet is wrapped over a layer of saturant followed by another layer of saturant. The saturant working time is 45 minutes at 25 ºC (MBrace, 1998). If additional layer of fiber sheet is required for desired strengthening then it is followed by additional layer of saturant. After application of the final layer of fiber sheet, extra saturant layer is applied to join the fiber sheet. Trapped air should be removed by pressurizing the fiber sheet through rolling process. Environmental protection of the fiber wrapping system is provided by mortar covering of 12mm. Sand is sprinkled thoroughly over the wrapped structure (Rai, Indolia, 2011).

Concrete Structure
(Primer + Putty) (0.4 mm)
1st layer resin(0.9mm)
Carbon fiber reinforced polymer sheet(0.165mm)
2nd layer resin



(a) (b)

(c)

Figure 2 (a)Thickness of material layers of cfrp sheet application on concrete structure by Scanning Electron Microscope (SEM) (Tumilian,1998), (b) Cfrp sheet application on column, (c) Cfrp sheet application on beam (Kanakubo et al., 2000).

Wrapping of cfrp sheet on column is uncomplicated as the wrapping can be done by continuous fiber. But beams are connected to the floor slabs which do not allow the continuous fiber wrapping. The cfrp sheets are wrapped around the open surface of the beam like three sided U wrap and anchored with the floor slab. 90° angled L shaped steel plates are used. These plates are bolted at the corner of the beam where the beam is connected to the bottom surface of the floor slab (Kanakubo et al., 2000).

Wrapping of cfrp sheets on walls, roof and floor system is accomplished in the same way as beam-column joint as anchoring is done in the corner face of the wall-roof and floor-wall connection. Use of cfrp sheet on these members is costly as they cover the large area of the whole structure. In this case cfrp sheets and laminates are used as strips of specific width and length. Walls can be strengthened by cfrp sheets in certain patterns such as two vertical strips only, two vertical strips with two diagonals, three vertical strips with four diagonals etc. Roof and floor slabs retrofitting technique also follows some certain patterns such as four cfrp strips/laminates with two strips placed parallely at specific distance and these two are orthogonal to other pair of strips placed similarly. They are placed on roof/floor slab in skew or orthogonal formation. Increasing the number of strips does not upgrade the slab capacity noticeably (Soudki et al., 2012).

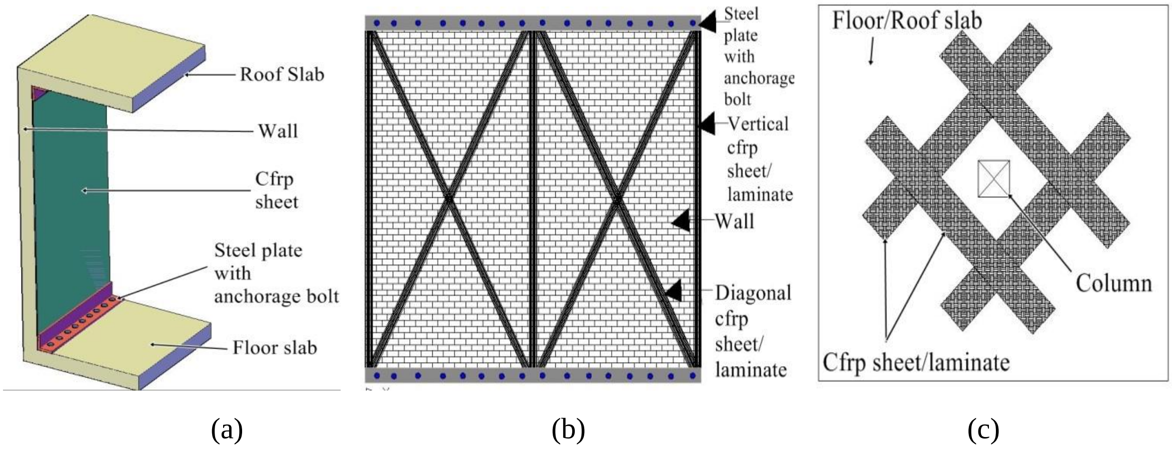


Figure 3 (a) Cfrp sheet application on wall, (b) Wall strengthen by two vertical and four diagonal cfrp sheet/ laminates (Bischof, Suter, 2014), (c) Roof/Floor slab strengthen by four cfrp sheet/laminate in skewed formation offset from column (Soudki et al.,2012).

RESULTS AND DISCUSSIONS

Pultrusion and filament winding are two other manufacturing processes that are not included here because of their complex machinery system. Among other processes, injection compression molding is the most widely used manufacturing method with high productivity. The anchoring system with steel plate and anchorage bolt has greater disadvantage of vulnerability of corrosion effect which will result in disastrous failure of the structure. Hence corrosion free environment should be established in the application place of cfrp and the steel plate and bolts should be properly coated with non-corrosive materials. Applications of CFRP should be done efficiently by taking into consideration its expensiveness, possibility of breaking in high impact load and high electrical conductivity.

CONCLUSION

Carbon fiber reinforced polymer is a strong, lightweight material in which carbon fiber is used as reinforcement with a resin matrix. Higher stress capacity, durability and best strength to weight ratio make it the most reliable and efficient material in construction engineering. The manufacturing processes followed for making CFRP determines its high potential in construction. Besides these simple manufacturing processes, other manufacturing processes and anchorage systems should be evaluated in details for commercial production and application of high strength CFRP. To engage CFRP in retrofitting and strengthening, its application procedures with failure mechanisms need to be known precisely. Design rules and specifications should be made for proper applications and research works should be exercised to evaluate its behavior in different loading conditions. Besides seismic retrofitting and masonry strengthening, CFRP has huge application in bridge engineering, aerospace industry, automotive engineering, manufacturing of sporting goods etc. Hence production, development and improvement of CFRP will open up a new technological era in the world of structural engineering in Bangladesh.

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INVESTIGATION ON PERFORMANCE BASED NON LINEAR PUSH OVER ANALYSIS OF FLAT PLATE RC BUILDINGS

R.K. Biswas[1*], Y. Dinar[2], Shisir Barua[1] & A.M. Jubary[1]

[1] *Department of Civil & Environmental Engineering, Uttara University, Dhaka 1230, Bangladesh.*
[2] *Dept. of Civil Engineering, TUM, Munich, Germany.*
**Corresponding Author, <ajibbiswas_aust@yahoo.com>*

ABSTRACT

Dhaka is one of the highly earthquake prone cities in the world. Due to improper planning and development, the disaster often converted to catastrophic event as evident in history. As flat plate buildings have weakness in lateral loading, so this kind of structure could be more vulnerable in earthquake event. According to BNBC (2006), the buildings are designed according to equivalent static method, response spectrum method and time history analysis. However, the real performance of a structure can be rarely found by these methods and so in reality, flat plate buildings could be damaged even under low ground shaking from distant locations. In developed countries the static nonlinear pushover analysis becoming an essential tool for seismic performance evaluation that not only find the maximum base shear of a seismic event for a particular displacement but also shows the collapsible zone. It is always expected that nonlinear inelastic pushover analysis will provides a better understanding about the actual behaviour of the structures during earthquake as real scenario simulation is not always possible under static analysis.

This investigation evaluate and compare the performances of bare frame flat plate structure, flat plate with different infill percentage cases, flat plate with different configuration of soft storey cases and flat plate with shear wall with each other, and later depending upon the significant findings it is explained for which level of seismic performance shear wall should be preferred over the infill structures that hope to be helpful for engineers to decide where the soft storey could be constructed in the flat plate structures. Above all, a better effect of pushover analysis could be summarized from the outcomes. Walls are represented by equivalent strut according to pushover analysis in accordance with the relevant codes, FEMA 356 and ATC- 40. The results and discussion are prepared on the basis of performance point, base shear, top displacement, storey drift and stages of number of hinges form which are simulated in ETABS 9.7.2 taking loads and other factors as per BNBC (2006).

Keywords: Flat plate, Earthquake, Equivalent static, Pushover analysis, Equivalent strut.

INTRODUCTION

Earthquake engineering is growing rapidly according to necessity. Pushover analysis has become the preferred analysis procedure for seismic performance evaluation purposes as the procedure is simple and considers post elastic behavior. This performance based approach requires a lateral load versus deformation analysis. This could be a method to observe the successive damage states of a building. This procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis. Pushover analysis of finite element was performed by ETABS 9.7.2 where the deficiency of an elastic analysis displays the following features i.e. the analysis considers the inelastic deformation and ductility of the members and the sequence of yielding of sections in members and redistribution of loads in the building are observed. Pushover analysis described in FEMA-356 and ATC-40. Seismic codes are unique to a particular region or country. They are indicator of the level of progress a country has made in the field of earthquake engineering. In this study a RC building with flat plate generates wide spread damage during seismic event which demand for proper seismic evaluation. Engineers are nowadays prone to construct shear wall without knowing the actual demands and requirements which may lead to a overdesigned state. The purpose of the paper is to summarize the basic concepts on which the pushover analysis is based, perform nonlinear static pushover analysis of medium height (7 storey) residential flat plate buildings as found in Dhaka city available and evaluate the performance of the shear wall consisting bare frame with respect to different infill configuration frame structures. Force unit is KN while displacements are measured in mm.

METHODOLOGY

Pushover analysis is a nonlinear static analysis where the lateral loads are increased keeping vertical loads constant while maintaining a predefined distribution pattern along the height of the building, until a collapse mechanism develops. With the increase in the magnitude of the loads, weak links and failure modes of the building are found. Pushover analysis can determine the behavior of a building, including the ultimate load and the maximum inelastic deflection. Local Nonlinear effects are modeled and the structure is pushed until a collapse mechanism gets developed. At each step, the base shear and the roof displacement can be plotted to generate the pushover curve. It gives an idea of the maximum base shear that the structure was capable of resisting at the time of the earthquake. For regular buildings, it can also give a rough idea about the global stiffness of the building. In soft storey the displacement will be maximum in nature as they have no sufficient strength to take loads from above storey but as the soft storey is shifted bottom to top of the structure the results may be found reverse where strength will eventually increases. 7 storied frame structures are simulated by ETABS 9.7.2 to perform the pushover analysis to meet the objectives of this study. Each and every storey is kept soft storey for different case to get the changing trend. Earth quake effect is assigned under UBC 94. Wind load is calculated according to Bangladesh National Building Code (BNBC) by developing an excel sheet to get point loads. Dead load and live load are taken according to standard practice. Load combinations are taken according to BNBC. To perform the pushover FEMA- 356 and ATC -40 are reviewed throughout the study. All three types of hinges required for performing pushover analysis of RC structure are chosen from the experimental. Allowable hinge deformation at different performance level for beams and columns is established. All three types of hinges are assigned to each element according to required type. Structures are then subjected to pushover analysis which includes progressive damage of elements with plastic deformation of the hinge assigned on the element of the structure as the structure is laterally pushed through. After simulation the structural response outcomes will be used to give the light on study objectives.

DETAILS OF PUSHOVER ANALYSIS

Pushover analysis provides a wide range of application options in the seismic evaluation and retrofit of structures. Mainly two guidelines are available for this analysis- FEMA and ATC 40. This paper mainly follows the procedures of ATC 40 in evaluating the seismic performance of residential building consisting shear wall in Dhaka. Equivalent Static lateral loads approximately represent seismic generated forces. Analysis is carried out till to failure of the structures. This analysis identifies weakness in the structure so that appropriate retrofitting could be provided in governing element. Basically, demand and capacity are the two component of the performance based analysis and design where demand is a representation of the seismic ground motion and capacity is a representation of the structure ability to resist seismic demand. The performance is dependent in a manner that the capacity is able to handle the seismic demand. Once the capacity curve and demand displacement are defined, a performance check can be done. In our study, nonlinear static pushover analysis was used to evaluate the seismic performance of the structures. The numerical analysis was done by ETABS 9.7.2 and guidelines of ATC-40 and FEMA 356 were followed. Overall evaluation was done using base shear, deflection, storey drift, storey drift ratio and stages of number of hinges form. Plastic hypotheses was used to mark the nonlinear behavior according to which plastic deformations are lumped on plastic hinges and rest of the system shows linear elastic behavior(Li 1996). The discrete structural performance levels are- Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5) and Not Considered (S-6) Whereas intermediate structural performance ranges are the Damage Control Range (S-2) and the Limited Safety Range(S-4) Figure 1.This definition of performance ranges are served by FEMA 356, 2000. The model frame used in the static nonlinear pushover analysis is based on the procedures of the material, defining force – deformation criteria for the hinges used in the pushover analysis. Figure 2 describes the typical force-deformation relation proposed by those documents.

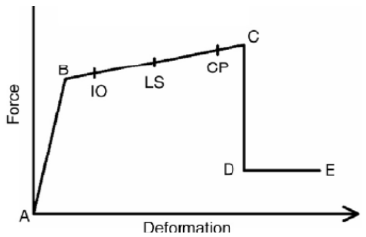


Fig.1 Force-Deformation for pushover analysis **Fig. 2 Force-Deformation for pushover analysis**

Five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge and these points labeled A to B – Elastic state, B to IO- below immediate occupancy, IO to LS – between immediate occupancy and life safety, LS to CP- between life safety to collapse prevention, CP to C – between collapse prevention and ultimate capacity, C to D- between C and residual strength, D to E- between D and collapse >E – collapse. In ETABS 9.7.2 those points could be identified by color bands to understand how plastic hinges form in each stage Fig. 2 where IO, LS and CP mean immediate occupancy, life safety and collapse prevention respectively.

METHOD OF REPLACEMENT OF INFILL

The approaches presented by Paulay and Pristlay (1992) and Angel et al.(1994), and later adopted by R. Shahrin & T.R. Hossain (2011) lead to a simplification in the infilled frame analysis by replacing the masonry infill with an equivalent compressive masonry strut as shown in Fig. 3-(a).

$\lambda 1H = = H\,[(Em\ t \sin 2\theta) / (4\ EcIcolhw)]\ ¼$ (1)

where t is the thickness of masonry wall. Main stone (1971) considers the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel as shown in Eq 2

$a = 0.175D\ (\lambda 1H)\text{-}0.4$ (2)

If there are opening present, existing infill damage, and/or FRP overlay, however, the equivalent strut must be modified using

$Amod = a\ (R1)i(R2)i\ \zeta 1$ (3)

Credit: R. Shahrin & T.R. Hossain (2011)

$\zeta 1$= strength increase factor due to presence of FRP overlay

Although the expression for equivalent strut width given by Eq 4 was derived to represent the elastic stiffness of an infill panel, this document extended its use to determine the ultimate capacity of in filled structures. The strut was assigned strength parameter consistent with the properties of the infill it represents. A nonlinear static procedure commonly referred to as pushover analysis, was used to determine the capacity of the infilled structure. The equivalent masonry strut is to be connected to the frame members as depicted in Figure 3. Where the bold double sided arrow represents the location of the strut in the structural model. The infill forces are assumed to be mainly resisted by the columns, and the struts are placed accordingly. The strut should be pin connected to the column at a distance lcolumn from the face of the beam. This distance is defined in Eq 3 and Eq 5 and is calculated using the strut width, a, without any reduction factors.

$lcolumn = a/ \cos\theta column$ (4)

$\tan\theta column = \{hm\text{-}(a/\cos\theta column)\}/l$ (5)

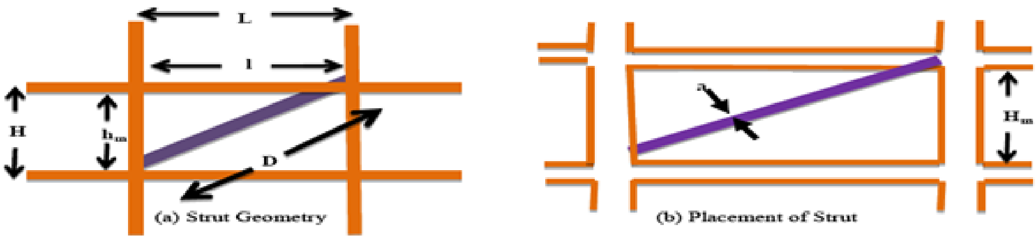


Fig. 3 (a) Strut Geometry (b) Placement of Strut

The strut force is applied directly to the column at the edge of its equivalent strut width. Fig. 3-(b) illustrates these concepts. Modulus of elasticity of the masonry units was chosen considering the ACI/ASCE/TMS masonry code as 1200 ksi.

DESCRIPTION OF THE STRUCTURAL MODELS AND CASES

For Numerical modelling, a sample flat plate three dimensional building is selected. The structure is eight stories high, with a story height of 3 meters. The bay lengths are 5m- 5m in both directions. In order to concentrate on the effects caused by the distribution of infill the prototype bare frame structure is regular throughout its bay length in both directions. The column sizes are 450 X 300 mm for all position and the slab thickness is 175 mm. The concrete strength is assumed to be 4000 psi with yield strength 60000 psi where Modulus of Elasticity is 3600 ksi. Masonry infills were modeled as equivalent diagonal strut with width of 485 mm and thickness of 125 mm. The masonry infill has compressive strength of 1 MPa. The model is assumed to be situated in Dhaka city so, according to Bangladesh National Building Code seismic zone 2 is taken. Parallel and periphery shear wall were modeled using 10 inch wall with compressive strength of 4000 psi and Modulus of Elasticity of 3600 ksi. Shear walls were modeled taking the half-length 2.5 m of each bay to resist the lateral loads only. Moment hinges (M3) were assigned to both ends of beams and axial hinges (P-M-M) were assigned to the column ends. Geometric non linearity (P-Δ) and large displacement is considered with full dead load and when local hinges fail redistribution of loads is allowed by unloading whole structure. The gravity loads used included self-weight of the members and loads of floor finish and live loads were applied to BNBC. All partition walls were assumed to be located directly on beams. The performance points marked by collapse and representing ultimate displacement capacity of the structure were evaluated at each step of the analysis according to guidelines of ATC-40 and FEMA 356

The load deformation responses of the numerical model specimens were followed through to failure by means of the capacity curve under 3 pushover cases. The curve was gained using pushover analysis, where the loading profile used was a triangular one com-menstruate to the dominate first mode distribution of the seismic loads. Depending upon the infill percentage 5 cases were taken with 25% interval i.e. bare, 100 % infill, 75% infill, 50% infill and 25% infill where each of the case represents a probable infill configuration. Two basic Shear wall cases are taken for study i.e. periphery shear-wall and parallel shear-wall and 8 different soft storey were investigated too.

- Infill cases: Bare frame, 100% Infill, 75% Infill, 50% Infill, 25% Infill
- Soft storey cases: GF Soft, 1st Soft Storey, 2nd Soft Storey, 3rd Soft Storey, 4th Soft Storey, 5th Soft Storey, 6th Soft Storey, 7th Soft Storey
- Shear Wall Cases: Parallel Shear Wall, Periphery Shear Wall

RESULTS AND DISCUSSIONS

Outcomes of analysis are summarized to observe structural responses. Seismic performance in terms of base shear, performance point and top displacement increases with shifting of soft storey upward in the structure. Performance point is much higher for shear wall consisting bare frames than any other configuration which propose bare frame could be a choice while using any types of shear wall Fig. 4.

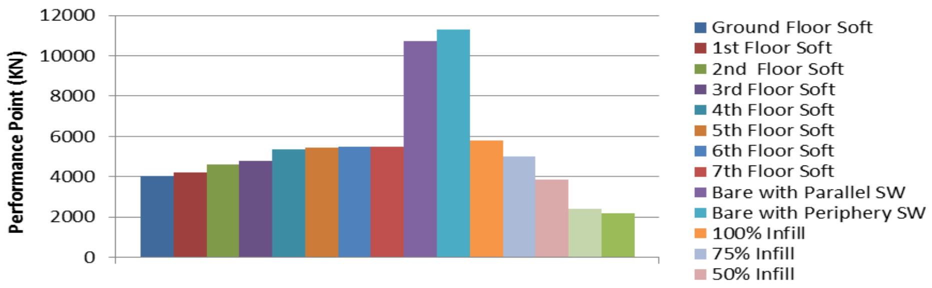


Fig. 4 The comparison of performance point between different soft storey cases

General bare frame has very low resistance against lateral force even from any soft story case. As the soft story shifted above the performance also increased. Considering this point soft story could be setup in upper floor to improve the lateral load bearing capability relatively. Similar scenario is reflected in base shear too which is nothing but upper point of the performance point.

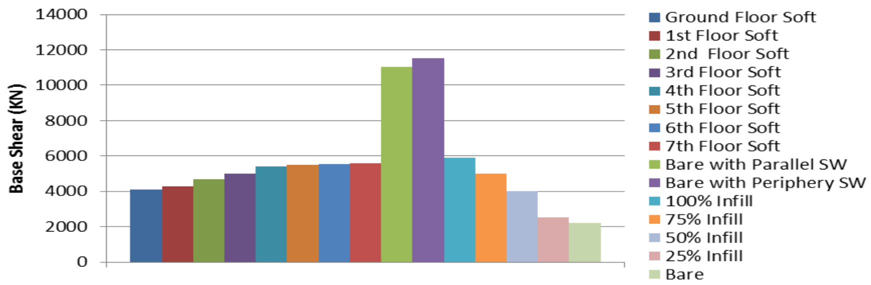
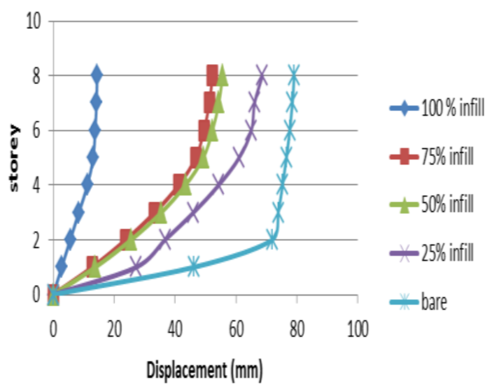


Fig. 5 The comparison of base shear between different soft storey cases

The difference of performance point and base shear increases with shifting of soft storey upward and constructing the shear wall. It seems early formation of soft storey is not desired in the structures of highly seismic risk areas. Hinges goes to collapsible condition after passing a few intermediate stages i.e. immediate occupancy and life safety. In linear static only the final displacement found by providing a constant load where the pushover sequentially increases the load from low to the governing one. By producing plastic hinges it identify the change of state of each member in each pushover step. Formation of maximum number of hinges in early stage is not good for structure which eventually represents that early reaching to the collapsible condition Table I. Looking on the number of hinges formed one thing is clear, higher infill ensures uniform hinge formation while shear wall significantly reduces the hinges formation than any other cases.

Gradual displacements changing ensure structural stability, uniform stiffness and less probability to the evaluation of plastic hinges. Plastic hinges eventually go to collapsible condition and cannot stand with load. To withstand against progressive loads formation of plastic hinge must be controlled by using special structural components. With the increment of percentage of infill the magnitude of displacement also reduced. Displacement reduces and performance increases with shipment of soft storey to upward.



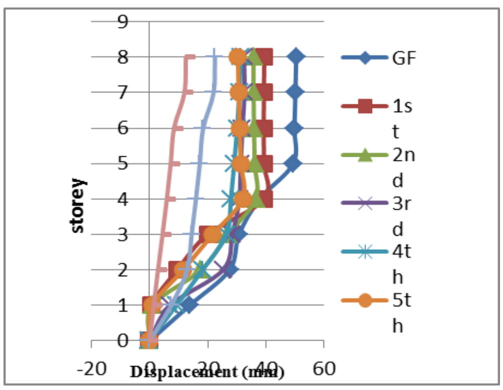


Fig. 6 Comparison of displacement of infill cases

Fig 7: Comparison of displacement of soft story cases

Table I

Number of hinges formed in Infill and Shear wall configuration

	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
Bare	220	30	0	90	0	3	7	0	350
Bare with Parallel SW	116	111	159	13	0	1	0	0	400
Bare with Periphery SW	84	49	176	87	0	4	0	0	400
100% Infill	520	75	76	21	22	1	0	0	715
75% Infill	389	81	26	0	0	0	51	13	560
50% Infill	346	32	18	0	0	0	49	10	487
25% Infill	400	0	1	0	0	0	19	0	420
GF Soft	607	2	21	46	0	4	0	0	680
1st Floor Soft	603	3	24	35	0	15	0	0	680
2nd Floor Soft	589	0	40	42	0	0	1	0	680
3rd Floor Soft	560	0	17	21	0	10	72	0	680
4th Floor Soft	550	0	28	20	0	12	70	0	680
5th Floor Soft	456	74	60	70	19	1	0	0	680
6th Floor Soft	618	0	23	0	0	0	39	0	680
7th Floor Soft	611	7	1	0	0	0	61	0	680
Bare	220	30	0	90	0	3	7	0	350

CONCLUSION

Lateral stiffness of flat plate building is low so lateral strengthening of flat plate structures is seriously a matter of concern. After summarizing the results lead to a decision that infill, shear wall and soft storey configuration significantly affects the performance, lateral stiffness property of the flat plate structure. Under performance based analysis, with the increment of infill percentage the performance increases. The comparison of performance of soft storey analysis reveals that shipment of soft storey upward increases the performance. Flat plate, Shear wall is the ultimate lateral load bearing structures which tripled the resistance capacity of a frame and periphery shear wall could be a better option for all directional lateral loads than parallel shear wall configuration.

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EFFECT OF ASPECT RATIO ON COLUMN STRENGTHENING USING FRP LAMINATES

Md Shajal Khan[1*], Md. Abul Bashar Emon[1] & Tanvir Manzur[1]

[1] *Department of Civil Engineering, BUET, Dhaka-1000,Bangladesh.*
**Corresponding Author, <khanshajal@gmail.com>*

ABSTRACT

Wrapping of FRP (Fiber Reinforced Polymers) laminates on reinforced concrete column has already been a very popular retrofitting scheme in recent times and the new trend is designing FRP confined columns to reduce column size. However, there is no guide line for retrofitting with FRP or established design procedure in BNBC (Bangladesh National Building Code) for FRP wrapped structural members. Engineers, now a days, resort to ACI 440.2R-08 for specifications and design process for using FRP. The analysis of FRP-confined columns is based on the principles of equilibrium and strain compatibility as in conventional RC column analysis, the major modification being the use of the stress–strain model for FRP-confined concrete. This paper presents the effect of aspect ratio and a number of FRP layers on the capacity of columns. Interaction diagrams considering various aspect ratios, reinforcement ratios and wrapping layers of FRP laminates are also constructed. Such diagrams can be used to determine the required number of layers of FRP wrapping for enhancing axial load and bending moment capacity of a column. The present study shows that enhancement of axial load capacity by FRP wrapping for columns with lower aspect ratios are considerably larger than those with higher aspect ratios. On the contrary, improvement of bending moment capacity by FRP confinement relies lesser on aspect ratio and the scale of enhancement is also smaller compared to that of axial load capacity enlargement.

Keywords: FRP laminates, Rectangular Column, Strengthening, Aspect ratio, Interaction diagram

INTRODUCTION

Recently retrofitting of structures has become an important issue to get continued service of existing structures such as building, bridge and others. Owing to structural deficiencies, as in a lot of cases, columns are found inadequate under seismic or lateral loads and thereby jeopardizing stability and performance of whole structure. In these cases, columns are needed to be repaired or retrofitted immediately. Recently, several initiatives have been taken to identify structurally deficient RMG factory buildings of the country after the tragic incident of Rana Plaza. Eventually, retrofitting of several buildings will become inevitable though such practice is not very common here in Bangladesh. There are several International Codes and Guidelines available for different retrofitting techniques. Utilization of Fiber Reinforced Polymers (FRP) laminates for wrapping columns is one of the most popular retrofitting schemes. FRP method is relatively popular since it requires less time and strength of columns can be restored or strengthened considerably. Columns are structural members subjected to combinations of axial compression and bending moment, rather than pure axial loading. The flexural effect may be induced by different factors, such as unbalanced moments at connecting beams, vertical misalignment or lateral forces resulting from wind or seismic activity. Confinement of concrete is an efficient technique to enhance the load-carrying capacity of reinforced concrete columns. Under the lateral confining pressure provided by the confining material, the concrete column is subjected to a tri-axial stress state, thereby increasing the ultimate stress and strain. This study also aims at identifying the effect of aspect ratio of rectangular column on interaction diagram for FRP confined reinforced concrete column. The method of analysis and procedure interaction diagram of FRP-confined columns is based on the principles of equilibrium and strain compatibility as in conventional RC column analysis, the major modification being the use of the stress–strain model for FRP-confined concrete as developed by Lam and Teng (Lam et al., 2003). Required number of FRP laminates layers for enhancing the capacity of reinforced concrete column can also be determined from the interaction diagrams constructed in this method. The following simplified P–M interaction diagrams could be used by practitioners as a design tool and for a clear illustration of the procedure, an example application is presented in this paper.

MATERIALS

In this study CFRP is used for column strengthening and constructing P-M diagram. The FRP material properties as per manufacturer's manual are presented in Table 1. Column properties are tabulated in Table 2.

Table 1: Properties of CFRP

Thickness per ply t_f	0.33 mm
Ultimate tensile strength f^*_{fu}	3792.11 MPa
Rupture strain ε^*_{fu}	0.0167 mm/mm

Modulus of elasticity E_f	227526.58 MPa

Table 2: Material and section properties of Columns

No.	b (mm)	h (mm)	Aspect ratio	A_g(mm^2)	A_{st} (mm^2)		ρ_g %	f'_c (MPa)	f_y (MPa)
1	300	300	1	90000	8-ϕ16	1560	1.73	24.13	413.68
2	300	375	1.25	112500	4-ϕ16+4-ϕ20	1997.5	1.78	24.13	413.68
3	300	450	1.5	135000	6-ϕ16+4-ϕ20	2387.5	1.77	24.13	413.68
4	300	525	1.75	157500	8-ϕ16+4-ϕ20	2777.5	1.76	24.13	413.68

INTERACTION DIAGRAM CONSTRUCTION METHODOLOGY

For constructing the interaction diagram of column, methodology as outlined by Rocca et al. (Rocca et al., 2009) has been followed for the present analysis. The following assumptions are considered for the analysis: (a) plane sections remain plane, (b) the tensile strength of concrete is neglected, and (c) complete composite action is assumed between both steel reinforcement concrete and FRP-concrete. Present analysis for FRP-confined columns follows the method for typical RC columns except for the compression zone in concrete where a specific stress-strain model for confined concrete is used. Instead of a continuous curve, a conservative P-M diagram is constructed by joining several points including five characteristic points with straight lines in Figure 1 (MacGregor J.; 1997).
These five critical points are as follows:

- Point A: uniform axial compressive strain of confined concrete ε_{ccu}(or $\varepsilon_{cu}$for the case of an unstrengthened cross-section).
- Point B: strain distribution corresponding to a maximum compressive strain ε_{ccu} (or ε_{cu}) and zero strain at the layer of longitudinal steel reinforcement nearest to the tensile face.
- Point C: strain distribution corresponding to the balanced failure with a maximum compressive strain ε_{ccu} (or ε_{cu}) and a yielding tensile strain ε_{sy} at the layer of longitudinal steel reinforcement nearest to the tensile face.
- Point D: strain distribution corresponding to the limiting tension-controlled failure having a maximum compressive strain ε_{ccu} (or ε_{cu}) and a tensile strain of 0.005 as per ACI 318-05 (American Concrete Institute; 2005) at the layer of longitudinal steel reinforcement nearest to the tensile face.
- Point E: point corresponding to the pure bending moment and zero axial force.

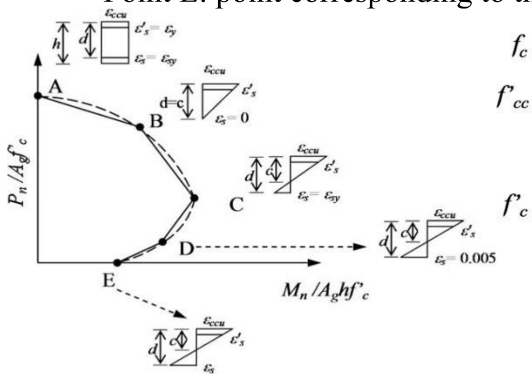

Figure 11: Simplified Interaction Diagram

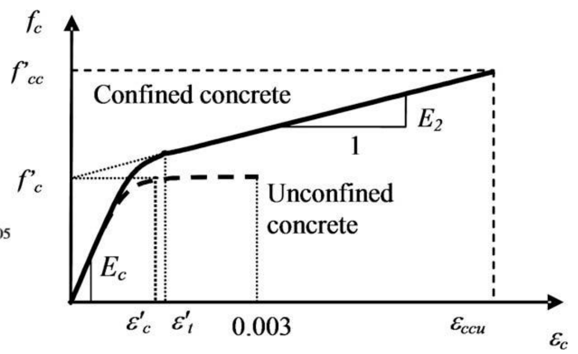

Figure 12: Stress–strain model for FRP Confined Concrete by Lam and Teng (Lam et al., 2003)

In the interaction diagram, point A and E represent the pure compression case (zero bending moment) and the pure bending moment case (zero axial force) respectively. For pure bending, conventional RC beam theory is applicable and since this case is rarely encountered, point E is omitted from the diagrams. For points B, C, and D, the position of the neutral axis c is directly computed by similar triangles in the strain distribution corresponding to each case (Figure 1). The nominal axial load P_n corresponding to point A can be foundusing Eq. (1) (M_n at point A equals zero).

$$P_{n\,(A)} = 0.8\{0.85 f_c' \left(A_g - A_{st}\right) + A_{st} f_y\} \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

At B, C and D, nominal axial load $P_n$and nominal bending moment $M_n$is given by Eq.(2a) and Eq. (2b) which is for rectangular cross sections.

$$P_{n\,(B,C,D)} = \int_0^c (b)\, f_c(y)\, dy + \sum A_{si} f_{si} \ldots\ldots\ldots\ldots\ldots\ldots (2a)$$

$$M_{n\,(B,C,D)} = \int_0^c (b)\left(\frac{h}{2} - c + y\right) f_c(y) dy + \sum A_{si} f_{si} d_{si} \ldots\ldots\ldots\ldots\ldots\ldots (2b)$$

In the expressions above, c is the distance from the neutral axis position to the extreme compression fiber in the cross-section. A_{si} and f_{si} are the cross-sectional area and the normal stress, respectively, of the i-th layer of longitudinal steel reinforcement. The parameter d_{si} is the distance from the position of the i-th layer of longitudinal steel reinforcement to the geometric centroid of the cross-section and 'y' is the variable of integration within the compression zone. The concrete stress $f_c$corresponds to the model by Lam and Teng (Lam et al., 2003) shown in Figure 2.
This model is used in this study to compute the stress–strain curve of the FRP confined concrete. It is given by the following Eq. (3a), Eq. (3b) and Eq. (3c).

$$f_c = \begin{Bmatrix} E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4 f_c'} \varepsilon_c^2 \quad ; \; 0 \le \varepsilon_c \le \varepsilon_t' \\ f_c' + E_2 \varepsilon_c \; ; \; \varepsilon_t' \le \varepsilon_c \le \varepsilon_{ccu} \end{Bmatrix} \ldots\ldots\ldots\ldots\ldots (3a)$$

$$E_2 = \frac{f_{cc}' - f_c'}{\varepsilon_{ccu}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3b)$$

$$\varepsilon_t' = \frac{2 f_c'}{E_c - E_2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3c)$$

Where, f_c and ε_c are the axial stress and the axial strain of confinedconcrete, respectively. $E_c$is the elastic modulus of unconfined concrete, ε_t' is the transition strain, E_2 is the slope of the second linearportion, and ε_{ccu} is the ultimate axial strain of confined concrete. The maximum FRP-confined concrete compressive strength is given by Eq. (4).

$$f_{cc}' = f_c' + \psi_f 3.3 \kappa_a f_l \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Where, $\kappa_a$is a geometry efficiency factor and will be discussed later in this article. The FRP confining pressure f_l is given by Eq. (5).

$$f_l = \frac{2 E_f n t_f \varepsilon_{fe}}{D} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The effective strain ε_{fe} in Eq. (5) is taken as the product of an efficiency factor κ_ε and the ultimate tensile strain ε_{fu}. The FRP strain efficiency factor κ_ε accounts for the difference between the actual rupture strain observed in FRP-confined concrete specimens and the FRP material rupture strain determined from tensile coupon testing. Various studies found the value of κ_ε in the proximity of 0.58 (Lam et al., 2003; Carey S., 2003; Carey SA., 2005). For the current study, the value of the parameter κ_ε is taken as 0.55 with practical design considerations. To ensure the shear integrity of the confined concrete, a minimum value 0.004 is adopted according to Priestley et al. (Priestley et al.,1996)and as recommended by ACI Committee 440 (ACI 440.2R, 2008).The ultimate axial strain of the FRP-confined concrete compressive stress–strain curve is given by Eq. (6)

$$\varepsilon_{ccu} = \varepsilon_c' \left\{1.50 + 12\kappa_b \frac{f_l}{f_c'} \left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right\} \le 0.01 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Where, the axial strain at the compressive strength of unconfined concrete ε'_c. Based on recommendations from the Concrete Society in the Technical Report (Concrete Society, 2004)a limiting value of ultimate axial strain of 0.01 in the case of confined concrete is introduced only in members under pure compression to prevent excessive cracking and the resulting loss of concrete integrity. This limit is applicable and the corresponding maximum value of f'_{cc} isrecalculated from the stress–strain curve.

The efficiency factors κ_a in Eq. 4 and $\kappa_b$in Eq. (6) can be computed according to Eq. (7) and (8) as they account for the geometry of the cross-section. In the case of circular cross-sections, they are taken as 1.0 and in the case of non-circular cross-sections, they depend on two parameters: the effectively confined area ratio $A_e/A_c$and the aspect ratioh/b. These factors are given by the following equations.

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Where, the ratio of A_e/A_c is expressed by following equation. Here r_c is the corner radius of rectangular column.

$$\frac{A_e}{A_c} = \frac{1 - \dfrac{\left\{\left(\frac{b}{h}\right)(h-2r_c)^2 + \left(\frac{h}{b}\right)(b-2r_c)^2\right\}}{3A_g} - \rho_g}{1-\rho_g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

P-M INTERACTION DIAGRAMS BASED ON DIFFERENT ASPECT RATIOS

For the purpose of strengthening each column type described in the materials section, the number of FRP ply is varied from 2 to 5 and corresponding $\phi P_n\,(kN)$ and $\phi M_n\,(kN.mm)$ diagrams are given in Figure 3 through 6.

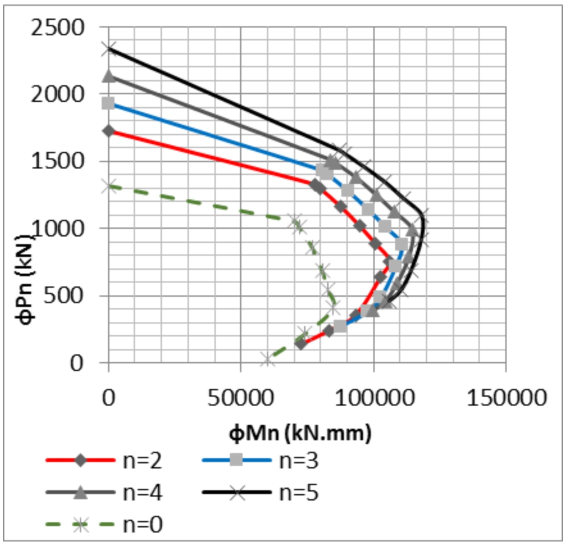

Figure 3: P-M diagram for various layers of FRP wrapping on Col.1

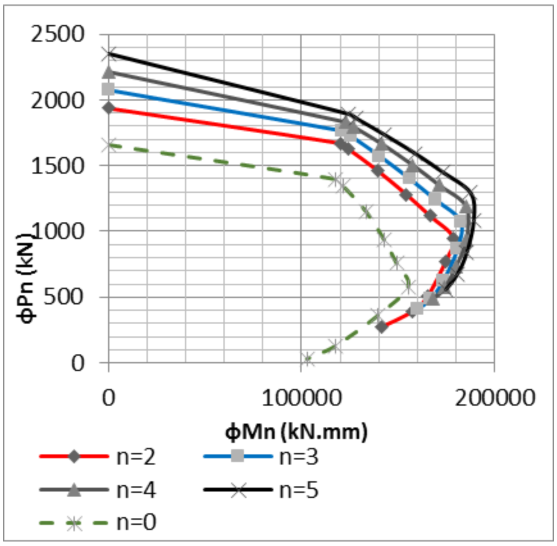

Figure 4: P-M diagram for various layers of FRP wrapping on Col.2

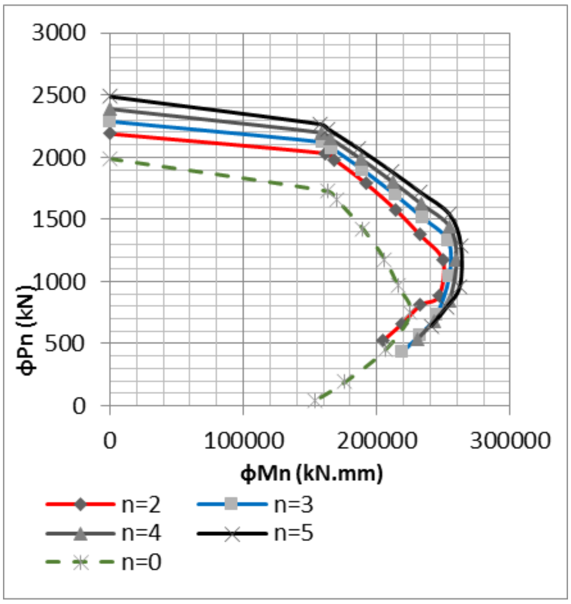

Figure 5: P-M diagram for various layers of FRP wrapping on Col.3

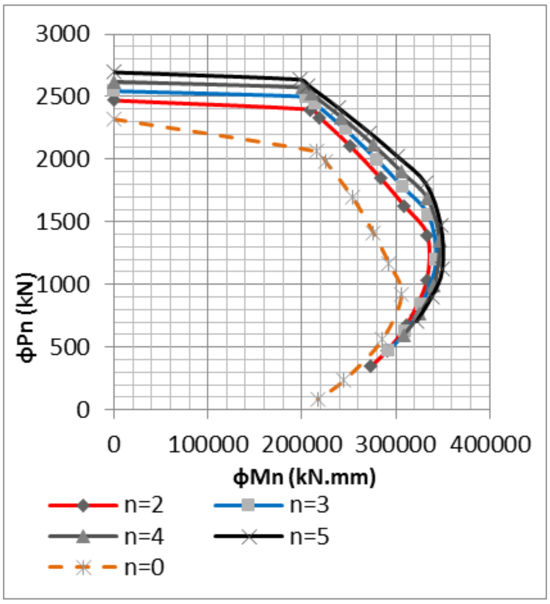

Figure 6: P-M diagram for various layers of FRP wrapping on Col.4

RESULT AND DISCUSSION

From the P-M diagrams, it is observed that FRP confinement increases the pure axial load capacity of columns with all aspect ratios; but for columns with higher aspect ratios, the increase is considerably lower than those with smaller aspect ratios. Increasing number of layers of FRP is also very effective to boost axial load capacity for columns with smaller aspect ratio whereas for columns with higher aspect ratio, additional layers of FRP result in much smaller increment in strength. For example, each additional layer increased ϕP_n by almost 15% for column 1 (aspect ratio= 1) but only a mere 4% for column 4 (aspect ratio= 1.75).Effect of aspect ratio and number of FRP layers on increase of capacity for pure axially loaded columns is presented in Figure 7 for a better understanding of the scenario. Figure 8 demonstrates the same effect for columns under combined moment and axial load such that a balanced failure is predicted i.e. simultaneous failure of concrete and reinforcing steel.

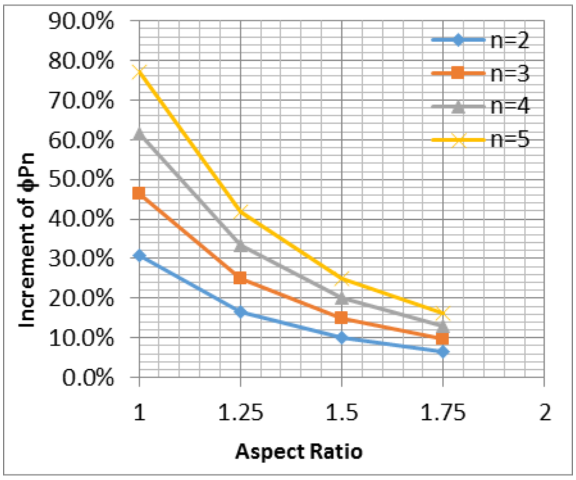

Figure 7: Increment of ϕP_n with respect to RC column (pure axial load condition)

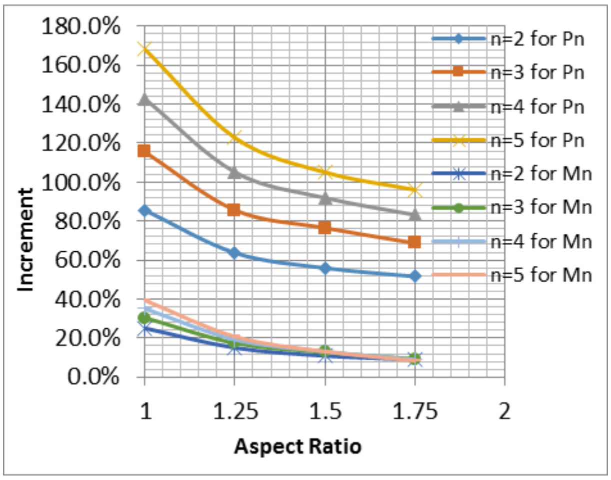

Figure 8: Increment of ϕP_n & ϕM_n with respect to RC column (balanced condition)

The curves for ϕP_n in Figure 8 show a similar trend as those of Figure 7 which means FRP wrapping is also effective in improving axial load capacity of columns that are subjected to combined axial load and moment. However, improvement of moment capacity by addition of FRP layers is not as significant as that of axial load capacity, irrespective of aspect ratio. Therefore, for columns deficient of moment capacity rather than axial strength, retrofitting with FRP may not always be the best option. On the other hand, for columns that need strengthening axial load capacity, FRP wrapping is always a good choice. The only consideration in this case is that the lower the aspect ratio, the better the performance.

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Appendix A:
Calculation of Interaction Diagram

Here, Col. 2 is considered an interior column
Considering the environmental reduction factor

(a) Design properties of FRP materials

$f_{fu} = C_E f_{fu}^{*}$
$f_{fu} = 3792.11 \times 0.95 = 3602.5\ MPa$
$\varepsilon_{fu} = C_E \varepsilon_{fu}^{*}$
$\varepsilon_{fu} = 0.0167 \times 0.95 = 0.01587\ mm/mm$

(b) Determination of simplified P-M diagram for strengthening Col. 2. A wrapping system composed of five plies will be starting point to construct the bilinear curve A-B-C and others points be compared with the position of neutral axis.

Point A: Fully compression with zero bending moment (Figure 1)
Design axial capacity

$$\phi P_{n(A)} = \phi 0.8\{0.85 f'_{cc}(A_g - A_{st}) + A_{st} f_y\}$$
$$\phi P_{n(A)} = 0.65 \times 0.8\{0.85 \times 36.915(112500 \quad - 1997.5) + 1997.5 \times 413.68\}$$
$$\phi P_{n(A)} = 2232.695\ kN$$

Where, the following parameters are

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left\{\left(\frac{b}{h}\right)(h - 2r_c)^2 + \left(\frac{h}{b}\right)(b - 2r_c)^2\right\}}{3A_g} - \rho_g}{1 - \rho_g}$$
$$= \frac{1 - \frac{\left\{\left(\frac{300}{375}\right)(375 - 2 \times 25)^2 + \left(\frac{375}{300}\right)(300 - 2 \times 25)^2\right\}}{3 \times 112.5 \times 10^3} - 1.78 \times 10^{-3}}{1 - 1.78 \times 10^{-3}}$$

$$\frac{A_e}{A_c} = 0.509438$$

$$and\ \kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 = 0.509438 \times \left(\frac{300}{375}\right)^2 = 0.32604$$
$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5} = 0.509438 \times \left(\frac{375}{300}\right)^{0.5} = 0.56957$$

Now $\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} = 0.55 \times 0.01587 = 0.008\ \mathrm{mm/mm}$

$$f_l = \frac{2E_f n t_f \varepsilon_{fe}}{\sqrt{b^2 + h^2}} = \frac{2 \times 227526.58 \times 5 \times 0.33 \times 0.008}{\sqrt{300^2 + 375^2}}$$
$$f_l = 12.508\ MPa\ and\ \frac{f_l}{f'_c} = 0.5183$$

Thus $f'_{cc} = f'_c + 3.3\psi_f \kappa_a f_f$
$= 24.13 + 3.3 \times 0.95 \times 0.32604 \times 12.508$
$= 36.915\ MPa$

Point B:

$$\phi P_{n(B)} = \phi\{(Ay_t^3 + By_t^2 + Cy_t + D) + \qquad \sum A_{si} f_{si}\} \ldots\ldots\ldots \text{Eq. 2(a)}$$
$$\phi P_{n(B)} = 0.65\{(-2.63 \times 10^{-4} \times 96.61^3 + 0.0761 \times 96.61^2 + (-7.355) \times 96.61 + 2639.7287) + (803.75 \times 413.68 + 390 \times 413.68)\} = 1882.852\ kN$$

Where, the following parameters are

$$\frac{A_e}{A_c} = 0.509438\,, \kappa_a = 0.32604, \kappa_b = 0.56967$$

$$\varepsilon_{fe} = min(0.004, \kappa_\varepsilon \varepsilon_{fu}) = 0.004$$

$$f_l = \frac{2\times 227526.58 \times 5 \times 0.33 \times 0.004}{\sqrt{300^2+375^2}} = 6.254\ MPa$$

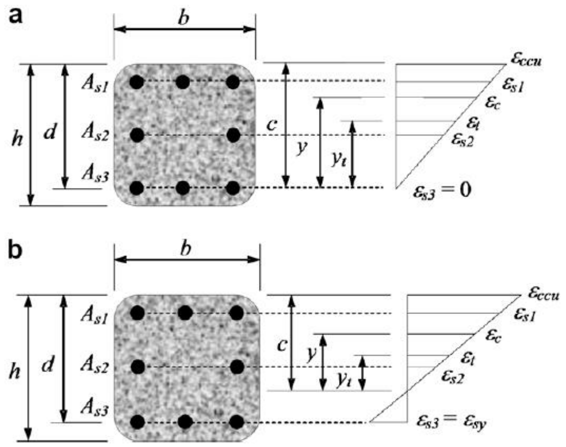


Figure 7: Strain Distribution for P-M diagram

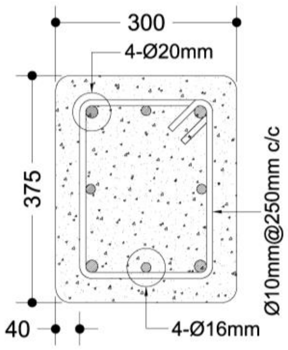


Figure 7: Details Cross-section of Col. 2

$$f'_{cc} = 24.13 + 3.3 \times 0.95 \times 0.32604 \times 6.254$$
$$= 30.522\ MPa$$

$$\varepsilon_{ccu} = \varepsilon'_c \left\{1.5 + 12\kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right\}$$

$$= 0.003\left\{1.5 + 12 \times 0.56957 \times \frac{6.254}{24.13}\left(\frac{0.004}{1.7 \times 24.13/23250.22}\right)^{0.45}\right\} = 0.00709mm/mm$$

$$E_2 = \frac{f'_{cc} - f'_c}{\varepsilon_{ccu}} = \frac{30.522 - 24.13}{0.00709} = 887.046\ MPa$$

$$\varepsilon'_t = \frac{2f'_c}{E_c - E_2} = \frac{2 \times 24.13}{23250.22 - 887.046} = 0.00216mm/mm$$

Now $c = 312.5mm$

And $y_t = c\frac{\varepsilon'_t}{\varepsilon_{ccu}} = 312.5 \times \frac{0.0021}{0.00709} = 96.61\ mm$

The coefficients A, B, C, D, E, F, G, H and I are

$$A = \frac{-b(E_c - E_2)^2}{12 f'_c}\left(\frac{\varepsilon_{ccu}}{c}\right)^2 = -2.63 \times 10^{-4} kN/mm^3$$

$$B = \frac{b(E_c - E_2)^2}{2}\left(\frac{\varepsilon_{ccu}}{c}\right) = 0.0761\ kN/mm^2$$

$$C = -bf'_c = -7.355\ kN/mm$$

$$D = bcf_c' + \frac{bcE_2}{2}\varepsilon_{ccu} = 2639.7287\ kN$$

The strains in each layer of steel are determined by similar triangles in the strain distribution (Figure 7). The corresponding stresses are then given as follows:

$f_{s1} = \varepsilon_{s1}E_s = 0.0057 \times 2 \times 10^5 \rightarrow 413.68\ MPa$
$f_{s2} = \varepsilon_{s2}E_s = 0.0028 \times 2 \times 10^5 \rightarrow 413.68\ MPa$
$f_{s3} = \varepsilon_{s3}E_s = 0 \times 2 \times 10^5 = 0\ MPa$
$A_{s1} = 803.75\ mm^2$
$A_{s2} = 390\ mm^2$
$A_{s3} = 803.75\ mm^2$

$$\phi M_n = \phi\left\{(Ey_t^4 + Fy_t^3 + Gy_t^2 + Hy_t + I) + \sum A_{si}f_{si}d_i\right\} \dots\dots\dots Eq.2(b)$$

$$E = \frac{-b(E_c - E_2)^2}{16f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2 = -1.97 \times 10^{-4} kN/mm^3$$

$$F = b\left(c - \frac{h}{2}\right)\frac{(E_c - E_2)^2}{12f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2 + \frac{b(E_c - E_2)}{3}\left(\frac{\varepsilon_{ccu}}{c}\right) = 0.0841\ kN/mm^2$$

$$G = -\left\{\frac{b}{2}f_c' + b\left(c - \frac{h}{2}\right)\frac{(E_c - E_2)}{2}\left(\frac{\varepsilon_{ccu}}{c}\right)\right\} = -13.347\ kN/mm$$

$$H = bf_c'\left(c - \frac{h}{2}\right) = 934.122\ kN$$

$$I = \frac{bc^2}{2}f_c' - bcf_c'\left(c - \frac{h}{2}\right) + \frac{bc^3E_2}{3}(\varepsilon_{ccu}) - \frac{bcE_2}{2}\left(c - \frac{h}{2}\right)(\varepsilon_{ccu}) = 99920.47\ kN.mm$$

The distances from each layer of steel reinforcement to the geometric centroid of the cross-section are shown below.

$d_1 = 125mm, d_2 = 0\ \&\ d_3 = 125mm$

Considering tensile force due to bending as negative

$$\phi M_{n(B)} = 0.65\{(-1.97 \times 10^{-4} \times 96.61^4 + 0.0841 \times 96.61^3 - 13.347 \times 96.61^2 + 934.122 \times 96.61 + 99920.47) + (803.75 \times 413.68 \times 125 + 390 \times 413.68 \times 0)\}$$

$$\phi M_{n(B)} = 124372.865\ kN.mm$$

Point C:

Following the same procedure of point B the design axial and bending moment capacities are

$\phi P_n = 1292.742\ kN$
$and\ \phi M_n = 187612.485\ kN.mm$

Where, the corresponding parameters are written below.

$$\varepsilon_{s3} = \varepsilon_{sy} = \frac{f_y}{E_y} = \frac{413.68}{2 \times 10^5} = 0.00207\ mm/mm$$

$$c = d\frac{\varepsilon_{ccu}}{\varepsilon_{ccu} + \varepsilon_{sy}} = 312.5\frac{0.00709}{0.00709 + 0.00207} = 241.81\ mm$$

$A = -4.38 \times 10^{-4}\ kN/mm^3$
$B = 0.0983\ kN/mm^2$
$C = -7.355\ kN/mm$
$D = 2043.716\ kN$
$E = -3.29 \times 10^{-4}\ kN/mm^3$
$F = 0.0898\ kN/mm^2$
$G = -9.117\ kN/mm$
$H = 406.844\ kN$
$I = 147797.7\ kN.mm$

The strains in each layer of steel are determined by similar triangles in the strain distribution (Figure 7). The corresponding stresses are then given as follows:

$f_{s1} = \varepsilon_{s1}E_s = 0.0053 \times 2 \times 10^5 \rightarrow 413.68\ MPa$
$f_{s2} = \varepsilon_{s2}E_s = 0.0016 \times 2 \times 10^5 = 319.19\ MPa$

$f_{s3} = \varepsilon_{s3}E_s = -0.00207 \times 2 \times 10^5$
$= -413.68\ MPa\ (tension)$
$A_{s1} = 803.75\ mm^2$
$A_{s2} = 390\ mm^2$
$A_{s3} = 803.75\ mm^2$
$d_1 = 125mm, d_2 = 0\ \&\ d_3 = 125mm$

ID: SEE 035

NUMERICAL MODELLING OF REINFORCED CONCRETE BEAMS WITH OPENING

S.R. Chowdhury[1*], R.B. Hafiz[1], S. Ahmed[1], & S. Barua[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka,Bangladesh.*
**Corresponding Author, <srchowy@yahoo.com>*

ABSTRACT

The provision of opening in beams is kept to facilitate the passage of utility pipes and service ducts. Normally opening is placed centrally in depth in order to avoid reducing the shear area near the supports. In this study, a three-dimensional nonlinear numerical finite element method using ANSYS 10.0, a finite element (FE) software has been employed to simulate the simply supported reinforced concrete beams consisting of circular openings with varying diameter. RC beam model is created using 3D SOLID65 element representing both linear and non-linear behaviour of concrete, SOLID45 3D Structural Solid element representing steel plates and LINK8 3D SPAR element representing reinforcing bars. The inherent assumption is that there is full displacement compatibility between the reinforcement and the concrete and no bond slippage occurs. First, the FE beam model is verified against experimental test data of RC solid beam without opening available in literature. Subsequently, a number of verified models of simply supported reinforced concrete beams with circular and square openings are loaded monotonically with two incremental concentrated loads. The dimensions of the full-size beams are 100 mm × 2050 mm×250 mm. The span between the two supports is 2000 mm. The circular openings vary from 60 mm to 150 mm. A model with equivalent square opening of 133 mm in width is also analysed. An attempt is made to study the effects of circular opening size and location on the behaviour of RC beams from cracking pattern and load-deflection.

Keywords: Opening, Finite Element, Modelling, Nonlinear.

INTRODUCTION

In the construction of modern buildings, many pipes and ducts are necessary to accommodate essential services like water supply, sewage, air-conditioning, electricity, telephone, and computer network. Usually, these pipes and ducts are placed underneath the soffit of the beam and, for aesthetic reasons, are covered by a suspended ceiling, thus creating a "dead space." In each floor, the height of this dead space that adds to the overall building height depends on the number and depth of ducts to be accommodated. An alternative arrangement is to pass these ducts through transverse openings in the floor beams, this arrangement of building services leads to a significant reduction in the headroom and results in a more compact design. Most engineers permit the embedment of small pipes, provided some additional reinforcement is used around the periphery of the opening. But when large openings are encountered, particularly in reinforced or pre-stressed concrete members, they show a general reluctance to deal with them because adequate technical information is not readily available. It is obvious that inclusion of openings in beams alters the simple beam behaviour to a more complex one. Unless special reinforcement is provided in sufficient quantity, the strength of such a beam may be reduced to a critical degree. Prentzas (1968), considered openings of circular, rectangular, diamond, triangular, trapezoidal and even irregular shapes. Although numerous shapes of openings are possible, circular and rectangular openings are the most common ones. Sometimes the comers of a rectangular opening are rounded off with the intention of reducing possible stress concentration at sharp comers, thereby improving the cracking behavior of the beam in service. Abdalla et al. (2003), used fibre reinforced polymer (FRP) sheets to strengthen the opening region in an experimental program. The test data reported by Somes and Corley (1974), indicated that when a small opening is introduced in the web of a beam, unreinforced in shear and the mode of failure remains essentially the same as that of a solid beam. Amiri and Masoudnia (2011), investigated the opening effects on the behaviour of concrete beams without additional reinforcement in opening region using FEM method.

In this study a three-dimensional nonlinear numerical finite element method using ANSYS 10.0, a finite element (FE) software, has been employed to simulate the simply supported reinforced concrete beams consisting of circular openings with varying diameter. An attempt is made to know the effects of circular opening size and location on the behaviour of RC beams from cracking pattern and load-deflection.

PROBLEM STATEMENT AND MODELING

This FEA calibration study includes modelling a concrete beam with the dimensions and properties corresponding to solid beam tested by Abdalla et al. (2003). The dimensions of the full-size beams were 100 mm×2050 mm×250 mm. The span between the two supports was 2000 mm. longitudinal reinforcements and shear stirrups are modeled throughout the beam. The goal of the comparison of the FE model and the beam from Abdalla et al. (2003) is to ensure that the elements, material properties and convergence criteria are adequate to model the response of the member and make sure that the simulation process is correct. Finite element modelling of the reinforced concrete beams with circular opening in varying diameters (150, 120, 110, 100, 80, 60 mm) is performed and in later a comparison will be made between circular opening (d=150mm) and equivalent square opening in the area (h=133 mm). Table 1 summarizes the model number for different opening diameter. the These beams have the same dimensions as the experimental beam tested by Abdalla et al. (2003). Space between the end of opening and the support is 200 mm. The centre of the circular opening is situated in 137.5 mm from the origin of coordinate system. Fig. 1(dimensions shown in the figure are in mm) shows the location of the circular opening inside the RC beams in this study. These simply supported beams were loaded incrementally to failure by applying two- point loads.

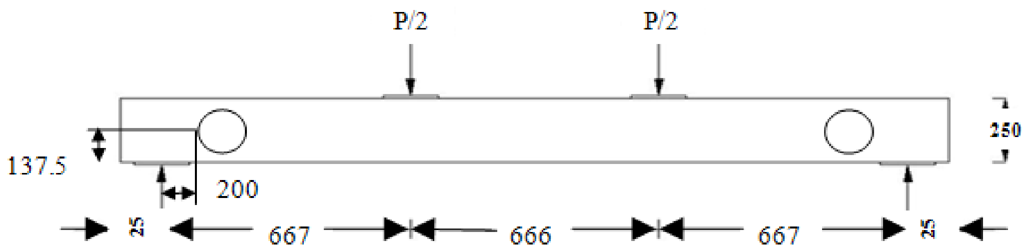


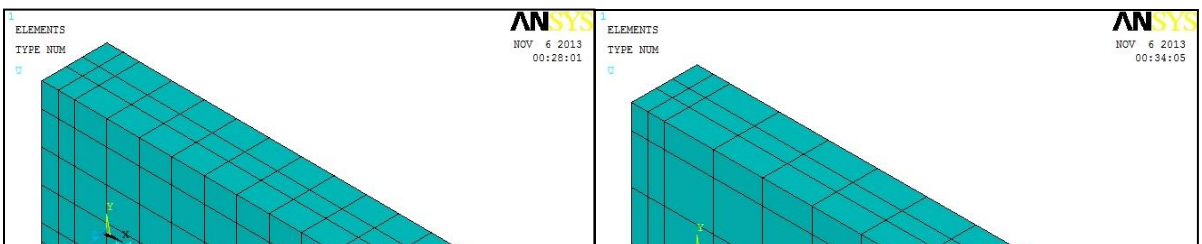
Fig. 1 Opening location in RC

Table 1: Different types of models used in this study

Model no.	1	2	3	4	5	6
Diameter of Circular Opening (mm)	60	80	100	110	120	150

Boundary conditions are needed to be applied at nodes in the supports to ensure that the model acts the same way as the experimental beam. The supports were modelled in a way that the roller and hinged supports were created. The force, P, is applied on all nodes that exist at the entire centreline of the plates.

In this research a convergence study was carried out to determine an appropriate mesh density. Various mesh sizes are examined in ANSYS.



(a)

(b)

(a) (b)

Fig. 2 Beam model in ANSYS with (a) Mesh 70 mm and (b) Mesh 100 mm

The ultimate load was obtained for each mesh size as tabulated in Table 2.

Mesh size (mm)	Mesh 40	Mesh 70	Mesh 100
Ultimate load (N)	77021	76286	75368

Table 2 Different mesh sizes and corresponding ultimate loads

From this table, it can be observed that the obtained ultimate load for mesh size 40mm (77,021N) is nearest to the ultimate load of experimental beam (83000N). For this reason, the mesh size equal to 40 mm was chosen for this study. Fig. 3 shows the load-deflection relationship for different mesh sizes.

Here, in this study, the discrete model was used to model reinforcement. Fig. 4 shows that the ultimate load for solid beam without opening obtained from the experimental test was 83,000N, while the ultimate load extracted from ANSYS analysis outputs is 77,021N for the model without stirrup & 79603N for the model with stirrup. Since the difference of ultimate load between the model with & without stirrup was 3.24% therefore the model without stirrup for further analysis purpose has been used to save some time. The difference of ultimate load between the model and experiment is about 7 % and it is proven that ANSYS software is an appropriate method to predict the behaviour of RC beams accurately.

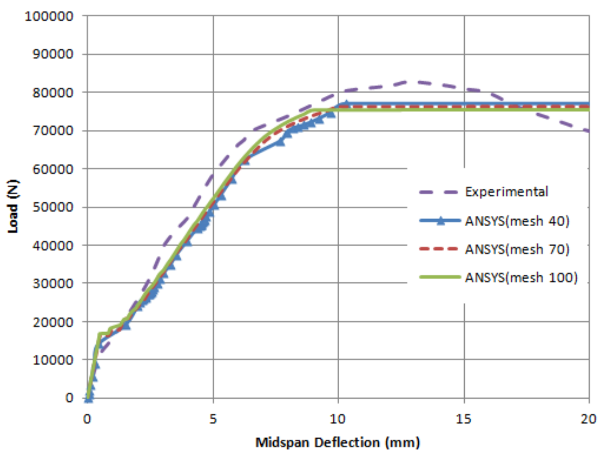


Fig. 3 Load-deflection relationships for different models with different mesh size

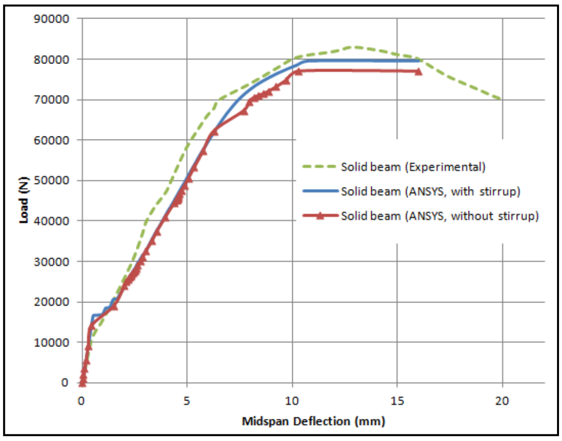
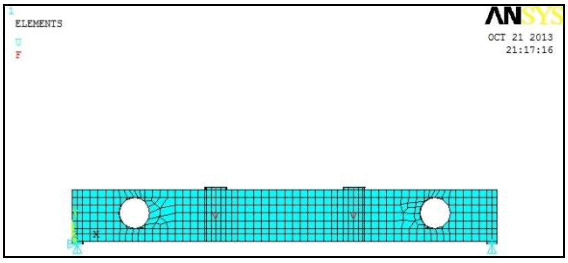
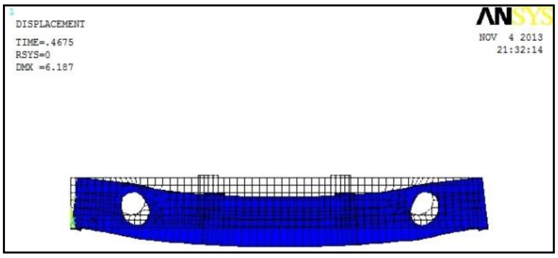


Fig. 4 Load-deflection relationships between simulated beam with stirrup and without stirrup by ANSYS and experimental beam.

RESULTS AND DISCUSSIONS



(a)

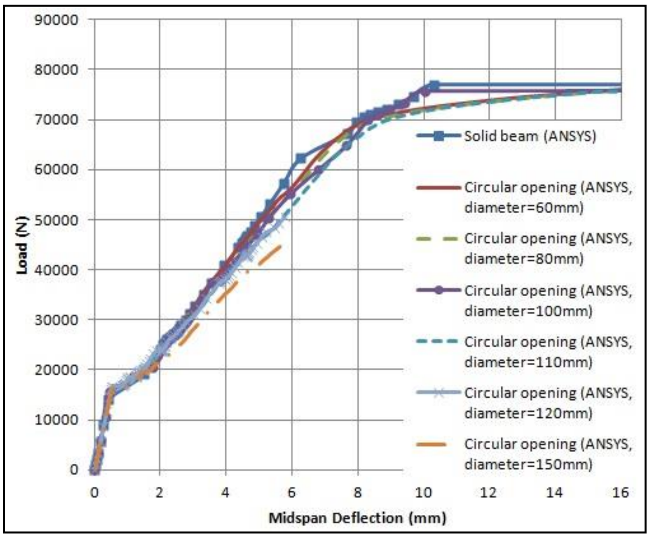


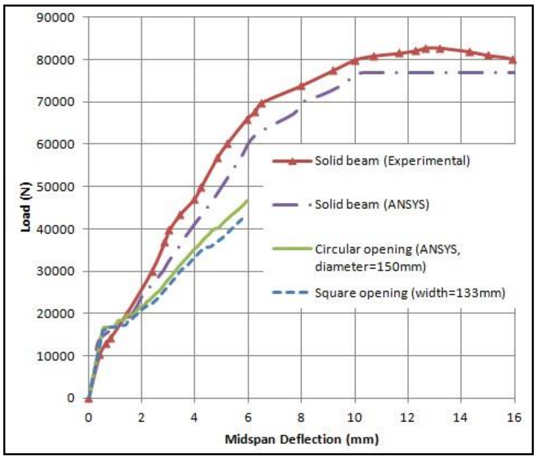
(b)

Fig. 5 (a) Model 6 with circular opening (diameter = 150 mm) and (b) its deflected shape for ultimate load.

Effects of Circular Opening Sizes with Varying Diameters

Figure 5 shows the modelling generated at ANSYS with deflected shape at collapse load. Figure 6(a) presents load-deflection curves for the solid beam and beams with different openings mentioned in this study. Difference between the largest opening with 110 mm (Model 4) in diameter and solid beam is around 2% reduction in the ultimate load capacity. This value is negligible. Hence, introducing the circular opening with diameter less than 44% of the depth of the beam has small effect on the behaviour of the beam because the depth of the compression chord is greater than the depth of compressive stress block. Whereas, for Model 5 and Model 6, where circular opening with diameter more than 44% of the depth of the beam (without special reinforcement in opening zone) , the depth of the beam has significant effect on the behaviour of the beam since tensile stress of the longitudinal reinforcement do not reaches to its yield stress even at the ultimate stage of the beam. Suppose for Model 6, obtained value of the ultimate load capacity from ANSYS analysis was 46,750 N, whereas the corresponding value for the solid beam (without opening) obtained from ANSYS was 77,021 N. Therefore, the reduction about 39.3% occurs in ultimate load capacity of the solid beam by creating the circular opening with 150 mm diameter. This reduction in the ultimate load capacity of the beam is considerable. In other words, by creating the circular opening with diameter equal to 60% of the depth of the beam, the capacity of the ultimate load reduces to 39.3%.





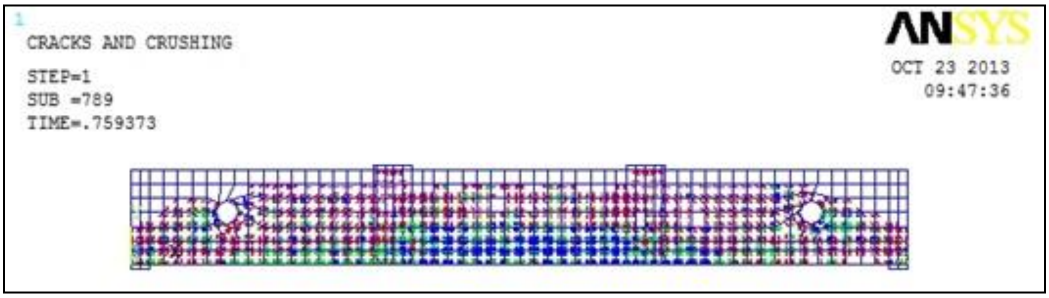
(a) (b)

Fig. 6 (a) Load vs. Deflection Curve for solid beam and all beams having different openings discussed in this study (b) Load-deflection curves for the solid beam, circular opening and beam with equivalent square opening.

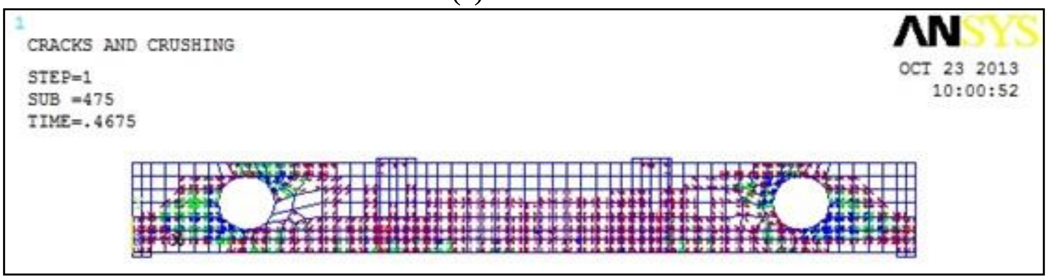
Effect of Circular Opening Compared to the Equivalent Square Opening

In this section the RC beam with square opening (width = 133 mm) has been used and the results compared to its equivalent circular opening (diameter = 150 mm). From Fig. 6(b), obtained value of the ultimate load capacity for square opening from ANSYS analysis is 42,270 N, while the corresponding value for the solid beam (without opening) and equivalent circular opening obtained from ANSYS are 77,201 N and 46,750 N, respectively. The difference in the ultimate load capacity between circular opening (d = 150 mm) and equivalent square opening (w = 133 mm) is about 9.58%.

Crack Pattern at Ultimate Load



(a)



(b)

Fig. 7: Crack pattern at the ultimate load for (a) Model 1 and (b) Model 6.

It shows that circular opening reduces the ultimate load less than the equivalent square opening and the circular opening has more strength than square opening. The main reason for more reduction of the ultimate load capacity in square opening is that the existing orthogonal corners cause to produce the stress concentration at these corners. Figure 7 shows crack pattern at the ultimate load for model 1 and Model 6. For model 1, the main cracks in this beam appear at the midspan and the failure caused by flexural cracks is quite similar to the solid beam without opening. Whereas for model 6, the main cracks occur at the bottom and top chord; in addition, the crack path led to the failure in the openings extends with 45 degrees from loading points toward supports and from the crack propagation, it is observed that the failure mode is shear at the opening region.

CONCLUSION

Findings of this study are given below:

- For circular opening with diameter less than or equal to 44% of the depth of the beam (without special reinforcement in opening zone) , tensile stress of the longitudinal reinforcement reaches to its yield stress before reaching to the ultimate stage of the beam.
- For circular opening with diameter more than 44% of the depth of the beam (without special reinforcement in opening zone) , tensile stress of the longitudinal reinforcement do not reaches to its yield stress even at the ultimate stage of the beam.
- The circular opening has more strength than equivalent square opening with difference of 9.58% in ultimate load capacity since stress concentration was found at the existing orthogonal corners of the model with square opening

The following recommendations are suggested for future researches which were not covered in the present study

- Based on the finite element method, future studies can include different types of loading to further test the design method. This comprises studies with continuous beams, multiple load points, cyclic loading and different opening shapes and also includes examining the vertical location of the transverse openings in the RC beams.
- Strengthening the region around the opening in the RC beams by providing high strength concrete.

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ID: SEE 038

NUMERICAL APPROACH TO FREQUENCY DOMAIN GUST RESPONSE ANALYSIS OF STRUCTURES

M. R. Awall[1*], H. Yamaguchi[2], T. Humyra[3] & N. H. M. K. Serker[1]

[1] *Department of Civil Engineering, RUET, Rajshahi, Bangladesh.*
[2] *Department of Civil and Env. Engg., Saitama University, Saitama, Japan.*
[3] *Department of Civil Engineering, RSTU, Natore, Bangladesh.*
**Corresponding Author, <robi95@gmail.com>*

ABSTRACT

Gust response is a kind of random vibration induced by turbulence in the oncoming wind. The existing gust analysis approach is especially challenged for involving new technologies and flexible structures are designed when subjected to atmospheric turbulence. In this study, frequency domain approach is established for gust response analysis of structures. Firstly, transfer functions are derived from the equation of motion of structures. Then according to the stochastic process, the power spectral density of the time-varying wind force is computed as function of the power spectral density of the time varying velocity. For distribution random excitation of structures, autocorrelation function is introduced and by solving the equations, Root Mean Square (RMS) displacement of structures is found. By using this frequency domain gust response approach, RMS displacements are calculated for three different types of structures including flexible structure, and the results as obtained are found quite acceptable.

Keywords: Gust Response, Frequency Domain Analysis, Random Vibration, Power Spectral Density, Root Mean Square Displacement.

INTRODUCTION

Gust excitation is caused by unsteady fluctuating forces induced by wind turbulence. It occurs over a wide range of wind speeds and normally increases monotonically with increasing wind speed. When a structure is immersed in a wind field, the structure will be subjected to static and dynamic wind forces caused by mean and fluctuating wind speeds, respectively. The purpose of gust analysis is the prediction or estimation of total gust response of structures. Gust response prediction other than aeroelastic instability is of major concern in the wind resistance design and evaluation of windinduced vibrations for flexible structure. The gust response significantly increases with the increasing span lengths of structure, which may lead to serious fatigue damage to structural components and affect functionally of the structure (Awall et al. 2009). The gust force should be considered in the design stage. To model the action of gust wind load, the gust forces resulting from turbulent wind and the self-excited forces due to the wind-structure interaction should be taken into account (Xu, 2013).

The gust response analysis can be treated by either frequency-domain approach or Time-domain approach. Early works on computational gust prediction carried out for airplane wings by Liepmann (1952). The spectral analysis and statistical computation method (frequency-domain approach) was introduced. It is generally agreed that the problem of computational dynamic response of suspension bridges or so-called line-like structures subjected to random gust loads in turbulent wind has proposed by Davenport (1962). Apparently, this computational gust prediction cored on spectral analysis and statistical computation in associated with modal-approached structural analysis. Though some assumptions and uncertainties accepted for their existence, but so far Davenport’s gust response prediction basically validates for structural applications and bridges. Furthermore, his method based on early research works and significant contributions by Liepmann (1955).

Recent developments on analytical models based on time-domain approach (Cheng et al., 2007; Chen and Kareem, 2002) or coupled flutter and buffeting forces (Liu and Xiang, 2000; Katsuchi et al., 1999) have highlighted in the buffeting response prediction. This time-domain analysis, however, is time-consuming and sophisticated. Frequency-domain gust response analysis has been applies for linear structural behaviors that being increasingly common-sense for gust response prediction of flexible structure. This study presents a general frequency domain framework for predicting gust response analysis of structures. Firstly, transfer functions are derived from the equation of motion of structures. Then according to the stochastic process, the power spectral density of the time-varying wind force is computed as function of the power spectral density of the time varying velocity. For distribution random excitation of structures, autocorrelation function is introduced and for estimating Root Mean Square (RMS) displacement of structures.

TRANSFER FUNCTION OF STRUCTURES

The basic equations of motion of an inclined cable are derived by Yamaguchi and Ito (1979) and Yamaguchi (1997). After considering damping, the equation of motion of transmission line cable can be re-write as the following form.

$$m\frac{\partial^2 y(x,t)}{\partial t^2} + c\frac{\partial y(x,t)}{\partial t} - T\frac{\partial^2 y(x,t)}{\partial s^2} = F(x,t) \qquad (1)$$

Where $y(x,t)$ is the deflection of the cable at time t and distance x, m is the mass per unit length and T is the cable tension, F is the gust force.

Generalized coordinate modal is needed to solve the above equation.

$$y(x,t) = \sum_{i=1}^{\infty} \phi_i(x)\bar{q}_i(t) \qquad (2)$$

Putting Eq. (2) in Eq. (1).

$$m\sum_{i=1}^{\infty}\phi_i(x)\ddot{q}_i(t) + c\sum_{i=1}^{\infty}\phi_i(x)\dot{q}_i(t) - T\sum_{i=1}^{\infty}\phi''_i(x)q_i(t) = F(x,t) \qquad (3)$$

Multiplying this equation by $\varphi_n(x)$, integrating and applying the orthogonally relationship.

$$M_n\ddot{q}_n(t) + C_n\dot{q}_n(t) - \left(\frac{T n\pi}{m L}\right)^2 M_n q_n(t) = F^*_n(x,t) \qquad (4)$$

Introducing here taught cable mode shape and frequency equation ($\phi = \sin(\frac{n\pi x}{L})$ and $\omega = (\frac{n\pi}{L})\sqrt{\frac{T}{m}}$) of Eq. (3).

[1] To find out transfer function $H(\omega)$, putting $F_n(t) = e^{i\omega t}$ and $q(t) = H(\omega)e^{i\omega t}$ in the Eq. (5).

$$\ddot{q}_n(t) + 2\xi_n\omega_n\dot{q}_n(t) - \omega_n^2 q_n(t) = \frac{F^*_n(x,t)}{M_n} \tag{5}$$

Here, $M_n = \int_0^L \phi_n(x)^2 m(x)dx$; By putting $\varphi_n(x)$ value integrating and putting limit, $M_n = \frac{mL}{2}$

$$\left|H_n(f_f)\right|^2 = \frac{1}{M_n^2 (2\pi f_n)^4} \cdot \frac{1}{\left[\left(\frac{f_f}{f_n}\right)^2 - 1\right]^2 + 4\xi_n^2\left(\frac{f_f}{f_n}\right)^2} \tag{6}$$

Another example the multi I-girder bridge is considering for gust response analysis. Whose total length is 660 m and main span is 183 m. Main span of the bridge can be model as a simple supported beam. Equation of motion of the simple supported beam is

$$m\frac{\partial^2 y(x,t)}{\partial t^2} + c\frac{\partial y(x,t)}{\partial t} + EI\frac{\partial^4 y(x,t)}{\partial x^4} = F(x,t) \tag{7}$$

Where, $y(x,t)$ is the deflection at time t and distance x, m is the mass per unit length and EI is the bending stiffness. In a similar procedure transfer function of multi I-girder bridge is formulated.

$$\left|H_n(\omega)\right|^2 = \frac{1}{M_n^2\omega_n^4} \cdot \frac{1}{\left[1 - \left(\frac{\omega_f}{\omega_n}\right)^2\right]^2 + 4\xi_n^2\left(\frac{\omega_f}{\omega_n}\right)^2} \tag{8}$$

MODEL OF THE GUST LOAD

Gust force acting on transmission line can be written as the following form.

$$F_n^*(x,t) = \int_0^L F_L(x,t)\phi_n(x)\,dx \tag{9}$$

For finding out the gust response, apply the fluctuating force, which is

$$F_L(x,t) = C_D \rho U D_c v(x,t) \tag{10}$$

Where, C_D is the drag coefficient and evaluated by wind tunnel test, ρ is the air density U is the mean wind velocity, Dc is the conductor diameter, $v(t)$ is the random vertical fluctuating component. Putting the mode shape and fluctuating force in equation (9) and integrating all over the length gives.

$$F_n^*(x,t) = C_D \rho \bar{U} D_c L v(x,t)\alpha_n; \quad \text{where,} \quad \alpha_n = \begin{cases} \dfrac{2}{n\pi} & n = 1,3,5\ldots\ldots \\ 0 & n = 2,4,6\ldots\ldots \end{cases} \tag{11}$$

According to stochastic process, the power spectral density of the time-varying wind force $S_{P_n}(f_f)$, can be computed as function of the power spectral density of the time varying velocity, *v(t).*

$$S_F(\omega_f) = \left(\frac{C_D \rho \bar{U} D_c L \alpha_n}{n\pi}\right)^2 S_v'(\omega_f, \Delta_{ij}) \tag{12}$$

Here, Δ_{ij} is the separation distance between two points.

Introducing spatial correlation between gust (Its gives the spatial distribution of gust).

$$S_F(\omega_f) = \left[\frac{C_D \rho \bar{U} D_c L \alpha_n}{n\pi}\right]^2 R(f_f, \Delta_{ij}) S_v'(\omega_f) \tag{13}$$

According to davenport experimental data, spatial correlation, $R(f_f,\Delta_{ij}) = \left(e^{-c\omega_f\Delta_{ij}/\bar{U}}\right)^2$; Where, c is the constant, ranges from 5 to 15 for vertical direction and from 10-40 for horizontal direction (Ghiocel, 1975). For structural design purposes, there are several formulas in the literature to find out the $S_v'(\omega_f)$, Here considering Kaimal (1972) proposed vertical spectrum formula.

$$S_v'(\omega_f) = \frac{2 f^* u_*^2}{f_f (1 + 5.3 f^*)^{5/3}}; \quad \text{Here, } u_* = \frac{k\bar{U}}{\ln\frac{z}{z_0}} \text{ and function } f^* = \frac{f_f z}{\bar{U}} \tag{14}$$

Where, k is von karman constant usually take 0.4; z is height of conductor above the surface; z_0 is surface roughness length. So, putting this all value in Eq. (13) gives.

$$S_{F_n}(\omega_f) = \left(\frac{C_D \rho \bar{U} D_c L \alpha_n}{n\pi}\right)^2 . e^{-c\omega_f\Delta_{ij}/\bar{U}} . \frac{2k^2 \bar{U} z}{\left(\ln\frac{z}{z_0}\right)^2 . \left(1 + 5.3\frac{f_f z}{\bar{U}}\right)^{5/3}} \tag{15}$$

Also, from similar procedure Power Spectral Density of gust force of multi I-girder bridge can be written as.

$$S_{F_n}(\omega_f) = \left(\frac{C_D \rho \bar{U} d L \alpha_n}{n\pi}\right)^2 . e^{-cw\omega_f\Delta_{ij}/\bar{U}} . \frac{200 k^2 \bar{U} z}{\left(\ln\frac{z}{z_0}\right)^2 . \left(1 + 50\frac{\omega_f z}{\bar{U}}\right)^{5/3}} \tag{16}$$

SOLVING THE PROBLEM

For distribution random excitation of the structure, the space-time correlation of $F_n^*(x, t)$ is as.

$$\int_0^L\int_0^L E\left[F^*_j(x_1,t)F^*_k(x_2,t+\tau)\right]=E\left[F^*_j(t)F^*_k(t+\tau)\right]=R_p(\tau) \qquad (17)$$

Here, $R_p(\tau)$ is the autocorrelation. And, the local time average energy density which is mean square displacement at the location x is

$$E(y^2)=E\left[\sum_{j=1}^{\infty}\sum_{k=1}^{\infty}\phi_j(x)\phi_k(x)\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}h_j(\theta_1)h_j(\theta_2)R_p(\theta_1-\theta_2)d\theta_1 d\theta_2\right] \qquad (18)$$

Now introducing this equation by the spectral density $S_F(f_f)$, which is $R_p(\theta_1-\theta_2)=\int_{-\infty}^{\infty}S_F(f)e^{if(\theta_1-\theta_2)}df$; and using the complex frequency response function, gives.

$$E(y^2)=E\left[\sum_{j=1}^{\infty}\sum_{k=1}^{\infty}\phi_j(x)\phi_k(x)\int_{-\infty}^{\infty}S_F(f_f)H_j(f)H_k(f)df\right]$$ and by introducing transfer function.

$$E(y^2)=\sum_{n=1}^{\infty}\int_{-\infty}^{\infty}\left[\frac{1}{M_n^2(2\pi f_n)^4}\cdot\frac{1}{\left[\left(\frac{f_f}{2f_n}\right)^2-1\right]^2+4\xi_n^2\left(\frac{f_f}{f_n}\right)^2}\right]\left[C_D\rho\bar{U}D_cL\right]^2.e^{\frac{-c^j f_f\Delta_j}{\bar{U}}}\cdot\frac{2k^2\bar{U}z}{(\ln\frac{z}{z_0})^2.(1+5.3\frac{f_f z}{\bar{U}})^{5/3}}\sin^2\frac{n\pi x}{L} \qquad (19)$$

$$\text{R.M.S. displacement} = \sqrt{E(y^2)}$$

NUMERICAL EXAMPLES AND DISCUSSIONS

To illustrate the proposed methodology, two different (8 and 2 conductors) bundle conductors transmission lines (transmission line A and B) and simple supported multi I-girder bridge were chosen as examples. Span length of transmission line-A is 615 m and transmission line-B is 439 m. Different parameters values were considered from company supplied data and are given in Table 1.

Table 1 Parameter values consider in numerical examples

Parameter	**Transmission line-A (8 conductors)**	**Transmission line-B (2 conductors)**	**Multi I-girder bridge**
Air density, ρ	1.218 kg/m^3	1.218 kg/m^3	1.225 kg/m^3
Drag coefficient, C_D	0.77	0.9	2.04
Von karman constant, k	0.4	0.4	0.4
Height of cable/deck above the ground surface, z	109 m	100 m	28.05 m (92 ft)

Surface roughness length z_0	22 cm for I_u= 20% 16 cm for I_u=15%	21.6 cm for I_u=15% 16 cm for I_u=10%	0.03 cm
Critical damping ratio, ξ	0.004	0.004	0.005
Constant, $c^{/}$	5	5	2
Δ_{ij}	2 m	2 m	3 m

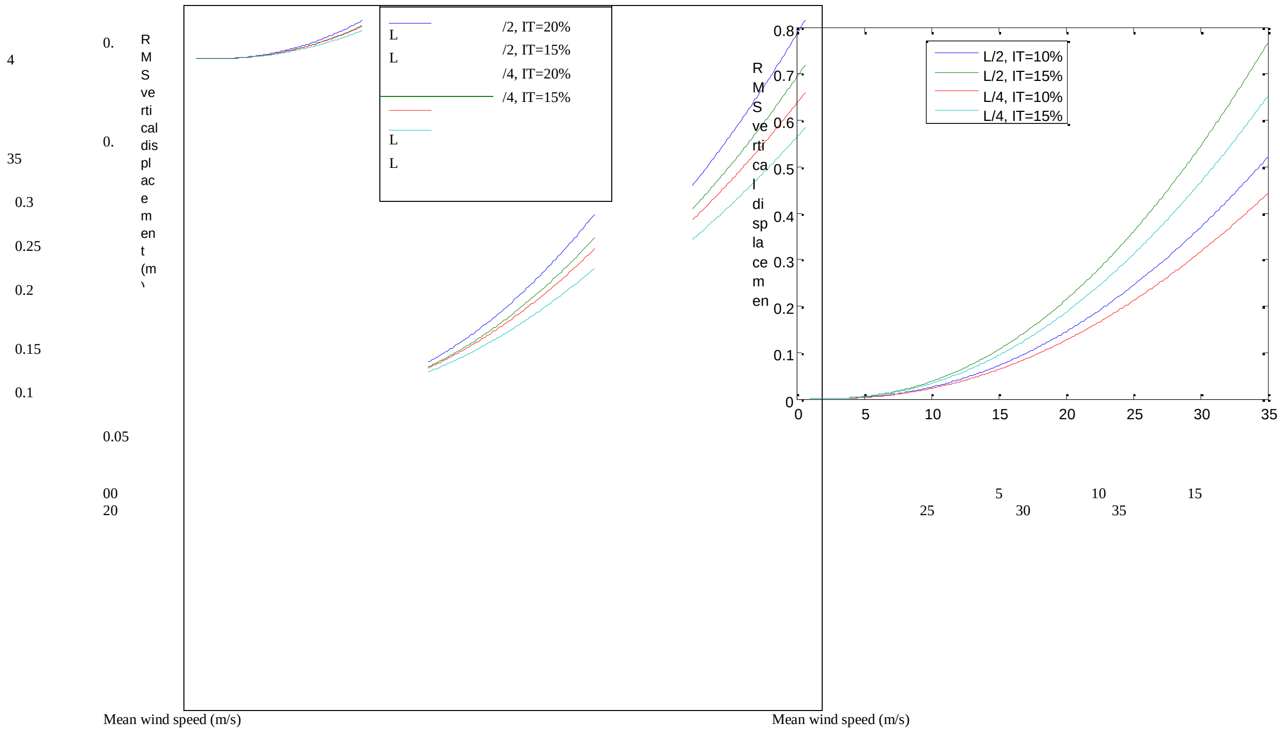

Fig.1 RMS vertical displacement of 8 conductors transmission line-A

Fig.2 RMS vertical displacement of 2 conductors transmission line-B

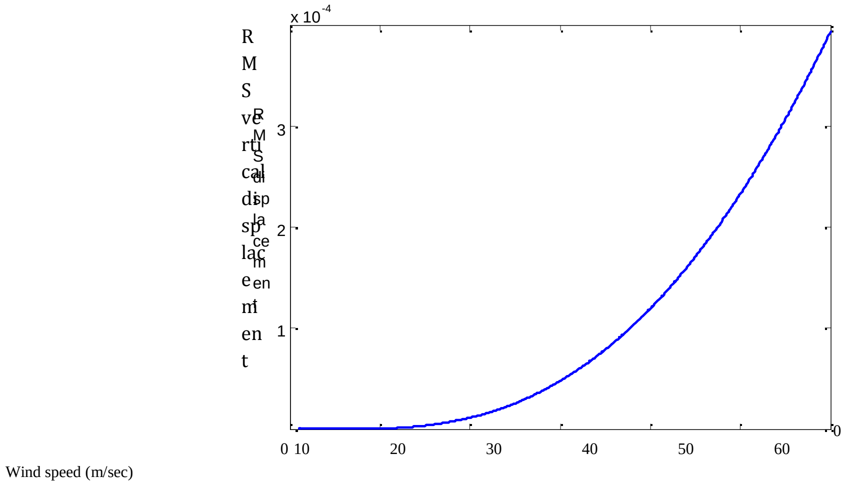

Fig.2 RMS vertical displacement of multi I-girder bridge

RMS vertical displacement at mid span and quarter span of transmission lines are calculated by using Eq. (19), and the results are shown in Fig. 1 and Fig. 2. Also, the RMS vertical displacement at mid span of multi I-girder bridge is shown in Fig. 3. From these figures, it can be seen that gust vibrations increase as the mean wind velocity increases that is gust response as a function of mean wind velocity. This is mainly due to the aeroelasticity introduced by motion-induced forces that give natural frequencies and critical damping ratios dependent on the mean wind velocity. At mid span gust responses are larger than the quarter span gust response. Also, the gust response is depending on turbulence intensity that is higher the turbulence intensity larger the gust response obtain shown in Fig. 1 and 2. Among two transmission lines, gust response of transmission line-B is larger than the transmission line-A, due to smaller stiffness of transmission line-B. Also the multi I-girder bridge gust response is much smaller than the transmission lines responses. Because, transmission line is much more flexible structure and the stiffness of multi I-girder bridge is larger than transmission line.

CONCLUSION

This methodology for calculating frequency domain gust response analysis of structures has been presented, which includes the stochastic process. For distribution random excitation of structures, autocorrelation function is introduced and by solving the equations, Root Mean Square (RMS) displacement of structures are found. By using this frequency domain gust response approach, RMS displacements are calculated of three different types of structures including flexible structure. Gust responses increase as the mean wind velocity increases that are gust response as a function of mean wind velocity. Also, the gust response is depending on turbulence intensity that is higher the turbulence intensity larger the gust responses are obtained. The structure which is more flexible, larger gust response is obtained and the results are quite acceptable.

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SEISMIC BEHAVIOR OF RC BUILDINGS HAVING DIFFERENT TYPES OF SLABS WITH OPENING

S. R. Chowdhury[1*], M. A. Alam[1], M .A .Hasan[1] & T. I. Tamzid[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <srchowy@yahoo.com>*

ABSTRACT

Under seismic loading, floor and roof systems in reinforced concrete (RC) buildings act as diaphragms to transfer lateral earthquake loads to the vertical lateral force-resisting system (LFRS) and, eventually to the foundations. Diaphragm stiffness effects the distribution of lateral loads to the lateral-load-resisting elements. Slab openings in floor systems may likely causes irregularities in the horizontal plane according to the earthquake code. In practice, two-way slabs take various forms and only two types of slab have been discussed in this study; one is edge-supported slab where slab is supported on all four sides and other is flat plate where slab is beamless and directly carried by columns. Relatively small openings usually are not detrimental in edge-supported slabs. The importance of small openings in slabs supported directly by columns depends upon the location of the opening with respect to the columns. From structural point of view, they are best located away from the columns, preferably in the area common to the slab middle strips. To validate this phenomenon, a 7- story, 7 × 3 bays RC building having 3m span and floor height of 3 meters, with the help of finite element software ETABS under earthquake loads by the method of equivalent static analysis is selected. Apart from this validation, the behaviour of multi-story RC buildings having three types of slabs (flat plates, flat plates with edge beam and edge supported slabs) with openings under seismic forces are compared according to the parameters like opening size, opening locations, no. of story and lateral displacement.

Keywords: Opening, Finite Element, Modelling, Slab.

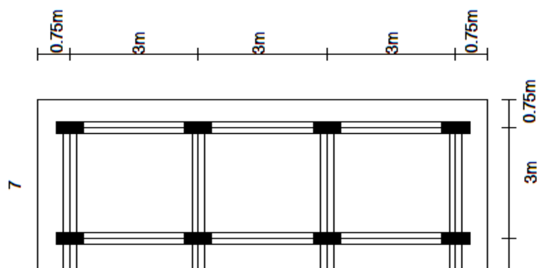
INTRODUCTION

Generally floor and roof systems are designed to carry gravity loads and transfer these loads to supporting beams, columns or walls. Furthermore, they play a key role in distributing earthquake-induced loads to the lateral load resisting systems by diaphragm action. The diaphragm acts as a wide flat beam that develops tension and compression on its edges. Introducing openings in existing reinforced concrete (RC) slabs can severely weaken the slabs due to the cut out of both concrete and reinforcing steel. For the purposes of design, two-way slab systems are divided into column and middle strips in two perpendicular directions. The column strip width on each side of the column centreline is equal to 1/4 of the length of the shorter span in the two perpendicular directions. The middle strip is bounded by two column strips. The permit of openings of any size in any new slab system, provided an analysis is performed that demonstrates both strength and serviceability requirements are satisfied. As an alternative to detailed analysis for slabs with openings, ACI 318-05 gives the chain saw with plunge cutting capabilities. Cutting openings in existing slabs should be approached with caution and avoided if possible. When cutting an opening in an existing slab, the effect on the structural integrity of the slab must be analysed. It’s advisable to analyse the slab for excess capacity and possible moment redistribution before making the final decision on the sizes and locations of the openings (ACI 318-05). For openings in two-way beam-supported slabs, the shear is transferred to the column through the beams. The total width of openings often be up to 1/4 of the span, as long as the beams are left intact. Openings can be more problematic because they may intersect the portion of the slab used as a T-beam. Although the least desirable location, openings with maximum dimensions up to 1/8 of the span can often be located at the intersection of two middle strips. When removing an entire panel of slab between beams, it's often an advantage to leave enough of an overhang to allow development of reinforcing bars from adjacent spans. In this case, the beams should be checked for torsion because the balancing moments from the portion of the slab that was removed will no longer be present (Newman A., 2001). In blast resistant applications, the size of openings in slabs can become very significant when determining ultimate resistance to pressure. Openings tend to attract yield lines, but they don't automatically weaken a slab. Openings result in less surface area to collect load, and under some conditions, the ultimate strength of a slab can actually increase. When a slab has openings that are covered there is no reduction in surface area collecting load and the designer has to add into the analysis the effects of the additional blast load collected by the cover. As far as possible, opening in slabs should be located in zones where shear stressess are small and bending moment are small below maximum. However small opening for pipe sleeves etc can be made any where in slab. Such opening have to be provided with extra bars known as trimming bars all round the opening. These bars not only tie the free ends of the bars which are trimmed to form the opening but also serve as reinforcement to make the opening stable aginst deformation or any other type of failure. Opening in the negative zone of the short span middle strip should always be avodied. In case of large openings, the edges of the slab around the opening may be thickened to enable them to behave like trimmer beams for taking up additional moments and forces . When very large size opening are to be provided in slab such as for stairs, lift etc, it is necessary to provide special beams around the opening. Such beams should preferably be framed into ensure perfect stability of the opening. (Kumar S., 1994).

In this study, multi-story RC buildings having three types of slabs (flat plates, flat plates with edge beam and edge supported slabs) with openings under lateral forces are analysed in Finite Element Software, ETABS and later on comparing parameters like opening size, opening locations, no. of story and lateral displacement, critical opening locations and sizes for these all three types of RC buildings have been found.

PROBLEM STATEMENT AND MODELING

The typical building plan layout of 3D reinforced concrete moment resisting building frame is studied as shown as Fig.1. RC buildings with 7- story, 7 × 3 bays , 3m span and floor height of 3 meters having three types of slab system (flat plates, edge supported and flat plate with edge beam) under lateral (earthquake and wind) forces have been analysed . For each slab system, opening locations in slab as shown in Fig. 2 and three opening sizes (1.5m-1.5m, 1m-1m, & 0.5m-0.5m) are maintained. Linear elastic analysis is performed with the help of finite element software ETABS 9.6.0 under lateral loads in equivalent static method. In the present study, earthquake and wind load is preferred as a source of lateral loading on the building frame as set forth by the provision of Bangladesh National Building Code (BNBC, 1993). The frame members are modelled with rigid end zones and the floors are modelled as diaphragms rigid in-plane. Poison's ratios for concrete and values of Elastic modulus of concrete are taken as 0.20 and 24821128 KN respectively. The unit weight of concrete is 23.56 KN/m^3. The specified compressive strength of concrete and bending reinforcement of yield stress are taken as 25000KN and 415000KN respectively. The live load, floor finish and partition wall on floor has been taken $2KN/m^2$, $1KN/m^2$ and $2.25KN/m^2$ respectively. The cross-sectional area of column and beam are taken 30cm×63cm and 30cm×50cm respectively. Slabs are taken as 15.25cm thick for all models. The wind load is applied as per BNBC, 1993 considering exposure condition A and wind velocity 210 km/hr. Earthquake load is applied as per BNBC, 1993 considering Seismic Zone Factor 0.15, Site Coefficient 1.5, and Importance Factor 1.



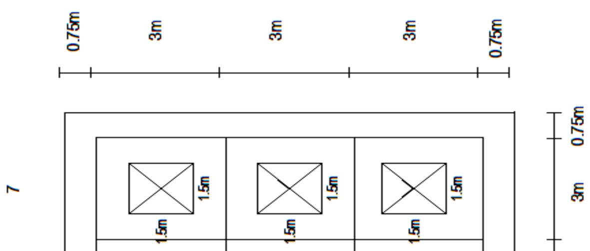
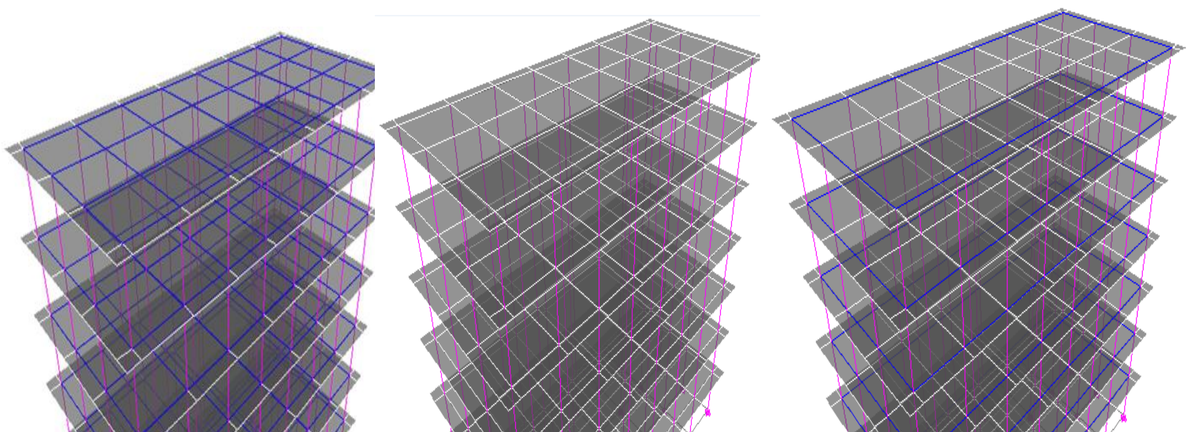


Fig.1 Plan view of seven storey building

Fig.2 Plan view for edge supported slab with opening 1.5m-1.5m

Opening locations are represented by a technique. Suppose openings at four corners are represented by A11, A17, A31 and A37 according to Fig. 2. Fig. 3 shows 3D views of seven storied building with (a) edge supported slab (b) Flat plate (c) Flat plate with edge beam which are obtained from ETABS.



(a) (b) (c)

Fig.3 3D view of seven storied building with (a) edge supported slab (b) Flat plate (c) Flat plate with edge beam from ETABS.

RESULTS AND DISCUSSIONS

In this paper lateral displacement only in short direction is presented since displacements in this direction are critical. It is found in BSc thesis, 2013 that among all the opening locations, lateral displacement of RC buildings with openings in slab at any corner (A11 or A17 or A31or A37 of Fig.2) is maximum and this is valid for buildings with all three types of slab systems which also validate the conception available in literature. Fig. 4 shows the variation of displacement with no. of story for all types of slab system and opening sizes with opening at corner. Edge supported slab with minimum opening size (0.5mx0.5m) concedes the minimum displacement whereas flat plate with maximum opening size (1.5mx1.5m) concedes highest displacement.

From Figs. 5(a), (b) and (c), it is concluded that for three types of building, maximum displacements occur at their largest opening sizes. Figure 5(d) indicates that displacement increases with the increasing of opening size for three types of building and the maximum displacement shows for the opening size of 1.5m-1.5m in flat plate building.

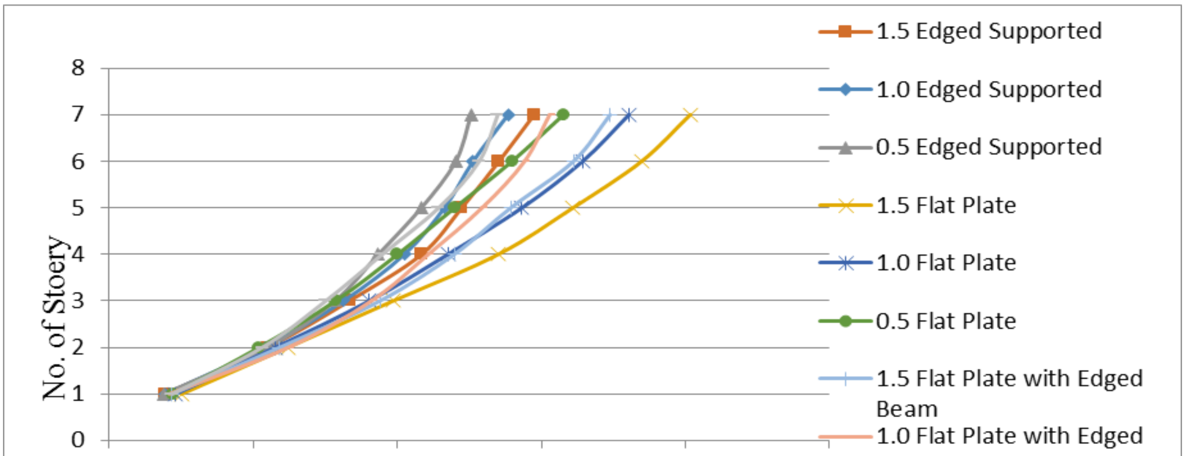


Fig. 4 No of storey Vs displacement (mm) for building slab system having with three sizes of opening placed at corner.

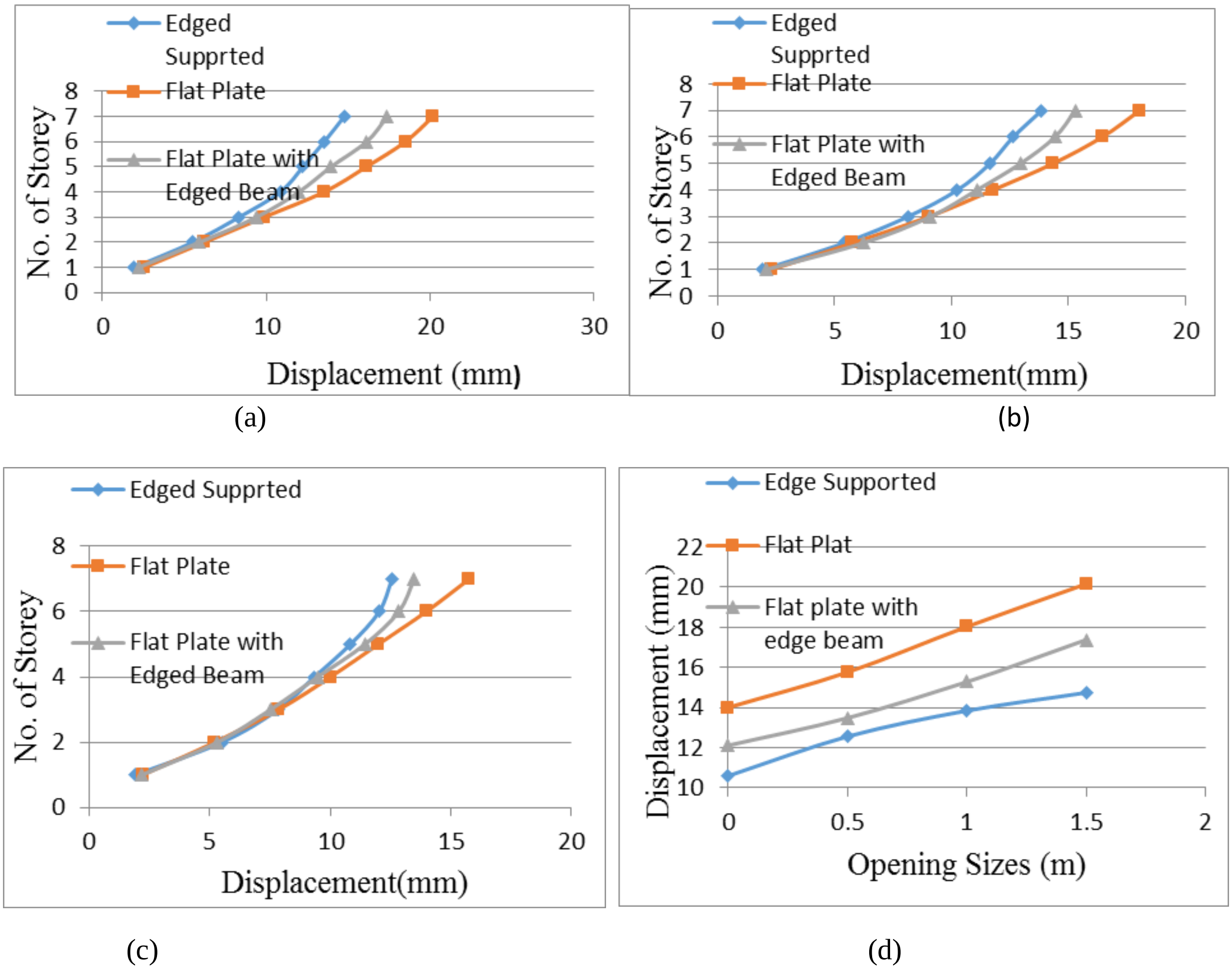


Fig.5 No of storey Vs displacement for different types of slab system having opening size of (a) 1.5m-1.5m (b) 1.0m-1.0m and (c) 0.5m-0.5m (d) Displacement (mm) Vs different Opening sizes (m) for three types of building.

CONCLUSION

Main findings from this study are given below:

- Lateral displacement of flat plate building is maximum than either the edge supported or flat plate with edge beam building for any types of opening size.
- RC building with edge supported slab system concedes the minimum displacement.

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- For all types of building with different slab systems, displacement increases with increasing opening sizes.
- For all types of slab system, opening location at corner gives maximum displacement.

For enrich this paper the following may consider for further future study.

- In this study finite element software ETABS 9.6.0 are used. There are also some other finite element software such as STAAD-Pro, ANSYS etc are available to analyze this kind of study.
- Only linear elastic analysis is performed in this study. To make comprehensive and complete comment non-linear dynamic analysis is highly recommended. This study can be performed by multi-modal analysis
- For the present study the analysis were performed for the symmetrical buildings to avoid torsional response under pure lateral forces. Further studies can be performed for the non symmetrical buildings.
- Only three different sizes of opening are considered. The study can be performed by changing the opening sizes and location of opening.

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SEISMIC VULNERABILITY ASSESSMENT OF EXISTING REINFORCED BUILDINGS AT SHOLOKBOHOR AREA

M. Rahman[1*], Fahmida Parvin[1], M. A. Yusuf[1], J. Uddin[1], M.G. Uddin[1], M.P. Alam[1]

[1] *Department of Civil Engineering, University of Information Technology and Sciences, Chittagong, Chittagong-4225, Bangladesh,*
**Corresponding Author, <mr_sunnyctg@yahoo.com>*

ABSTRACT

With the rapid growth of population and construction, natural disaster like earthquake causes enormous damage to the human lives as well as hazard to the total environment. Now-a-days, many of the existing buildings are lackof adequate earthquake resistance due to the insufficient knowledge in design and construction according to the building codes and prevalent earthquake resistant design practice. So it is important to check the seismic resistance for the structural configuration and structural degree of damage during strong motion/intensity of earthquake. This evaluation also helps to decide whether structural modifications are required or not. In this study, rapid individual building assessment based on statistical method as demonstrated in Istanbul, Turkey has been considered to evaluate seismic vulnerability of reinforced concrete buildings. Numbers of stories above ground level, soft story effect, pounding effect, topographic effects, overhang effects, local soil condition parameters are selected for “Walk down Evaluation”. Then six discriminant parameters are used, namely number of stories above ground level, minimum normalized lateral stiffness index, minimum normalized lateral strength index, normalized redundancy score, existence of soft stories and overhang ratio for preliminary evaluation. In this method, the discriminant scores obtained from two discriminant functions are combined in a way to classify existing buildings as “Low Risk Group”, “High Risk Group” and “Requiring for Further Study”. This paper presents seismic vulnerability assessment application in a City area entitled as “Sholokbohor” of Chittagong, the port city of Bangladesh. Finally results found in different stage of analysis are shown in GIS map.

Keywords: Vulnerability, Reinforced Concrete Buildings, Low Risk Group, Requiring for Further Study, High Risk Group.

INTRODUCTION

Bangladesh is possibly one of the country’s most vulnerable to potential earthquake threat and damage. An earthquake of even medium magnitude on Richter scale can produce a mass damage in major cities of the country, particularly Dhaka, Sylhet and Chittagong, without any previous notice. Construction of new buildings strictly following building code or development of future controls on building construction are the activities which will be functional in future. However, under the present stage of human occupancy, buildings, infrastructures and other physical structures of different areas of a city will not be equally vulnerable to any such shock. Earthquake vulnerability of any place largely depends on its geology and topography, population density, building density and quality, and finally the coping strategy of its people and it shows clear spatial variations.

The location of Bangladesh close to the boundary of two active plates: the Indian plate in the west and the Eurasian plate in the east and north. As a result the country is always under a potential threat to earthquake at any magnitude at any time, which might cause catastrophic death tolls in less than a minute. In the basic seismic zoning map of Bangladesh Chittagong region has been shown under Zone II with basic seismic coefficient of 0.15, but recent repeated shocking around this region indicating the possibilities of potential threat of even much higher intensity than projected. According to Global Hazard Assessment Program (GSHAP), the most hazardous division in Bangladesh is the Port City Chittagong. About 80-90 percent of buildings and physical infrastructures in Chittagong are vulnerable to future massive earthquake measuring 6-7 magnitudes on the RS, as most of these were not designed to withstand against seismic load.

Hilly terrain of this city corporation area may create huge land slide during a heavy earthquake. As, most of the building contain sloppy ground around them. Asian Disaster Preparation Center (ADPC) Seismic Hazards assessment has carried out at the Chittagong City Corporation Area of some buildings and found many vulnerable existing buildings. Now further evaluation of the seismic resistance and the assessment of possible damage are quite imperative in order to take preventive measures and reduce the potential damage to civil engineering structures and loss of human lives during possible future earthquakes.

A pilot application in a ward named by Sholokbohor of Chittagong City Corporation area has been conducted which is a most densely populated area of the city. Seismic risks of RC structures were evaluated and the concerned authority will be noticed of the probable disaster by providing these data. It is also necessary to find out a suitable retrofitting measure for earthquake vulnerable structure.

MATERIALS AND METHODS

Now the seismic vulnerability evaluation method can be classified into three several groups that is “Walkdown Evaluation Method”, “Preliminary Assessment Method (PAM)” & "Linear or Non-linear Analysis of the buildings" .The first approach Walkdown Evaluation is a simplest method and its goals is to classify the building according to the priority level by the immediate investigation of the building. The second stage that is known as Preliminary assessment methodologies (PAM) are applied when more in-depth evaluation of building stocks is required. These analyses require data on the dimensions of the structural and non-structural elements in the most critical story. It is possible to survey large building stocks by employing the preliminary evaluation methodology within a reasonable time span. The procedures in third tier employ linear or nonlinear analyses of the building under consideration and require the as-built dimensions and the reinforcement details of all structural elements.

Walkdown Evaluation Method:

The procedures in FEMA 154 (1988), FEMA 310 (1998) Tier 1 and the procedure developed by Sucuoglu and Yazgan (2003) are examples of walkdown survey procedures. Structural parameters that have to be observed during the field surveys and the value given to each parameter by the observer are “Number of Stories”, “Existence of a soft Story”, “Existence of heavy Overhangs”, “Apparent Building Quality”, “Existence of short Columns”, “Pounding Effect”, “Topographic Effects” and “Local Soil Conditions”. **Table 1** shows base score and vulnerability score for RC buildings depending on the above parameters and **Table 2** shows Vulnerability Scores Multipliers respectively.
The seismic performance scores PS can be obtained by

$$PS = (BS) - \Sigma\,(VSM) \times (VS) \tag{1}$$

Where,

BS = Base Score from **Table 1**

VS = Vulnerability Scores (VS) from **Table 1**

VSM = Vulnerability Scores Multipliers from **Table2**

Table 1: Base Scores and Vulnerability Scores for RC buildings

Number of Stories	Base Scores (BS)			Vulnerability Scores (VS)					
	Zone I	Zone II	Zone III	Soft Story	Heavy Overhang	Apparen t Quality	Short Column	Poundin g	Topog. Effects
1 or 2	100	130	150	0	-5	-5	-5	0	0
3	90	120	140	-15	-10	-10	-5	-2	0
4	75	100	120	-20	-10	-10	-5	-3	-2
5	65	85	100	-25	-15	-15	-5	-3	-2
6 or 7	60	80	90	-30	-15	-15	-5	-3	-2

Table 2: Vulnerability Scores Multipliers (VSM)

Soft Story	Does not exist = 0; Exists = 1
Heavy overhangs	Does not exist = 0; Exists = 1
Apparent quality	Good = 0; Moderate = 1; Poor = 2
Short column	Does not exist = 0; Exists = 1

Pounding effect	Does not exist = 0; Exists = 1
Topographic effects	Does not exist = 0; Exists = 1

Preliminary Assessment procedure:

In preliminary Assessment is required to identify highly vulnerable building to damage. Yucemen et al. 2004, Ozcebe et al. (2003) and Yakut et al. (2003) employed the discriminant analysis technique to develop a preliminary evaluation methodology for assessing seismic vulnerability of existing low-rise to medium-rise RC buildings in Turkey. The procedure is applicable to RC frames and frame-wall structures, having up to seven stories. Six discriminant parameters that is Number of stories (n), Minimum normalized lateral stiffness index (mnlstfi), Minimum normalized lateral strength index (mnlsi), Normalized redundancy score (nrs), Soft story index (ssi), Overhang ratio (or).
The damage index or the damage score corresponding to the life safety performance classification (DI_{LS}) and immediate occupancy performance classification (IOPC) where DI_{IO} is the damage score corresponding to IOPC shall be computed from the discriminant function given in **Equation 2 and 3**.

$$DI_{LS}=0.620-0.246mnlstfi-0.182mnlsi-0.699nrs+3.269ssi+2.728or-4.905 \quad (2)$$

$$DI_{IO}=0.808n-0.334mnlstfi-0.107mnlsi-0.687nrs+0.508ssi+3.884or-2.868 \quad (3)$$

In the proposed classification methodology, buildings are evaluated according to both performance levels. The steps to be followed are listed below.

(1) Calculate DI_{LS} and DI_{IO} scores by using **Equation 2 and Equation 3**, respectively.

(2) Determine the cutoff values for each performance classification by using **Equation 4** and **Equation 5**. The LS_{CVR} and IO_{CVR} values in **Equation.4** and **Equation 5** are correspond to **Table 3** based on the number of stories above the ground level. The CMC values are adjustment factors, shall be taken from **Table 4** based on the building location relative to the fault and the soil type at the site.

$$CV_{LS} = LS_{CVR} + |LS_{CVR}| \times (CMC - 1) \quad (4)$$

$$CV_{IO} = IO_{CVR} + |IO_{CVR}| \times (CMC - 1) \quad (5)$$

(3) By comparing the CV values with associated DI value calculate performance grouping of the building for life safety performance classification (LSPC) and immediate occupancy performance classification (IOPC) as follows and classification is shows in **Table 5**:

If $DI_{LS} > CV_{LS}$; take $PG_{LS}=1$
If $DI_{LS} < CV_{LS}$; take $PG_{LS}=0$
If $DI_{IO} > CV_{IO}$; take $PG_{IO}=1$
If $DI_{IO} < CV_{IO}$; take $PG_{IO}=0$

Table 3: Variation of LS_{CVR} and IO_{CVR} Values with Number of Stories

n	LS_{CVR}	IO_{CVR}
3 or less	0.383	-0.425
4	0.430	-0.609
5	0.495	-0.001
6	1.265	0.889
7	1.791	1.551

Table 4: Variation of CMC values with soil types and distance to fault

Soil Type	Shear Wave Velocity (m/s)	Distance to Fault (km)				
		0-4	5-8	9-15	16-25	>26
B	>760	0.778	0.824	0.928	1.128	1.538
C	360-760	0.864	1.000	1.240	1.642	2.414
D	180-360	0.970	1.180	1.530	2.099	3.177
E	<180	1.082	1.360	1.810	534	3.900

Table 5: Comparison of LSPC and IOPC with Classification of Structure

LSPC	IOPC	Classification
PG_{LS}=0	PG_{IO}=0	Low Risk Group (LRG)
PG_{LS}=1	PG_{IO}=0	Moderate Risk Group (MRG)/ Requiring further study
PG_{LS}=0	PG_{IO}=1	Moderate Risk Group (MRG)/ Requiring further study
PG_{LS}=1	PG_{IO}=1	High Risk Group (HRG

Seismic Vulnerability Assessment Process in the Study area:

- Observation structural types and identification of RC structures with in the study area.
- Selection of vulnerability parameters for the Walkdown Survey and Conduct Walkdown Survey
- Calculation of Building Seismic Performance Score
- Selection of buildings for Preliminary assessment on the basis of performance score.
- Collection of data for preliminary assessment
- Calculation of the Damage Index and Cutoff Values for each Performance Classifications
- Comparison of damage index with cutoff values and identification of vulnerable buildings.
-

DATA ANALYSIS AND RESULT:

Study area:

The study area in Sholokbohor ward no.8 of Chittagong City Corporation Area shown in **Fig.1** is situated on the banks of Karnaphuli River and most densely populated area of the Chittagong City. The soil profile is 21 meters or more in depth and containing more than 6 meters of soft medium stiff clay but not 12 meters of soft clay which site soil characteristics type is S3 as per Bangladesh National Building Code (BNBC-1993).

Structures belongs to Ward 8 can be classified into three types depending upon construction material, geometric configuration, construction procedure. These are Katcha (mud house & bamboo house), Semipucca (brick masonry with CI sheet roofing) and Reinforced Concrete buildings. Total number of structure in this ward is 2579 where Katcha structure is 469, Semipucca is 1354 and Reinforced Concrete building is 756. This information is given in **Fig.2**. This paper includes only the RCC structures and is classified according their heights as shown in **Table 6**.

Table 6: Number of buildings according to their height

Number of Stories	1 Stories	2 Stories	3 Stories	4 Stories	5 Stories	6 Stories	7 Stories	>7 Stories	Total
Number of Building	96	132	138	172	142	66	4	6	756

Walkdown Evaluation Method:

As describe this evaluation process to the previous section, there are observed different number of stories with the different performance scores by the basis of eight numbers of parameters. The performance scores of existing RC structures for different number of stories are shown in a following

Table 7 and also the risk classification based on different performance score are shown by a graphical representation as well as in a GIS map based on different number of stories in **Fig.3**

Fig 1: Study Area

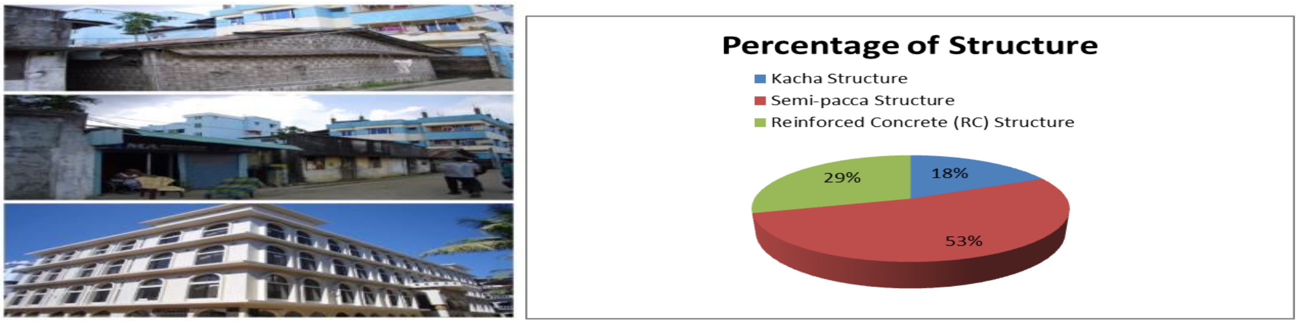


Fig.2: Types of structure & their percentage

Table 7: Summary of Walk down Evaluation of existing RC structure in Study Area

	1 stories	2 stories	3 stories	4 stories	5 stories	6 stories	7 stories	>7 stories
PS>60	96	133	138	172	19	16	2	0
40<PS≤60	0	0	0	0	112	17	0	1
0<PS≤40	0	0	0	0	9	34	2	5

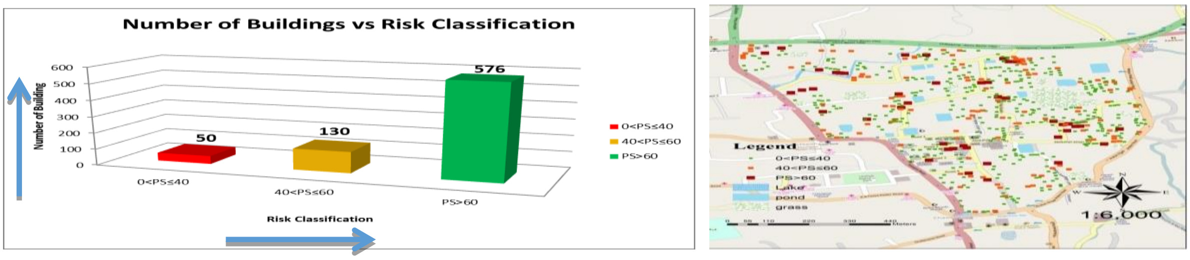
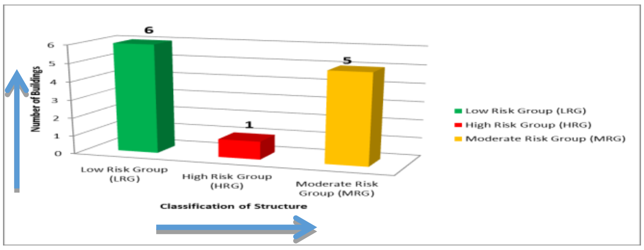


Fig.3: Risk classification based on performance score

Preliminary Assessment procedure:

In Preliminary Evaluation stage, buildings which having performance score of 40 or less found in Walkdown evaluation were given the priority. Selecting 12 buildings out of 50 in which 2 buildings were five stories and 10 buildings were six stories. After taking all the specific information about the structural system of each building including all dimensions of structural and non-structural elements, buildings are evaluated according to both the life safety performance classification and the immediate occupancy performance classification performance levels. And the classifications are done as mention to the previous section. At the end of the Preliminary Survey, the total surveyed structures are classified into three categories that is Low Risk Group (LRG), High Risk Group (HRG) and requiring further study. From preliminary assessment of 12 buildings we found that 6 buildings are Low Risk Group (LRG), 1 High Risk Group (HRG) and 5 require further study; which is shown by a graphical representation as well as in a GIS map in **Fig.4**.



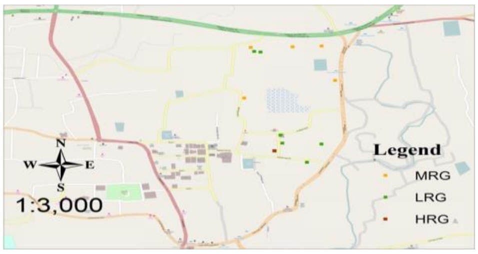


Fig.4: Preliminary assessment result

DISCUSSIONS

Here the total evaluation procedure is classified into two categories. Total 2579 structures were surveyed in Walk down Evaluation stage; among them 18% kacha, 53% semi pacca and 29% RC structure. Then eight parameters were used to classify RC structures based on Performance Score (PS) and categorized them into three i.e $0<PS\leq40$, $40<PS\leq60$ and $PS>60$. Total 576 RC structures were in $PS>60$, 130 were in $40<PS\leq60$ and 50 were equal or less than 40 in Performance Score (PS). 12 RC structures which were in $0<PS\leq40$ in performance score; were mostly 5 storied or above selected for second stage and six discriminant parameters were collected with their structural and non-structural elements dimensions. There were 6 "Low Risk Group (LRG)", 1 "High Risk Group (HRG)" and 5 "Requiring for Further Study / Moderate Risk Group (MRG)" structures by comparing two discriminant function in both "life safety performance classification" and "immediate occupancy performance classification".

CONCLUSION

For the buildings belong to Ward 8 of Chittagong City Corporation, both the "Walkdown Evaluation" and "Preliminary Assessment" procedures gave consistent results. It can be ceased that the Walkdown Evaluation procedure can be enhanced by the Preliminary Assessment procedure and proper retrofitting option should be needed for the lower performance score based buildings in the long run as well as the entire building stock of Chittagong City Corporation area should be screened by using the Preliminary Assessment technique.

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ID: SEE 043

SUITABILITY OF A RECENTLY PRACTICED RETROFITTING TECHNIQUE

Md. AyatullahKhomeni[1*], Abu Bakar Md. Siddique[1], Tanvir Manzur[1] & Eqramul Hoque[1]

[1] *Dept. of Civil Engineering, BUET, Dhaka-1000, Bangladesh.*
**Corresponding Author, <ayatullah.buet08@gmail.com>*

ABSTRACT

Columns and foundations are the two most vulnerable structural elements of a building and need immediate strengthening if not properly designed. Sometimes, strengthening of columns is also required for further vertical extension or change of usage pattern of a building for which it was not originally designed. Usually, columns should be retrofitted up to foundation level and foundation should also be strengthened to support the additional loads. However, it is observed in some instances that columns are strengthened to certain depth below EGL since excavation up to original footing is difficult, time consuming and costly. In such cases, a new footing is constructed at that level. The primary hypothesis behind this type of construction practice is that soil below newly constructed footing is compacted enough to distribute the additional load. Such retrofitting technique is relatively less complicated to execute in field and could be recommended if an appropriate guideline is available. In order to establish a guideline, it is important to understand the behavior of such retrofitted column-footing system and identify factors that predominantly influence the behavior. In this study, an effort has been made to investigate the behavior of such retrofitted columns and footings through extensive finite element analysis. From the study, it is found that the dimension of newly constructed footing under retrofitted column and sub grade modulus of the soil mainly govern the fraction of the total load transferred to the new footing. It is also evident that this transferred portion augments with the increase in new footing area and sub grade modulus under the new footing.

Keywords: Column, Footing, Retrofitting, Strengthening, Sub-grade Modulus of soil.

INTRODUCTION

Retrofitting is the process of modifying something after it has been manufactured. It is a method of increasing resistant capacity of a structure by various techniques. It is also a technical intervention in structural system of a building that improves the resistance to earthquake by optimizing the strength, ductility and earthquake loads (ACI, 2008). It is beyond doubt that many buildings in our country have been designed and constructed without following the appropriate codes. After the tragic Rana Plaza incident, all RMG factory buildings are now under extensive scrutiny and structural assessment of these factories are in full swing now. It is obvious that a good number of buildings will be rated as inadequately designed and retrofitting of these buildings will eventually become necessary. Undoubtedly, columns and foundations are the two most important structural elements of any building and need urgent strengthening if not properly designed. During such retrofitting operation, columns need to be strengthened up to foundation level which often becomes extremely difficult to execute under prevailing field condition. Therefore, in many instances it has been observed that columns are strengthened to certain depth below EGL and a new footing is constructed at that level. Dimension of columns below the new footing remains same. In such construction practice it is assumed that soil below newly constructed footing is compacted enough to distribute the additional load beyond the capacity of original column. This retrofitting technique is easier to execute in field and should be recommended if found appropriate through proper investigation. It is, therefore, imperative to investigate the behavior of such retrofitted column-footingsystem and identify factors that predominantly influence the behavior. A research initiative has undertaken to study the behavior of such retrofitting scheme through extensive finite element analysis. The main objective of the study is to evaluate if a significant portion of the load is being transferred to the newly constructed footing or not under different soil conditions both below the new footing and originally constructed footing. Footing size and column dimensions are also vvaried in this parametric study. Finite element package ABAQUS 6.9 is utilized (Reddy J. N., 1993; Hibbit, 2009). Initial pertinent outcomes of this research have been presented in this article. This study is based on results obtained from analysis of regular shaped square type foundation. Columns are subjected to both pure axial loading and moment in the analysis.

MATERIALS CHARATERISTICS AND MODEL PARAMETERS

In this analysis, concrete and steel are used as elastic material. Young's modulus of concrete is considered as 5800 ksi and Poison's ratio is taken as 0.19. Element type C3D8R (cubic 8 node linear brick element) is used for meshing of concrete column. Deformed bar of φ-12mm is used as main bar and φ- 10mm bar is used as tie bar having Young's modulus of 29000 ksi and Poisson's ratio of 0.3. Element type B31 is used for meshing reinforcement. B31 refers 2-node linear beam. Table 1 shows different soil subgrade modulus that is used under main foundation and retrofitted foundation. Table 2 shows dimension of original column, retrofitted column, original footing and new footing.

Table 1: Subgrade modulus of soil

Soil Subgrade modulus under original foundation, K1 (lb/in^3)	Soil subgrade modulus under retrofitted foundation, K2 (lb/in^3)	
118.5	K2=K1	118.5
	K2=1.5*K1	177.75
	K2=2*K1	237

Table 2: Dimension of column and foundation used in the study

Original column size (inch)	Retrofitted foundation column size (inch)	Original foundation size (inch)	Retrofitted foundation size (inch)
10 x 10	12 x 12	120 x 120	60 x 60
			90 x 90
			120 x 120
10 x 10	16 x 16	120 x 120	60 x 60
			90 x 90
			120 x 120
10 x 10	20 x 20	120 x 120	60 x 60
			90 x 90
			120 x 120

RESULTS AND DISCUSSION

As already mentioned effect of newly built foundation under retrofitted columns has been investigated considering three different footing and retrofitted column sizes (Table 2). Three different soil subgrade modulus's (with respect to soil subgrade modulus under original foundation) under new foundation have also been considered in the analysis. Table 3 shows percentage of load supported by new foundation for different sizes of new foundation and different soil subgrade modulus below them. Results of able 3 are for 20" x 20" retrofitted column. It is found that for 60" x 60" dimension of new foundation, around 21.9% of axial load and 50.2% of applied moment transfer to the new foundation considering both soil have similar sub grade modulus (118.5 lb/in^3). It is also observed that amount of load transferred to new foundation increases with the increase in size of new foundation with diminishing trend (Fig. 1). For new foundation size of 90" x 90", about 39% of axial load and 83.35 of applied moment is taken by the new foundation considering the similar soil sub grade modulus as before. Percentage of load transferred to new foundation also significantly depends on soil subgrade modulus under new foundation. For example, it is obtained that around 52.7% of axial load and 93.6% of applied moment are taken by the new foundation for equal soil subgrade modulus (118.5 ib/in^3) when a new foundation of 120" x 120" in dimension is constructed. When soil subgrade modulus under new foundation is increased by 1.5 times as compared to soil below original footing, the percentage of axial load and moment transferred to new foundation is found as 62.1% and 94.5%, respectively. However, when soil subgrade modulus under new foundation is increased by twice the value under existing foundation, the increment in percentage of axial load and moment transferred to new foundation is relatively less (found as 68.2% and 96.4%, respectively) as compared to previous case of 1.5 times increment of soil subgrade modulus. Fig.2 and Fig. 3 show such trend in amount of load (both axial load and moment) transferred to new foundation for different soil subgrade modulus under new and existing foundation. It is evident that the amount of moment taken by the new foundation remains almost similar for larger size of new foundation.

Table 3: Percentage of load supported by new foundation with varying foundation size and modulus of soil subgrade for 20" x 20" retrofitted column.

Size of new foundation (inch)	Soil subgrade modulus (lb/in^3)		% of load supported by new foundation	
	Under existing foundation	Under new foundation	Axial Load (%)	Moment (%)
60 x 60	118.5	118.5	21.9	50.2
		177.75	29.7	60.1
		237	36	66.7
90 x 90	118.5	118.5	39	83.3
		177.75	48.8	88
		237	55.8	90.7
120 x 120	118.5	118.5	52.7	93.6
		177.75	62.1	94.5
		237	68.2	96.4

Table 4: Variation of retro foundation capacity w.r.t distance between existing foundation and new foundation for 16" x 16" retrofitted column.

Distance between existing foundation and new foundation (ft)	Soil subgrade modulus (lb/in^3)		% of load coming to new foundation	
	Under existing foundation (K1)	Under new foundation (K2)	Axial load (%)	Moment (%)
1.33	118.5	177.75	47.5	80.6
3			48.6	88.1

4.67			49.7	91

Table 4 represents the percentage of load coming to new foundation due to variation in distance between two foundations for retrofitted column size of a 16" x 16". It is observed that the amount of load (both axial load and moment) transferred to new foundation increases with the increase in distance between new and original foundation. For a distance of 1.33ft between two foundations, about 47.5% of axial load and 80.6% of moment are taken by the new foundation. When the distance between two foundations is increased to 4.67ft then the percentage of axial load and moment coming to newly constructed foundation are augmented to 48.6% and 88.1%, respectively. Fig. 4 shows graphically the variation of percentage of axial load and moment transferred to new foundation for three different distances between original foundation and new foundation as provided in Table 4.

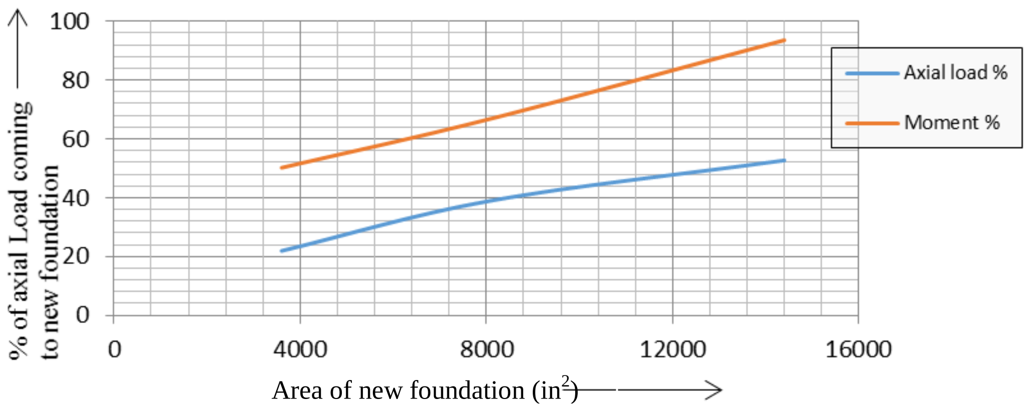

Fig. 1: Effect of new foundation size on percentage of load (axial load and moment) transferred to them for similar soil subgrade modulus under new and existing foundation

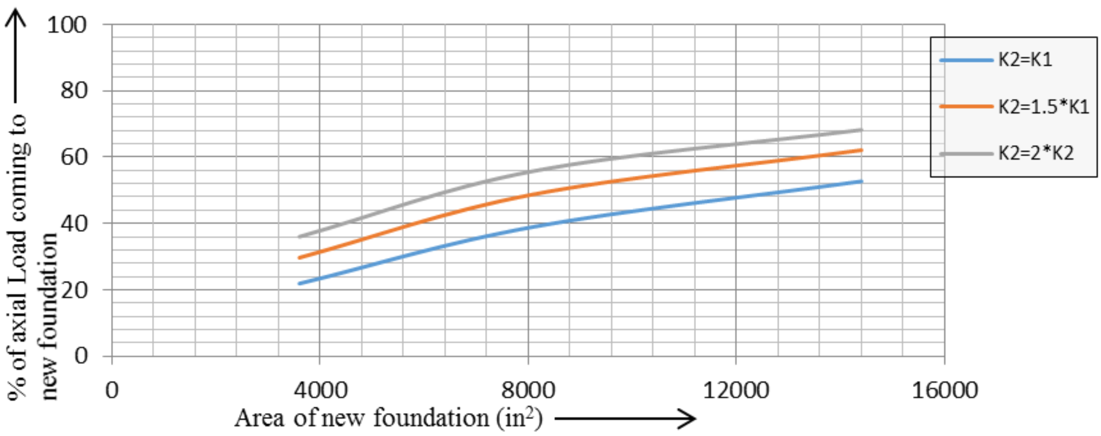

Fig. 2: Area of new foundation vs % of axial load coming to new foundation for different soil subgrade modulus under new and existing foundation

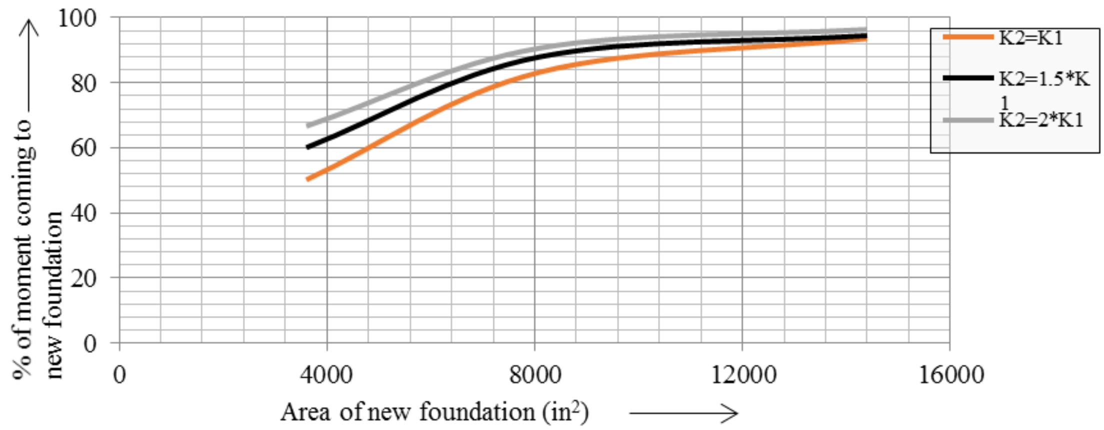

Fig. 3: % of moment coming to new foundation vs Area of new foundation for different soil subgrade modulus under new and existing foundation

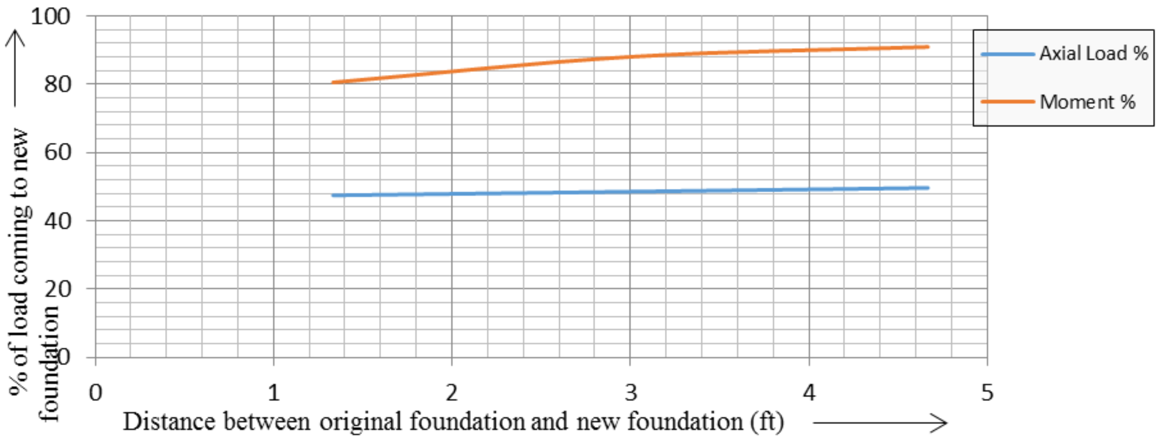

Fig. 4: % of load coming to new foundation vs distance between original foundation and new foundation.

CONCLUSION

From the limited study performed in the study, the following conclusions can be tentatively drawn;

- The increase in size of new foundation results in increase in the percentage of axial load and moment coming to new foundation with a diminishing trend.
- The soil subgrade modulus under newly constructed foundation has significant effect on the amount of load (both axial load and moment) transferred to the new foundation. Therefore, it is extremely important to compact the soil under new foundation.
- Size of retrofitted column has insignificant effect on amount of load transferred to new foundation.
- The distance between original foundation and newly built foundation also has some effect on amount of load (particularly in case of moment) transferred to new foundation.
- Finally, it could be concluded that the two most important parameters that should be considered during implementation of such retrofitting technique are size of the new foundation and subgrade modulus of soil under them.

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FLEXURAL RETROFITTING OF REINFORCED CONCRETE BEAM USING FERROCEMENT

M.R. Alam[1], M.A. Hossain[1*], R. Dey[1] & M.M. Uddin[1]

[1] *Dept. of Civil Engineering, CUET, Chittagong-4349, Bangladesh.*

**Corresponding Author*

ABSTRACT

Strengthening of deteriorated existing concrete structures is necessary to extend their life span. Glass fiber reinforced polymer (GFRP), carbon fiber reinforced polymer (CFRP), ferrocement, etc. are the popular materials being used for retrofitting of the concrete structures. In the present study, an attempt has been made to determine the performance of the RC beams retrofitted with ferrocement materials. A total number of sixteen flexural deficient reinforced concrete beams were cast for this purpose. Among all the sixteen beams, four beams were treated as the control beams and the other twelve beams were treated as ferrocement retrofitted beams (FRB). These twelve beams were separated into three types (each type contains 4 beams), such as FRB1(wire mesh-one layer, 12mm thick cement mortar), FRB2 (wire mesh-two layers, 16mm thick cement mortar) and FRB3 (wire mesh-three layers, 20mm thick cement mortar). The first cracking load, ultimate load, deflections and failure modes of each of the beams were determined experimentally. The effect of number of layers of wire meshes on the first cracking load, ultimate load carrying capacity, deflection behavior and failure modes of the retrofitted beams were investigated. The first cracking loads for FRB1, FRB2, and FRB3 are found to be increased by 15.38%, 53.84% and 75.64%, respectively and the ultimate loads for FRB1, FRB2, FRB3 are also found to be increased by 12%, 43.2% and 60%, respectively. It was found that the deflections and crack widths of the ferrocement retrofitted beams were found smaller than those obtained from the control beams.

Keywords: Retrofitting, Strengthening, First-cracking load, Flexure deficient, Deflection, Ultimate load.

1. INTRODUCTION:

Deterioration of concrete and corrosion of embedded reinforcement within the structure might make the R.C.C members structurally weak. Many natural hazards like wind, earthquake, etc., also produce cracks of structural elements, even collapse of the entire structure. Restrengthening and retrofitting of structures is the process by which load carrying capacity of existing structure is enhanced against any adverse forces. This can be accomplished through the repairing/improving of existing structural elements. Polymer injection followed by RC jacketing, steel plate bonding, use of advanced composite materials like fibre reinforced polymer (FRP), glass fibre reinforced polymer (GFRP), ferrocement etc. are the various retrofitting techniques. The choice of a particular retrofitting depends upon the type, nature and cause of the distress to be repaired. Retrofitting using ferrocement is gaining popular in Bangladesh and other developing country due to its high strength to weight ratio and ease of construction. The flexural performance of deteriorated reinforced concrete beams repaired with ferrocement was found superior both at the service and ultimate load caring cases than those improved by conventional method [Andrew and Sharma, 1998]. Kaushik, S.K. and Dubey, A.K., 1994 found that the crack width of the rehabilitated composite beams with ferrocement jacketing reduces by 36%. An increase in the number of layers improves the cracking stiffness of the composite beams [Nassif, H.H et al, 1998, Vidivelli, B. et al, 2001, Nasif, N.H. et al 2004].

In this present study experimental program has been designed carefully and performed at the Strength of Materials Laboratory of the Chittagong University of Engineering &Technology accordingly. A total of sixteen flexural deficient rectangular reinforced concrete beams were cast and tested. Third point loading was applied on the beams to determine their first cracking load, ultimate failure load and deflections under load controlled condition. The effect of number of layers of wire mesh on the first cracking load, ultimate load, deflections behavior and failure modes of the retrofitted beams as well as controlled beams were investigated.

2. EXPERIMENTAL INVESTIGATION:

2.1 MATERIALS:

Ordinary Portland Cement (OPC) conforming Type-I of 43 Grade satisfying the requirements of ASTM C-150 [18] was used for this investigation. A mix proportion of 1:1 of two types of river sand (FM 2.4 and 1.67) to achieve a combined fineness modulus 2.1 was used for the preparation of concrete. Crushed stone in angular shape was used as coarse aggregate. 20 mm down graded coarse aggregate with a unit weight of 1650 kg/m^3 and absorption capacity of 0.51% was used in this mixture. Normal tap water conforming drinking quality was used for the preparation and curing of concrete specimens. The concrete mix was designed according to ACI method 211 for slump value of 25.4 mm and 28 days cylinder compressive strength of 22 MPa (3200 psi). Design concrete mix of 1: 1.5: 3 and w/c of 0.5 was found to be appropriate for this requirement. Three cylinder specimens were cast and tested (at the age of 28 days) to determine compressive strength of concrete mixture. The average compressive strength of concrete was 22 N/mm². Steel bars of diameter 12 mm and 10 mm having 60 ksi yield strength were used as longitudinal reinforcement and 8 mm diameter bars were used as shear reinfocement. The wire mesh used in the ferrocement jacketing was 20 BWG (British Standard Wire Gauge) woven GI (Galvanized Iron) of 12 mm square openings.

2.2 Preparation of testing specimen:

The experimental work comprises casting of 4 sets of reinforced concrete (RC) beams having concrete of grade M22, cross sectional dimensions of 150mm x 225mm x 1000mm length. Four different configurations such as FRB1, FRB2, FRB3 & control beams were prepared for retrofitting of the beams using ferrocement. Total 16 no. of RC beams are cast and cured for 28 days by wet jute bags. First set of (4 no.) RC beams designated as FRB1 was retrofitted using single layer wire mesh and 12 mm thick cement mortar. Second set of (4 no.) RC beams designated as FRB2 was retrofitted using 2-layer of wire mesh and 16 mm thick cement mortar. Third set of (4 no.) RC beams designated as FRB3 was retrofitted using 3-layer wire mesh and 20 mm thick cement mortar. Fourth set of (4 no.) RC beams (no retrofitting) treated as control beams.

Formwork made of wood material for beam size 150mm x 225mm and 1000mm long is thoroughly cleaned and properly sealed to avoid leakage of concrete ingredients. Reinforcement cage is then placed carefully inside the formwork keeping 20mm clear cover for the top and bottom bars as shown in Fig. 2.2.1. Fig. 2.2.1: Formwork for beam

Concrete was placed gently in the formwork and compacted using temping rod of specific dimension- 610 mm long bullet nosed metal rod of 16 mm in diameter (Fig. 2.2.2).

Fig. 2.2.3: Curing of beams

Fig.2.2.2: Hand compaction of concrete

Curing of all the beams was done by wet jute bags for 28 days using clean water for maintaining a satisfactory moisture content and favorable temperature shown in Fig. 2.2.3 above.

2.3 Reinforcing details of beams:

All the beams mentioned in the previous section are identical. Reinforcement details of beam, their material and sectional properties shown in Fig 2.3.1 & 2.3.2 respectively.

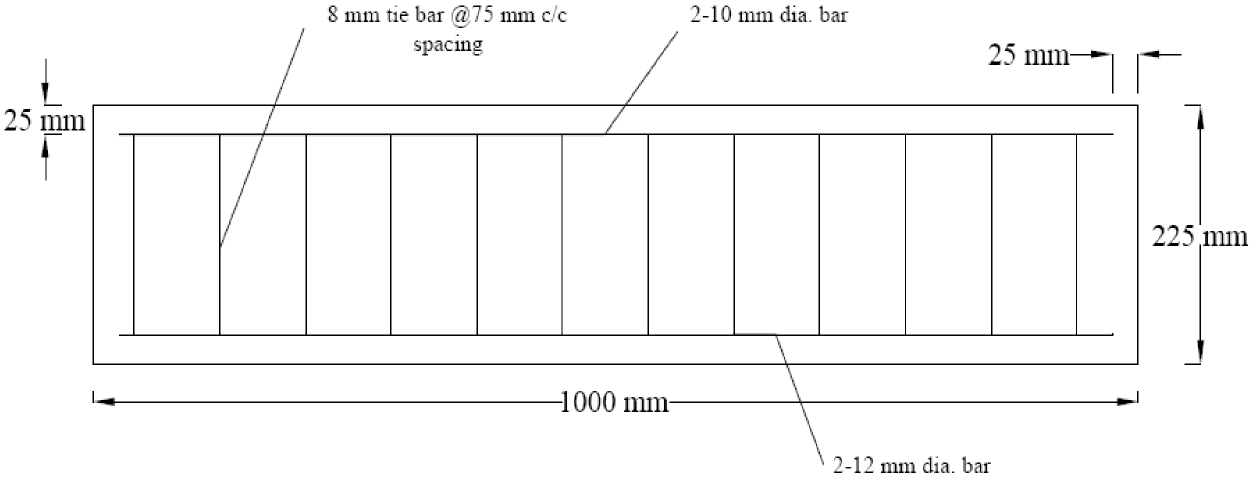


Fig. 2.3.1: Reinfocement detail of beams

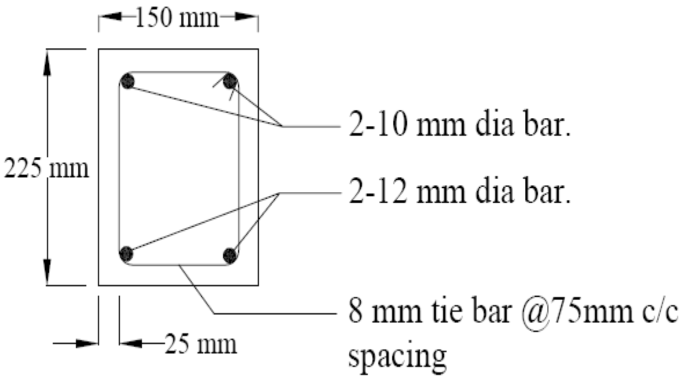
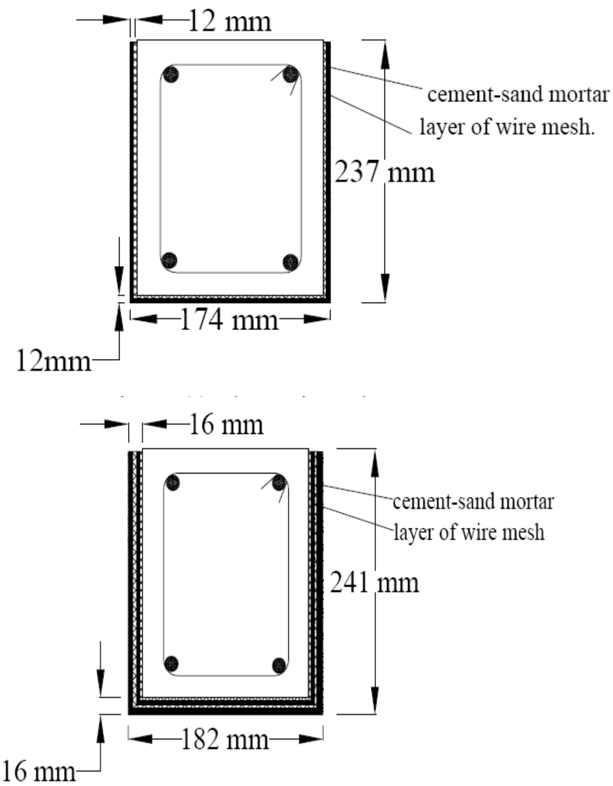


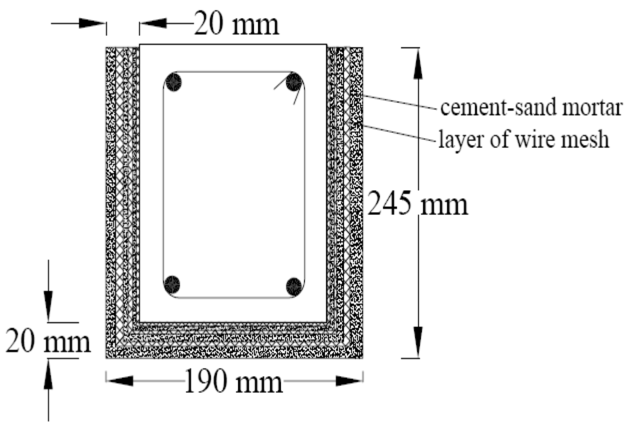
Fig. 2.3.2: Section of beams

2.4 Retrofitting of Beams:

After 28 days of curing using wet jute bags RC beams were removed and the beams were dried for a few hours to obtain surface dry condition. In order to simulate damage, the beams were preloaded before applying ferrocement jacketing. Preloading was done with the same setup described in the section 3 and was loaded until appeaing of first crack. Then, different types of ferrocement jacketing were applied to different RC beams. Fig. 2.4.1 shows the cross sectional details of the ferrocement retrofitted beams. The beams were retrofitted on three sides of RC beams using 1,2, 3 layers of wire mesh and thickness of cement mortar layer was 12 mm,16 mm and 20 mm consecutively. At first original beam surfaces are cleaned and wet by water. Thereafter, cement mortar having thickness half of the ferrocement layer is put on the beam surface. Ferrocement wire mesh is fixed on this wet surface as quick as possible. After fixing wire mesh, cement mortar is put again on the wire mesh to make required thickness of ferrocement wire. A gap of 3 mm was kept between RC beam specimen and ferrocement jacket at both the top and bottom of the specimen to avoid direct compression on the ferrocement jacket. Afterward, all the ferrocement jacketed beams were subjected to curing using wet jute bags for 28 days.



FRB1 FRB2



FRB3

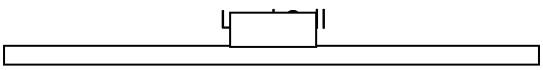
Fig. 2.4.1: Cross sectional details of ferrocement retrofitted Beams

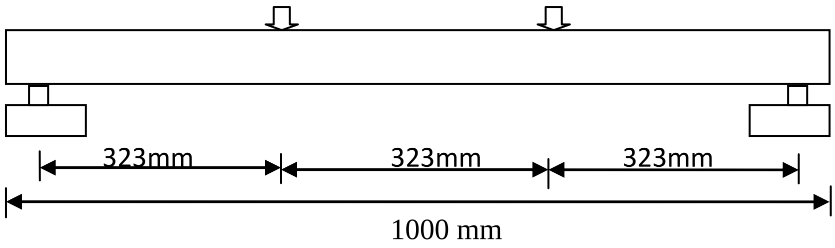
a) Retroffiting of RC beam with wire-mesh in 3 sides   b) Retroffited beams (Typical cross-section)

Fig. 2.4.2: Retrofitting of beams using ferrocement jacketing

TESTING PROCEDURES AND INSTRUMENTATION:

The control beams and the retrofitted beams were tested for flexural strength. After curing, all the specimens were kept in room temperature for few hour to attain standard surface dry condition. The tests were performed on a Universal Testing Machine (UTM) with a capacity of 1000 KN. The testing procedures for all the specimens are same. The beams were subjected to third point loading to determine their load carrying capacity. This load case was chosen because it gives constant maximum moment and zero shear in the sections between the loads, constant maximum shear force between support and load. The moment was linearly varying between supports and load. The span between the supports was 970 mm and the load was applied at points dividing the length into three equal parts as shown in Fig. 3.1(a). Steel plates were used under the loads to distribute the load over the width of the beam. Deflection measuring gauge was used to measure the deflection at one third span, mid span and two third points as shown in Fig. 3.1(c). Deflections at one third, two third and mid span corresponding to load at 10 KN intervals were recorded during the test using deflection measuring gauges. Figure 3.1 shows the experimental setup of a beam.





a) Third point loading of beam

b) UTM (Capacity-1000KN) c) Deflection measurement by deflection measuring gauge

Fig. 3.1 : Experimental setup for testing of beams

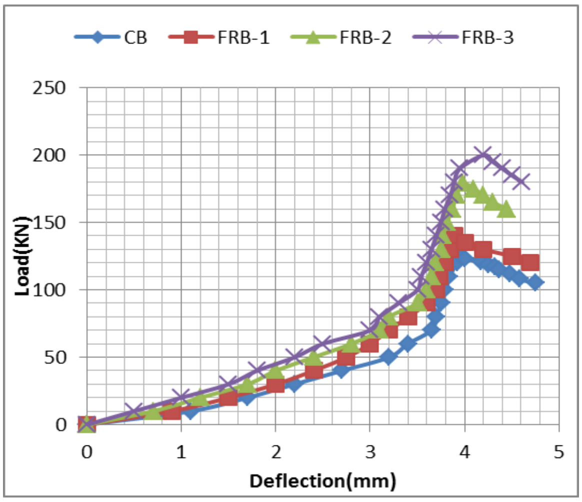
2. RESULTS AND DISCUSSION:

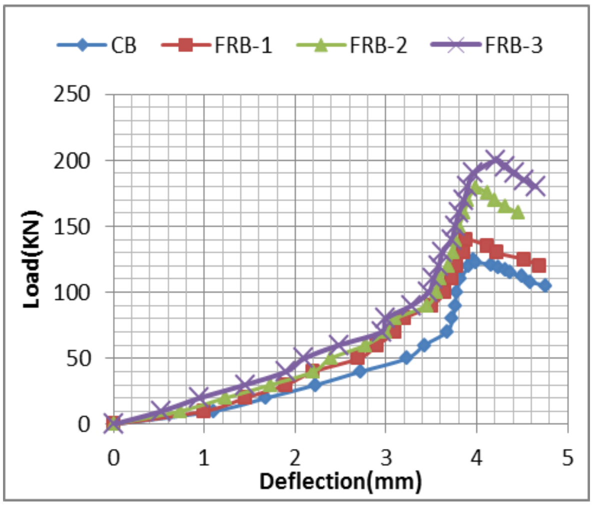
Four sets of beams were tested for determining first cracking load, ultimate load carrying capacity and all the beams were tested until failure. The results obtained from the experimental investigation for first cracking load both jacketed and non-jacketed beams are given in Table 4.1. The deflections of each type of retroffited beams for third point loading are analyzed and compared with the control beam. Results are shown in table 4.2. The load deflection behavior is also compared between beams jacketed with different layer of ferrocement. Comparision shown in Fig. 4.1. It is seen that the control beam has less load carrying capacity and high deflection values in compared to that of externally retrofitted beam.

Table 4.1: Load- Deflection compasion

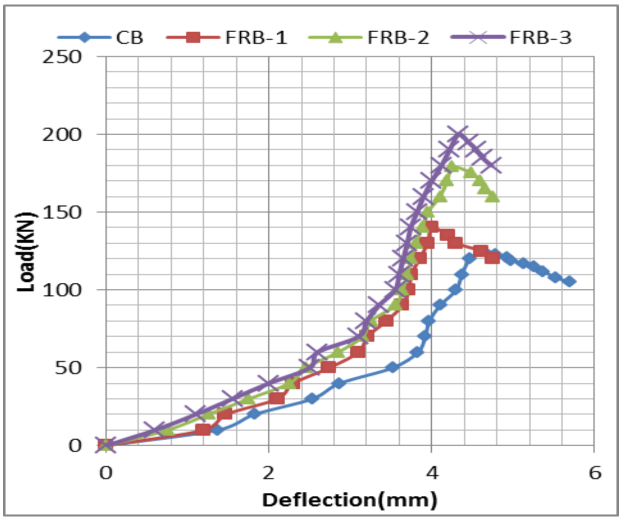
Load (KN)	Deflection of Control Beam (mm)			Deflection of FRB1 (mm)			Deflection of FRB2 (mm)			Deflection of FRB3 (mm)		
	1/3	2/3	mid-	1/3	2/3	mid-	1/3	2/3	mid-	1/3	2/3	mid-

	span	span	span	span	span	span	span	span	span	span	span	span
0	0	0	0	0	0	0	0	0	0	0	0	0
10	1.1	1.1	1.37	0.9	1	1.2	0.7	0.73	0.75	0.5	0.53	0.6
20	1.7	1.7	1.82	1.5	1.45	1.47	1.2	1.23	1.25	1	0.95	1.1
30	2.2	2.2	2.53	2	1.9	2.1	1.7	1.73	1.75	1.5	1.45	1.55
40	2.7	2.7	2.87	2.4	2.2	2.3	2	2.2	2.25	1.8	1.9	2
50	3.2	3.2	3.52	2.75	2.7	2.74	2.4	2.39	2.45	2.2	2.1	2.5
60	3.4	3.4	3.83	3	2.9	3.1	2.8	2.77	2.85	2.5	2.48	2.6
70	3.65	3.65	3.92	3.2	3.1	3.21	3.1	3	3.15	3	2.95	3.1
80	3.7	3.7	3.97	3.4	3.2	3.44	3.2	3.1	3.25	3.1	3	3.2
90	3.75	3.75	4.1	3.6	3.5	3.64	3.5	3.45	3.55	3.3	3.28	3.35
100	3.79	3.79	4.3	3.7	3.65	3.72	3.6	3.55	3.64	3.5	3.47	3.55
110	3.84	3.84	4.37	3.73	3.72	3.75	3.65	3.6	3.7	3.54	3.52	3.6
120	3.92	3.92	-	3.79	3.78	3.85	3.69	3.68	3.75	3.6	3.58	3.65
130	-	-	-	3.85	3.86	3.95	3.75	3.74	3.8	3.65	3.62	3.7
140	-	-	-	3.89	3.88	4	3.77	3.76	3.88	3.7	3.72	3.74
150	-	-	-	-	-	-	3.81	3.8	3.95	3.75	3.76	3.82
160	-	-	-	-	-	-	3.87	3.86	4.1	3.79	3.8	3.9
170	-	-	-	-	-	-	3.91	3.9	4.19	3.85	3.86	4
180	-	-	-	-	-	-	3.98	-	-	3.89	3.9	4.12
190		-	-		-	-		-	-		3.96	4.22
200		-	-		-	-		-	-		4.21	4.33





a) 1/3rd Span b) 2/3rd Span



c) Mid-span

Fig. 4.1: Load vs Deflection diagramFailure patern of control beam and retrofitted beams are shown in the Fig. 4.2 and 4.3. In case of control beam, a few number of wide cracks were observed, however in case of ferrocement retrofitted beams a more number of hair line cracks developed during loading between two loading point. The beams were finally failed due to widening of one of these cracks. Ultimate failure of retrofitted beam is shown in Fig. 4.4.

Fig. 4.2: Failure of control beams

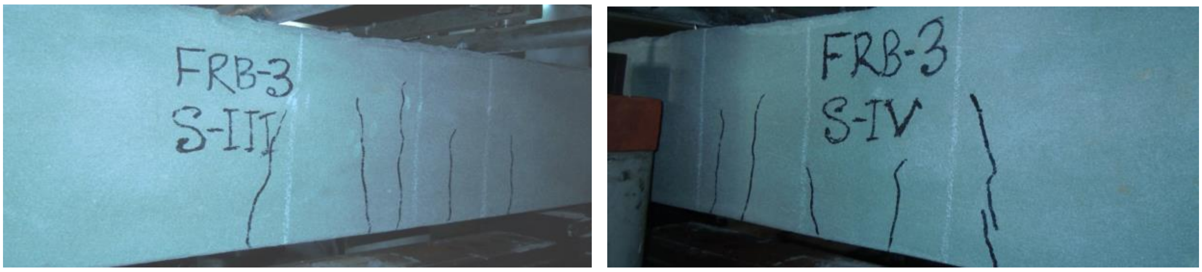


Fig. 4.3: Cracks on retrofitted beams

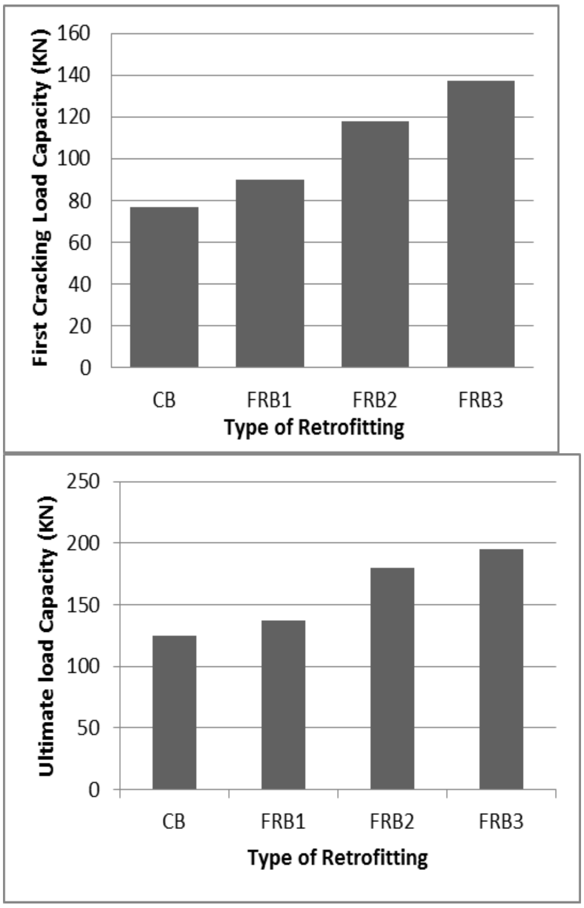
Fig. 4.4: Ultimate failure of retrofitted beams

After retrofitting the beams became strong enough in flexure but showed insufficient shear strength, so retroffited beams were failed in shear zone between the support.

Table 4.2: First cracking load and ultimate load

Sl. No.	Beam Designation		First Cracking Load (KN)	% Increase in First Crack Load	Ultimate Load (KN)	% Increase in Ultimate Strength
1.	CB	S-I	77	-	126	-
		S-II	81	-	125	-
		S-III	78	-	124	-
		S-IV	76	-	125	-
2.	FRB1	S-I	90	15.38	138	10.4
		S-II	88	12.82	136	8.80
		S-III	90	15.38	138	10.4
		S-IV	89	14.10	140	12
3.	FRB2	S-I	120	53.85	177	41.6
		S-II	118	51.28	178	42.40
		S-III	118	51.28	178	42.40
		S-IV	116	48.72	179	43.20

4.	FRB3	S-I	138	75.64	200	60
		S-II	136	74.36	198	58.4
		S-III	137	75.64	195	56
		S-IV	135	73.08	199	59.2



a) First cracking load b) Ultimate failure load

Fig. 4.5: Variation of first cracking load and ultimate failure load

When all the retrofitted beams were considered it is observed that the beams retrofitted with 3-layers of wire mesh and 20 mm thickness cement mortar shows a better load deflection behavior in compare to the other retrofitted beams. From table 4.1 it is seen that the deflection of FRB1, FRB2 and FRB3 beams under first cracking load decreased by 12%, 13.9% and 17.2% respectively. From table 4.2 it is seen that first cracking load for FRB3 beam is about 90% greater then the control beam. Ultimate load carrying capacity of FRB3 beams increased by 58.40 % than the control beams and had the highest load capacity than all other retrofitted beams [Fig. 4.5(b)].

3. CONCLUSION:

In this experimental investigation, the effect of number of layers of wire mesh in ferrocement jacketing on the performance of deteriorated RC beams are studied. The following conclusions were drawn based on the test results and the subsequent findings of this study:

- The deflections of the retrofitted beams were lesser than that of the control beams. Therefore stiffness of the ferrocement retrofitted beams is increased compared to that of the control beams.

- The first crack load for the beams retrofitted with one, two and three layers of wire mesh increased by 14.42 %, 51.28 % and 74.68 % respectively.
- The ultimate load carrying capacity for the beams retrofitted with one, two and three layers of wire mesh increased by 10.4 %, 42.4 % and 58.4 % respectively.
- In the case of ferrocement retrofitted beam with two and three layers of wire mesh showed lesser deflection and higher percent increase in the ultimate load carrying capacity. The difference in their performance is not significant enough but high cost involve in FRB3 retrofitting technique will offset the better performance of it.
- The failure of control beam and retrofitted beams were characterized by the formation of flexural cracks in the tension zone.
- The crack width for the retrofitted beams is decreased compared to the control beams.

Hence, improved ferrocement jacketing with 2-layers of wire mesh and 16 mm mortar thickness could be used as an effective retrofitting technique for deteriorated or initially cracked reinforced concrete structures.

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RELATIVE STIFFNESS OF SLAB-COLUMN JOINT IN FLAT-PLATE

S. Ahmed[1] & M. R. Amin[1*]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.,*
**Corresponding Author, <ruhulbuet@gmail.com>*

ABSTRACT

It is a common practice that in the beam-column frame structure, soft beam-strong column is provided to ensure that the failure is initiated by the failure of beam. In the same way, in flat plate structure soft slab and stiff column should be maintained - otherwise the failure occurs due to unbalanced moment; column will fail prior to the slab and may lead to collapse of whole structure. In this paper, attempts were made to investigate the possibility of such kind of risk. The study has been carried out in three steps. In the first step, a finite element model of a interior slab-column joint subjected to unbalanced moment by applying lateral load was developed. In the second step, the determination of balanced slab thickness for different column size was made, at which slab and column failed simultaneously. Finally, the required slab thickness according to the ACI code was calculated and compared with balanced thickness. From the study, it has been found that the ACI required slab thickness is higher than the balanced slab thickness though the differences reduces with increasing column size. So, there may be a possibility of slab being stiffer than column for small column sizes.

Keywords: Flat-Plate, Stiffness, Finite element, Column, Slab.

INTRODUCTION

Different reinforced concrete floor and roof systems have been developed since the inception of RCC era. Flat-plate is a very common and competitive structural system for cast-in-place slabs in building since no beams, column capitals or drop panels are involved, which means that formworks become extremely simple. The structural concept is at a great disadvantage; because of the risk of brittle "punching" failure at the column. Significant efforts have been made in the past and are still being made to develop methods for reliable prediction of the slab-column joint. The term 'punching' is often used to describe the failure of a slab-column connection. It is associated with a particular collapse mechanism in which the column and all attached portion of slab push through the slab. The slab breaks along a sloping surface that extends from the compression side of the slab at the face of the column to the tension side at some distance from the column. The average angle of the failure surface relative to the horizontal is usually in between 20 to 45 degree. For cases of balanced load (no net moment transfer between the slab and the column), the inclined surface surrounds the column. The result is the classic punching failure surface in the shape of a truncated cone or pyramid. Cases of unbalanced load have a combined failure mode. The punched region is confined to the area near the more heavily loaded face of the column. The two adjacent side regions show extensive torsional cracking while the area near the opposite face may show little or no distress.

From the above discussion it is clear that to ensure proper safety of flat-plate structure it is required to maintain weak slab-strong column condition. But, proper guidance is missing in the leading codes. So, it is important to study the existing code provision if it results in a slab that is stronger than column. For this, it is needed to find out the balanced failure condition that is for a column size find out the thickness of slab for which slab and column fail simultaneously. From this it can be understood that if the slab thickness is more than balanced thickness, the slab will be stiffer than column, which is not desirable. Flat plate mainly deals with concrete structure so one can balance it by compressive stress. In this paper, a model of slab-column joint would be developed by using finite element software ANSYS and find out the balanced thickness for different column sizes.

The specific objectives of the present study are as follows:

■ To make a proper model of slab-column joint by using FE software.
■ To find out the balanced slab thickness for different column size.
■ To calculate the required thickness according to ACI code for different column sizes, subjected to lateral load at balanced failure condition and compare it with thickness as found from FEM.

METHODOLOGY

The term 'joint' is defined as the volume of concrete common to the slab and the column. Field and laboratory experience indicate that slab-column failures typically are attributable to failure of the slab or column outside the joint. With the exception of connections having inadequate anchorage within the joint, joint failures have not been observed. This contrasts with the behaviour of beam-column connections for which joint shear failures have been observed. The absence of joint shear failures in slab-column connections can be attributed partly to absence of large joint shear in typical cases and joint confinement afforded by the surrounding column and slab. The analysis of punching shear in flat plates and flat slabs assume that the shear force 'V' is resisted by shearing stresses uniformly distributed round the perimeter 'b_o' of the critical section, a distance d/2 from the face of the supporting column. If significant moment are to be transferred from the slab to the columns, as would (from unbalanced gravity loads on either side of a column or from horizontal loading due to wind or seismic effects), the shear stress on the critical section is no longer uniformly distributed. This situation can be modelled as shown in figure 1.

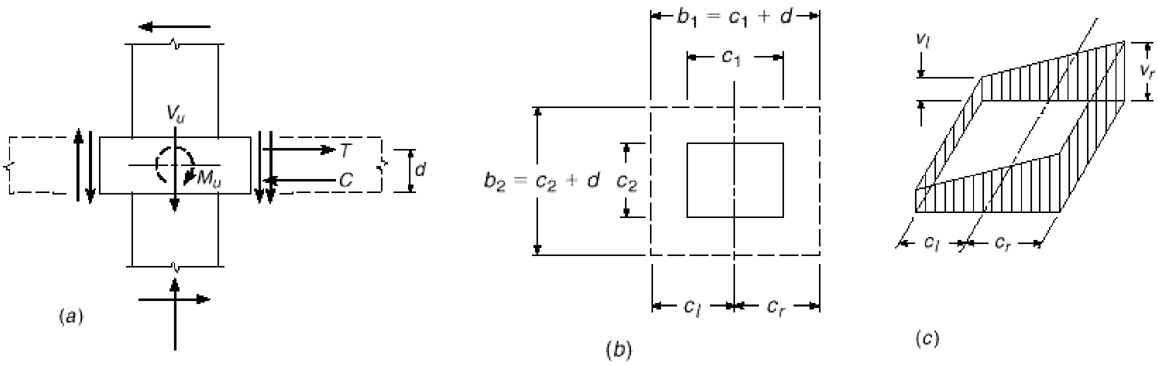

Fig. 1: Transfer of moment from slab to column: (a) forces resulting form vertical load and unbalanced moment; (b) critical section for an interior column; (c) shear stress distribution for an interior column.

MODELLING OF SLAB AND COLUMN

For the analysis purpose, a typical model of an interior slab-column joint is chosen. The column connected with slab (having thickness 't' and span length 'l_x' & 'l_y') at centre has a dimension of 'c_x' & 'c_y'). Square size slab panel with square column has been analysed so $l_x = l_y$ and $c_x = c_y$. A slab panel of 20 feet × 20 feet with column sizes ranging from 10 inch × 10 inch to 30 inch × 30 inch is chosen for parametric study. Floor-to-floor height is taken as 10 feet (= 120 inch). The column above and below the slab is kept 5 feet (= 60 inch) high considering that the inflection point is at mid-height of column.

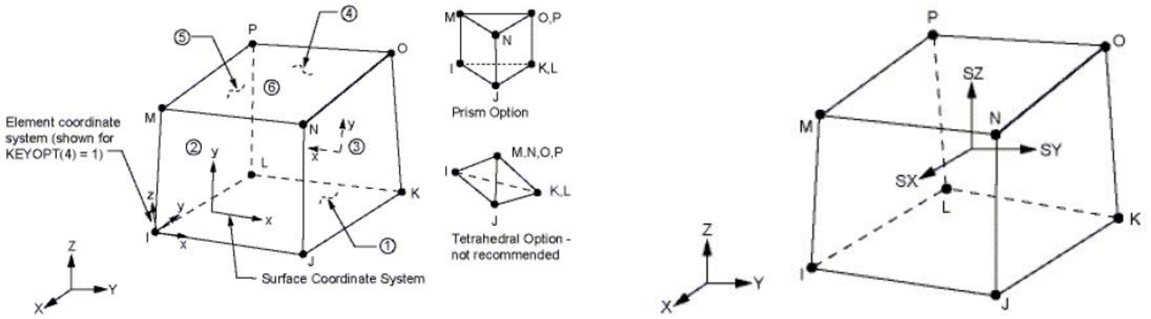

Fig. 2: SOLID 45 3-D structural solid and stress output

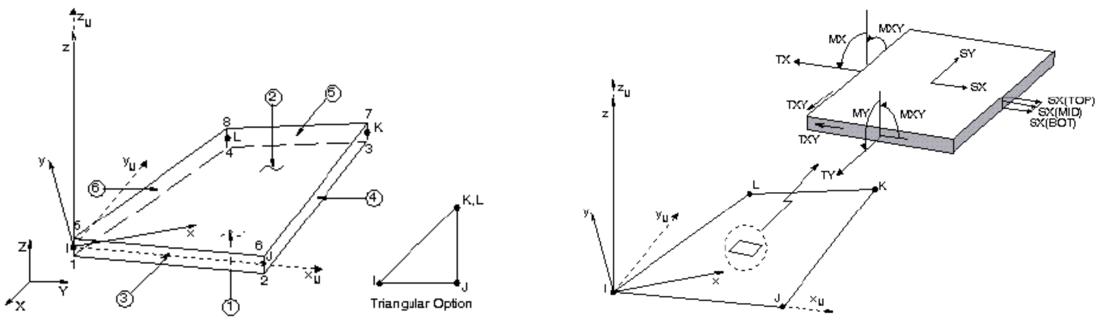

Fig. 3: SHELL63 elastic shell and stress output

Using FEM software ANSYS, column is model with SOLID45 element and slab with SOLID45 and SHELL63. SOLID45 element is defined by eight nodes having three degrees of freedom at each node - translation in the nodal x, y and z direction (figure 2). SHELL63 element has both bending and membrane capabilities (i.e. both in-plane and normal loads are permitted) having six degrees of freedom at each node - translation in the nodal x, y and z direction and rotation in the nodal x, y and z direction (figure 3). Since the zone near the slab-column joint is considered critical, this zone is modelled with SOLID45 elements. It is found from past test results that the zone of maximum stress always develops into the intersection of column strips. Column strip is the zone extending from centre line of the column up to quarter line of span length. This intersection portion of column strip is modelled with SOLID45. The rest portion of the slab is modelled with SHELL63. An isometric and cross-sectional view of the FE model is shown in figure 4 and figure 5.

Materials

In this study, reinforced concrete is considered as isotropic and homogeneous material. In reality, both slab and column are made of reinforced concrete but in this case no reinforcement has been modelled. The following are the assumed material properties used for concrete.

Compressive strength, $f_c^/ = 4000$ psi

Modulus of elasticity, $E_c = 57000\sqrt{f_c^/}$ psi = 3604996 psi

Unit weight, $\gamma_{concrete} = 150$ pcf

Poisson ratio (μ) = 0.20

Loads and Boundary Conditions

Any kind of external vertical concentrated and distributed load is omitted and the self-weight of column and slab also neglected. Moment is applied at the joint by applying lateral load on top and bottom edge of the column. Load 'F' is applied on each node of the column edge along –ve 'x' direction at top and +ve 'x' direction at bottom edge so they produce a couple. If number of node on edge is 'n', the magnitude of moment is n×F×120 lb.-inch as the moment arm is = (5+5) feet =10 feet = 120 inch. The slab is fixed along it's perimeter and the displacement of support is zero. The ends of column are kept free. Loads are applied to the structure in a slow and steady fashion (static loads).

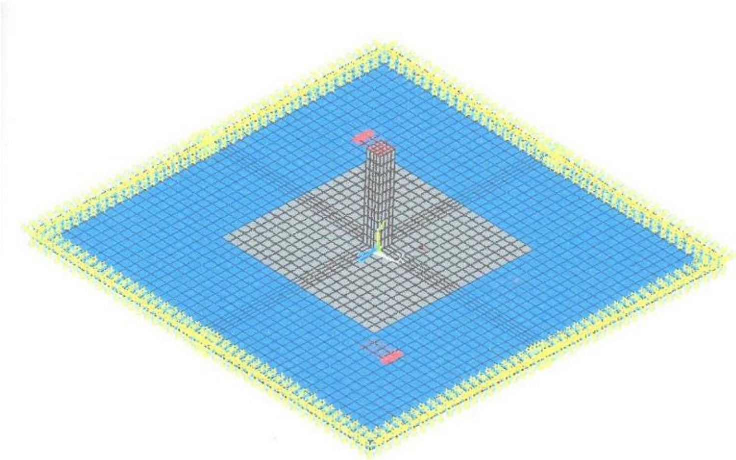

Fig. 4: Isometric view of finite element model of an interior slab-column joint of flat-plate

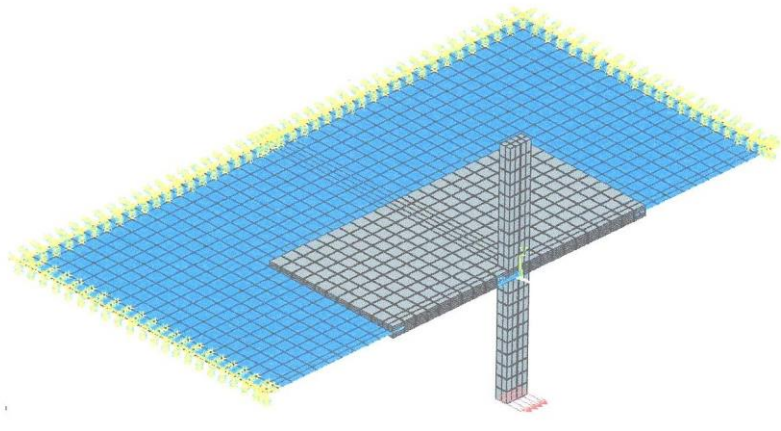

Fig. 5: Cross section of finite element model of an interior slab-column joint of flat-plate

RESULTS AND DISCUSSION

Balanced failure condition means that column and slab fails simultaneously. To find out balanced failure condition, it was tried to determine it by examining the stress. When stress was equal to the ultimate strength of the concrete, it was taken as the failure condition. In reinforced concrete structures, concrete takes the compression as it is strong in compression and re-bars takes the tension. As in this model no reinforcement was used - compressive stress was examined only to determine balance failure condition. When the critical compressive stress on the slab elements and column elements are same and equal the ultimate strength of concrete, it is taken as balance failure condition as column and slab reaches the maximum stress.

Balanced Failure Condition of Slab-Column Joint

Balanced slab thickness for different column sizes is shown in Table 1. If the slab thickness is greater than the balanced thickness, the slab will become stiffer than the column. At this condition, if the structure fails due to unexpected, unbalanced moment of large magnitude, the column will fail prior to the failure of the slab and the whole structure may collapse instantaneously. It is huge risk against the safety that should be avoided. In any case, the slab thickness should not be more than the balanced thickness. It should always be less than balanced thickness. Any point below the curve represents soft slab-strong column condition, which is desired. On the other hand, any point above of the curve represents the undesirable strong slab-weak column state. So, the curve of balanced failure shows a boundary. Upper side of the line is risky zone and lower side is safe zone. According to BNBC (1993) minimum size of column is 10". From this discussion, the figure 6 can be drawn showing the risky zone and safe zone for relative slab thickness and column size.

Table 1: Column sizes and slab thickness (from FEM and as per ACI)

Column size (inch × inch)	Balanced slab thickness (inch)	Slab thickness according to ACI (inch)
10 × 10	5.10	7.09
12 × 12	6.35	8.29
14 × 14	7.63	9.49
16 × 16	8.95	10.68
18 × 18	10.33	11.88
20 × 20	11.73	13.08
22 × 22	13.15	14.28
24 × 24	14.60	15.48
26 × 26	16.07	16.67
28 × 28	17.55	17.87
30 × 30	19.10	19.07

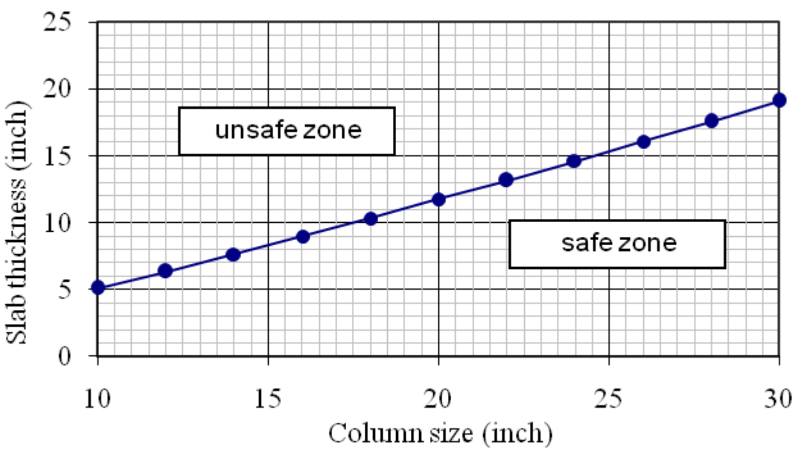

Fig. 6: Risky zone and safe zone for relative slab thickness and column size

Slab Thickness according to ACI

According to the ACI specification, 40% of unbalanced moment is transferred by shear and 60% of unbalanced moment is transferred by flexure for flat-plate with square columns. Attempts have been made to determine the required thickness of the slab to resist this shear. This shear stress can be determined by the following equation

$$v_u = \frac{V_u}{A_c} + \frac{M_{uv} c_r}{J_c}$$

Where, v_u = ultimate shear stress due to external loads

V_u = direct vertical reaction (ultimate) to be transferred to the column

A_c = area of critical section = $2d \times [(c_1 + d) + (c_2 + d)]$

M_u = unbalanced moment to be transferred

M_{uv} = portion of M_u transferred by shear

c_r = distance form centroid of critical section to the right face of section

J_c = property of critical section analogous to polar moment of inertia

$$M_{uv} = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{(c_1 + d)}{(c_2 + d)}}} M_u$$

$$\varphi v_c = 4\varphi\sqrt{f_c^/}$$

$$v_u = \frac{M_{uv} c_r}{J_c}$$

$$c_r = (c + d)/2$$

$$J_c = \frac{2d(c_1 + d)^3}{12} + \frac{2(c_2 + d)d^3}{12} + 2d(c_2 + d)\left(\frac{c_1 + d}{2}\right)^2$$

According to ACI code, in no case, ultimate shear stress v_u exceed φv_c. From this point of view, slab thickness has been determined. This procedure is elaborately explained in the following example.

Let, column dimension, c = c_1 = c_2 = 10 inch (square column section)

M_u = moment required for balance failure = 1170732 lb.-inch

M_{uv} = moment transferred by shear = $0.4 M_u$ = 468293 lb.-inch

$$\varphi v_c = 4\varphi\sqrt{f_c^/} = 4 \times 0.85 \times \sqrt{4000} \text{ psi} = 215 \text{ psi}$$

$$v_u = \varphi v_c$$

$$\Rightarrow \frac{M_{uv} c_r}{J_c} = 4\varphi\sqrt{f_c^/}$$

In this example, V_u = 0, as no vertical load was considered.

$$\Rightarrow \frac{468293 \times \left(\frac{10+d}{2}\right)}{\frac{2d(10+d)^3}{12} + \frac{2(10+d)d^3}{12} + 2d(10+d)\left(\frac{10+d}{2}\right)^2} = 215$$

This equation has been solved by trial and error method and found the value of d = 6.09 inch. If 1 inch cover is considered below reinforcement, the thickness becomes 7.09 inch. Similarly, the slab thickness for other column sizes was calculated and is presented in Table 1.

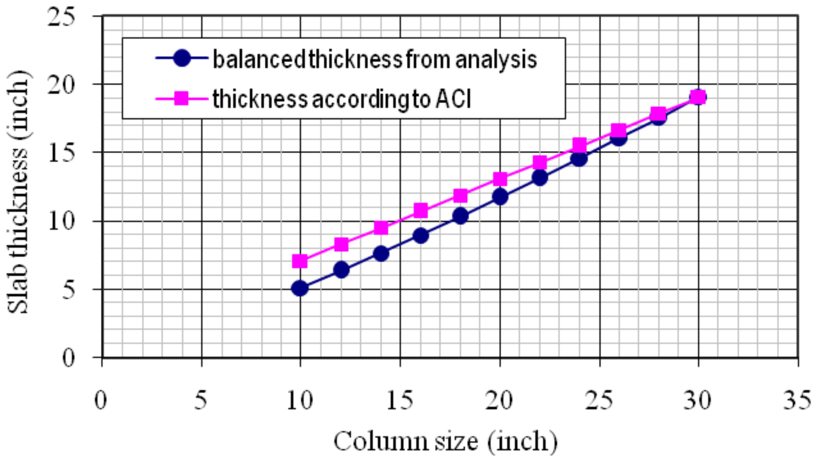

Fig. 7: Column size vs. slab thickness for balanced stress

Comparison between Balanced Thickness and ACI Required Thickness

In previous articles, balanced slab thickness and ACI required thickness for different column sizes has been determined. This comparison is shown in figure 7. It is found that ACI required thickness slab thickness is more than the balanced slab thickness. This indicates that slab is stiffer than column and column may fail first and may lead to the total collapse of the structure. Difference between two slab thicknesses diminishes with increase of column size. For 10" × 10" column size, the balanced thickness from analysis is 5.10" but ACI required thickness is 7.09". For 20" × 20" column size, their difference is reduced to 1.35" and finally for 30" × 30" column size, two thicknesses come nearly equal.

CONCLUSION

To ensure proper safety of flat-plate structure, it is required to maintain weaker slab-stronger column condition. For this, a model of slab-column joint was implemented by finite element software ANSYS and studied its behavior. Balanced slab thickness for different column sizes for which the slab and column failed simultaneously was determined. This balanced slab thickness has been compared with ACI required thickness to investigate the adequacy of the code stipulation. The summary of the study and some recommendations for future investigations are given below -

■ Comparing balanced slab thickness with that of ACI requirement, it has been found that the thickness according to ACI code is higher than the balanced thickness. This means that slab is stronger than column. For this condition, if failure occurs due to lateral load, column may fail prior to the slab and this may lead to the total collapse of the structure.

■ Difference between balance thickness and ACI required thickness diminishes with the increase of column size. For smaller column size, the balanced slab thickness is significantly less than ACI required thickness. For higher column size, however, their difference is gradually reduced.

■ In this study, consideration was given only to the punching shear criteria for determining the ACI required slab thickness and analyzed for a particular model of 20 feet × 20 feet slab panel. From this limited scope, it cannot be conclusively summarized that there exists risk in ACI code but it is indicated that there may be a possibility.

■ This study covers the analysis of interior slab-column joint only with square column. Exterior and corner slab-column joint may be investigated with rectangular and circular column. Also, non-linear analysis including effect of reinforcement in slab and column may be carried out in future.

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ID: SEE 051

FINITE ELEMENT MODELING, ANALYSIS AND VALIDATION OF THE FLEXURAL CAPACITY OF RC BEAMS MADE OF STEEL FIBER REINFORCED CONCRETE (SFRC)

M. M. Islam[1], M. S. Rahman[1], S. M. Mitu[1], M. S. Salekin[1], M. R. Islam[1], M. A. Chowdhury[1] & A. Siddique[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.*
**Corresponding Author, <mashfiq7777@gmail.com>*

ABSTRACT

Steel fiber reinforced concrete (SFRC) is a composite material made of hydraulic cement, fine and coarse aggregate and others components commonly used in concrete and a dispersion of discontinuous steel fibers. Fiber reinforcement considerably improves the flexural strength, direct tensile strength, fatigue strength, shear and torsional strength, shock resistance, ductility and failure toughness. Within this context, this paper investigates the flexural capacity of reinforced concrete (RC) beams made of SFRC and modelling in the finite element (FE) platform of ANSYS 11.0. The beams (flexure critical beams) are constructed without any flexural reinforcement, i.e. only shear reinforcement is provided, to observe the actual flexural capacity enhancements due to presence of steel fibers. The steel fibers are prepared in the laboratory and three steel fiber aspect ratios are maintained, i.e. 40, 60 and 80. End enlarged type of fibers are made for this research and the beams are casted with fiber of 3 steel fiber aspect ratios. These beams are tested in a 1000kN capacity digital universal testing machine (UTM). The flexural capacity is found to be enhanced by 50%, 94% and 79% for steel fiber with a aspect ratio of 40, 60 and 80, respectively, compared to control beam (without fiber) specimens. The load deflection behavior, failure pattern and stress & strain distribution of FE models uphold good agreement with the experimental results, which ensures the validation of the FE modelling and analysis. This paper provides real experimental data, and FE modelling on flexure behavior of SFRC RC beams which will be useful for the construction industry of Bangladesh.

Keywords: Steel Fiber Reinforced Concrete (SFRC), Steel Fiber Aspect Ratio, Finite Element (FE) modelling, ANSYS 11.0, RC beams, Flexural Capacity.

INTRODUCTION

Steel fiber reinforced concrete (SFRC) can be considered as a new and totally different concrete construction material in context of Bangladesh. Fiber reinforcement can considerably improve the flexural strength, direct tensile strength, fatigue strength, shear and torsional strength, shock resistance, ductility and failure toughness of concrete matrices (Ramakrishna et al. 1980). Steel fiber reinforced concrete (SFRC) is being increasingly used in shotcrete as tunnel linings, pre-cast structures, off-shore platforms, water-retaining structures, , bridges, industrial or factory pavements, highways, roads, parking areas, bridge decks, airport runways and also in versatile engineering structures and specially the structures in high seismic risk areas. Fiber reinforcement in cement matrices enhance the tensile strength and stiffness of the resulting composite by controlling the crack propagation (Swamy and Al-Ta'an, 198, Islam et al., 2014a). It has been used in concrete since the early 70's and for different applications. In Bangladesh the use of steel fiber reinforced concrete has not yet been started, indeed a lot of researches are on going on fiber reinforced concrete. One of the most beneficial aspects of the use of fibers in concrete structures is that non-brittle behavior after concrete cracking can be achieved with fibers. Finite Element (FE) Analysis software ANSYS 11.0 is used to model and analyze the RC beams made of plain concrete and SFRC made of stone concrete. A reasonable modelling of concrete, SFRC and reinforcement on a finite element (FE) platform using suitable element type, adequate mesh size, appropriate boundary conditions, realistic loading environment and proper time stepping can help to estimate the structural responses. A satisfactory and reliable agreement is found between the experimental specimens and the FE models. Modulus of elasticity, Poisson's ratio and tensile strength of concrete are found to be the main controlling parameters. A good correlation is obtained between FE model and experimental results. This experimental investigation is intended validate the ANSYS 11.0 FE model of SFRC and plain RC beams with the experimental results to provide a reliable FE model for analyzing future problems on SFRC.

MATERIALS, METHODOLOGY AND RESULTS

This research intends to investigate the flexural capacity enhancement of RC beams made of SFRC experimentally as well as in the FE framework of ANSYS 11.0. To estimate the actual flexural capacity enhancement of beams due to steel fiber mixed in the concrete matrix, the beam is casted with no longitudinal reinforcement (Fig. 1a), i.e. only shear reinforcement is provided and also the middle span is kept free from any shear. The web reinforcements are connected by soft wooden stick (Fig. 1b) to ensure that no contribution of flexural strength by the wooden sticks so that only the contribution of steel fibers may be evaluated. A total of 4 flexure critical (FC) beam specimens (one control i.e. plain concrete and three SFRC) of size 6x5x36 inch (150x125x900 mm) are made and tested in a digital universal testing machine (UTM) of 1000kN capacity at constant rate of 0.5 mm/min until the failure. Three different steel fibers aspect ratio (SFAR) are selected i.e. 40, 60 and 80. Mild steel cold drawn wires (Type V as per ASTM A 820/A 820M-06) are selected to make the steel fibers and are prepared manually in the laboratory. In this case the diameters of the steel fiber is 1.18 mm, so the lengths are differed for different aspect ratio. Effective lengths for the SFAR 40, 60 and 80 are 47.2 mm, 70.8 mm and 94.4 mm respectively and the original lengths of fibers are 67.2 mm, 90.8 mm and 114.4 mm respectively. The steel fiber volume ratio is maintained 1.5%. According to ACI-544.4R-88, enlarged-end fibers show the maximum energy absorption as well as tensile strength enhancement so that the fibers are bent at 120^o at the ends to make the enlarged ends (Fig. 1 c, d). The tensile strength of steel fiber is 1100 MPa (160 ksi) which satisfies the minimum requirement of ASTM A 820/A 820M-06 (i.e. 345 MPa or 50 ksi). All the beams are made of stone concrete (SC) and are made with OPC with a Cement:FA:CA of 1:1.5:3 and w/c of 0.5. To estimate the compressive and splitting tensile strength of plain concrete and SFRC, cylinders are tested in same UTM. Mild steel rebar of 60 grade (yeild strength 60 ksi or 420 Mpa) is used as shear reinforcement. To measure the lateral strain of cylinders at the failure location, lateral displacements are measured by analyzing the image histories obtained from high speed video clips and high definition (HD) images employing an image analysis technique which is called digital image correlation technique i.e. DICT (Islam et al. 2011, Islam et al. 2014b). This is a customized simultaneous data acquisition system in which the load and displacement data obtained from the load cell of UTM are synthesized with the strain results gathered from DICT. The compressive strength increases about 17.6% for steel fiber aspect ratio 40 with respect to control specimen but for steel fiber aspect ratio 60 and 80 it doesn't show any significant effect due to uneven distribution of concrete in cylinders for larger length of steel fibers. But in case of ductility is increased about 5, 3.6 and 3 times for steel fiber aspect ratio 40, 60 and 80 respectively which one of the major requirement of this investigation. For steel fiber aspect ratio 40, 60 and 80 the tensile strength enhanced about 58%, 117.5% and 64.1% respectively and the ductility also increased about 15, 9.2 and 13 times for steel fiber aspect ratio 40, 60 and 80 respectively. Table 1 and 2 provides the experimental results of the compressive, tensile and beam testing results and also graphically represented in Fig. 2.

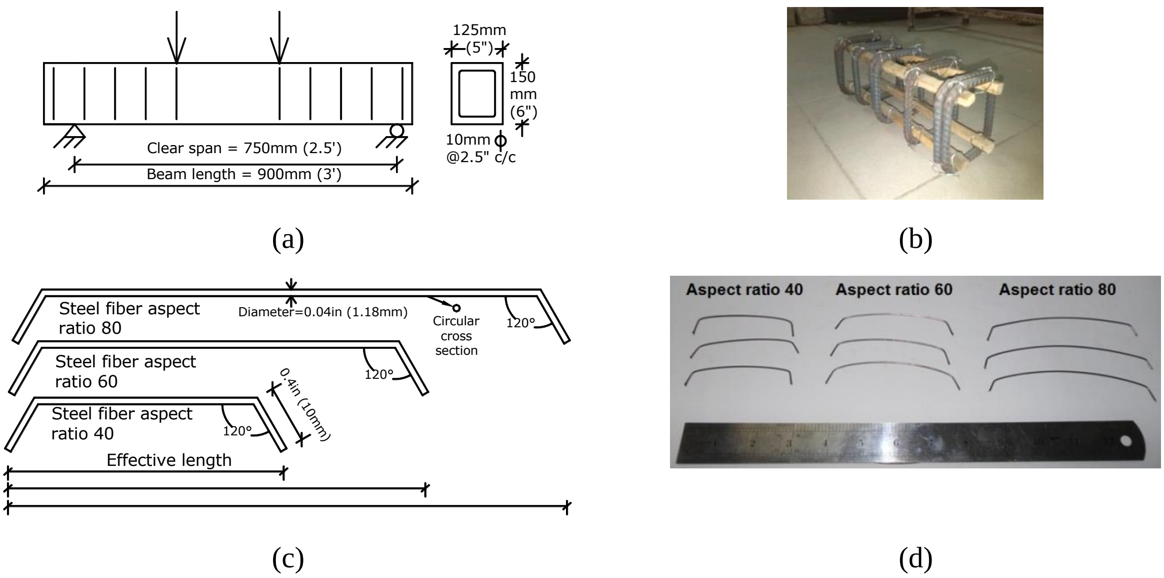


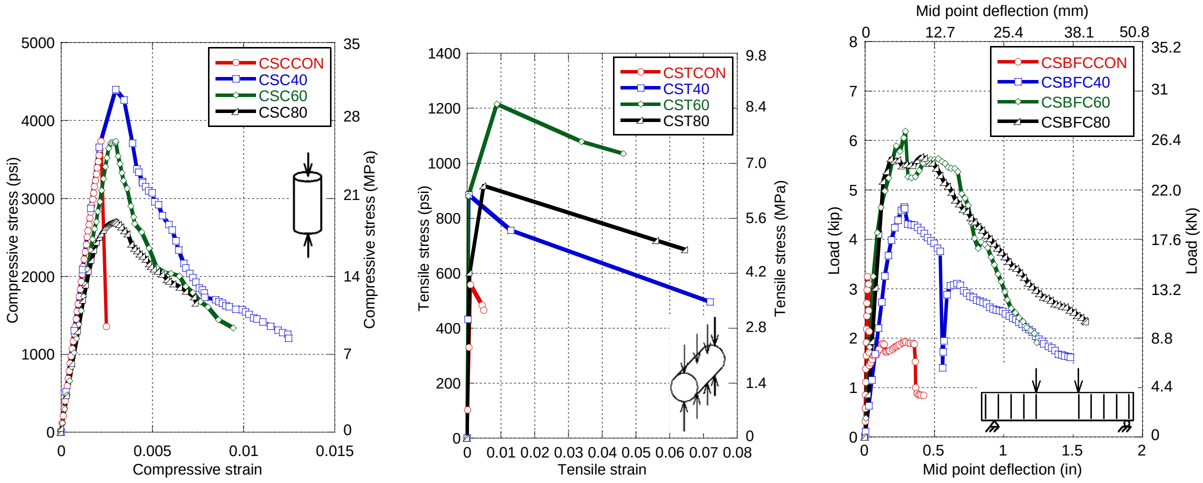
Fig. 1: (a) beam detailing, (b) shear reinforcement, (c) and (d) geometry and images of steel fibers.

Table 1: Summary of the concrete cylinder specimen

Specimen ID	Maximum compressive strength, psi (MPa)	Specimen ID	Maximum tensile Strength, psi (MPa)	Steel fiber aspect ratio
CSCCON	3741 (26)	CSTCON	560 (4)	-
CSC40	4400 (30)	CST40	885 (6)	40
CSC60	3733 (25.7)	CST60	1218 (8.5)	60
CSC80	2691 (19)	CST80	919 (6.5)	80

Table 2: Summary of results of the FC beam specimens

Specimen ID	First cracking load, kN (kip)	Enhanced load capacity (%)	Ultimate load kN (kip)	Enhanced Load capacity (%)	Ductility enhanced (times)	Enhanced Ductility (%)	Steel fiber aspect ratio
CSBFCCON	6.4 (1.5)	-	14 (3.2)	-	-	-	-
CSBFC40	11 (2.5)	66.67	21 (4.67)	50	3.7	270	40
CSBFC60	19 (4.3)	186.67	27.2 (6.18)	94	3.125	212.5	60
CSBFC80	23.1 (5.25)	250	25 (5.67)	79	4	300	80



(a) (b) (c)

Fig. 2: Experimental results of plain concrete and SFRC (a) compression (b) splitting tension (c) load deflection behaviour of beams.

FE MODELLING AND ANALYSES

Plain concrete and SFRC are modeled using SOLID65 element in ANSYS 11.0, which is a three dimensional solid element having eight nodes with three degrees of freedom at each node, i.e., translation in the nodal x, y and z directions. The SOLID65 element (Fig. 3a) is capable of plastic deformation, cracking in tension, crushing in compression, modelling reinforcement behaviour and is also applicable as reinforced composites (ANSYS 2005), such as, fibreglass or steel fiber reinforced concrete (SFRC). The shear reinforcements are modeled using LINK8 element, which is a 3D spar element with three degrees of freedom at each node same to SOLID65 (Fig. 3b). Table 3 shows the input data for SOLID65 and LINK8. Fig. 3c and 3d shows the FE model of beam and deformed shape. Fig. 4 represents the validation of load deflection behaviour gathered from the experimental investigations with the FE analysis. Typical stress and strain contours of FC beam specimens are shown in Fig. 5 and Fig. 6 shows the similarity of failure patterns between the experimental beams and FE models done on ANSYS 11.0. The beams are not reinforced with longitudinal rebar so only flexural cracks are observed in both cases. The flexural cracks are found at mid-span and all cracks are vertical. In the both cases the failure pattern is quite similar which validates the FE modeling and analysis. This indicates that the FE modeling of SFRC RC beams using the pertinent parameters gathered from experimental testing are valid and there remains a good agreement as well as it can be used in future SFRC model of different fiber volume and span lengths.

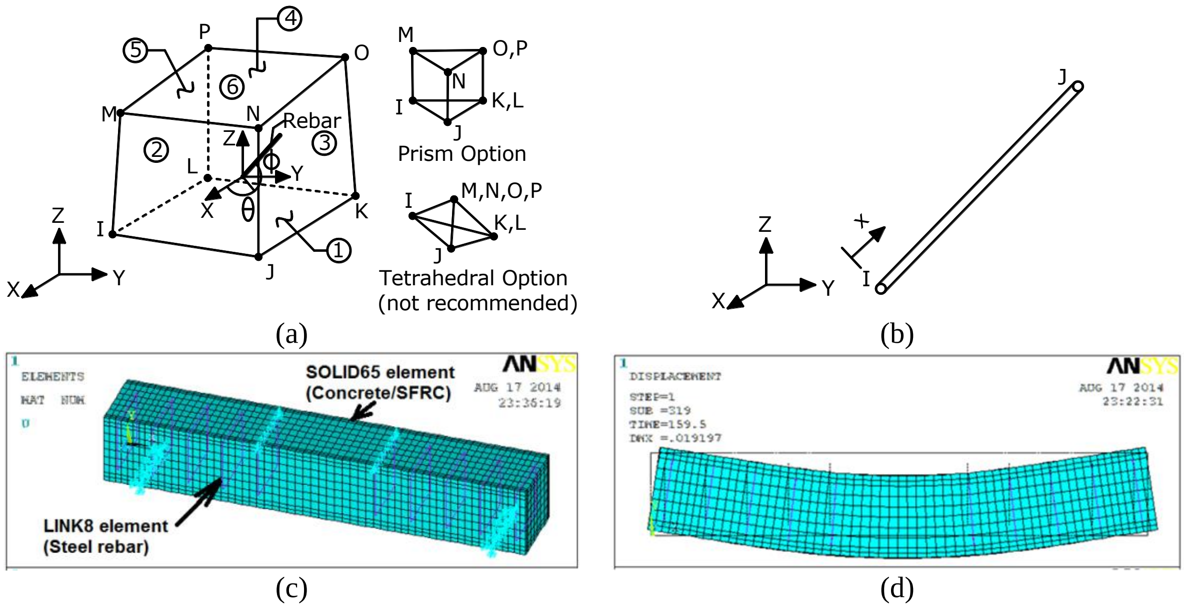


Fig. 3: (a) Geometry of SOLID65 (b) geometry of LINK8 element in ANSYS 10.0 platform (c) meshing and boundary condition and (d) deformed shape.

Table 3: FE input data for SOLID65 and LINK8 element

Properties for FE model	Beam specimen (SOLID65)				Rebar (LINK8)
	CSBFCCON	CSBFC40	CSBFC60	CSBFC80	
Density	$2.69 g/cm^3$	$2.77 g/cm^3$	$2.72 g/cm^3$	$2.74 g/cm^3$	$7.8 g/cm^3$
Tensile strength	4 MPa	6 MPa	8 MPa	6.3 MPa	-
Poisson's ratio	0.325	0.325	0.325	0.325	0.3
Shear transfer Co-efficient: Closed crack	0.5	0.5	0.5	0.5	-
Open crack	0.3	0.3	0.3	0.3	-
Yield stress	-	-	-	-	420 MPa

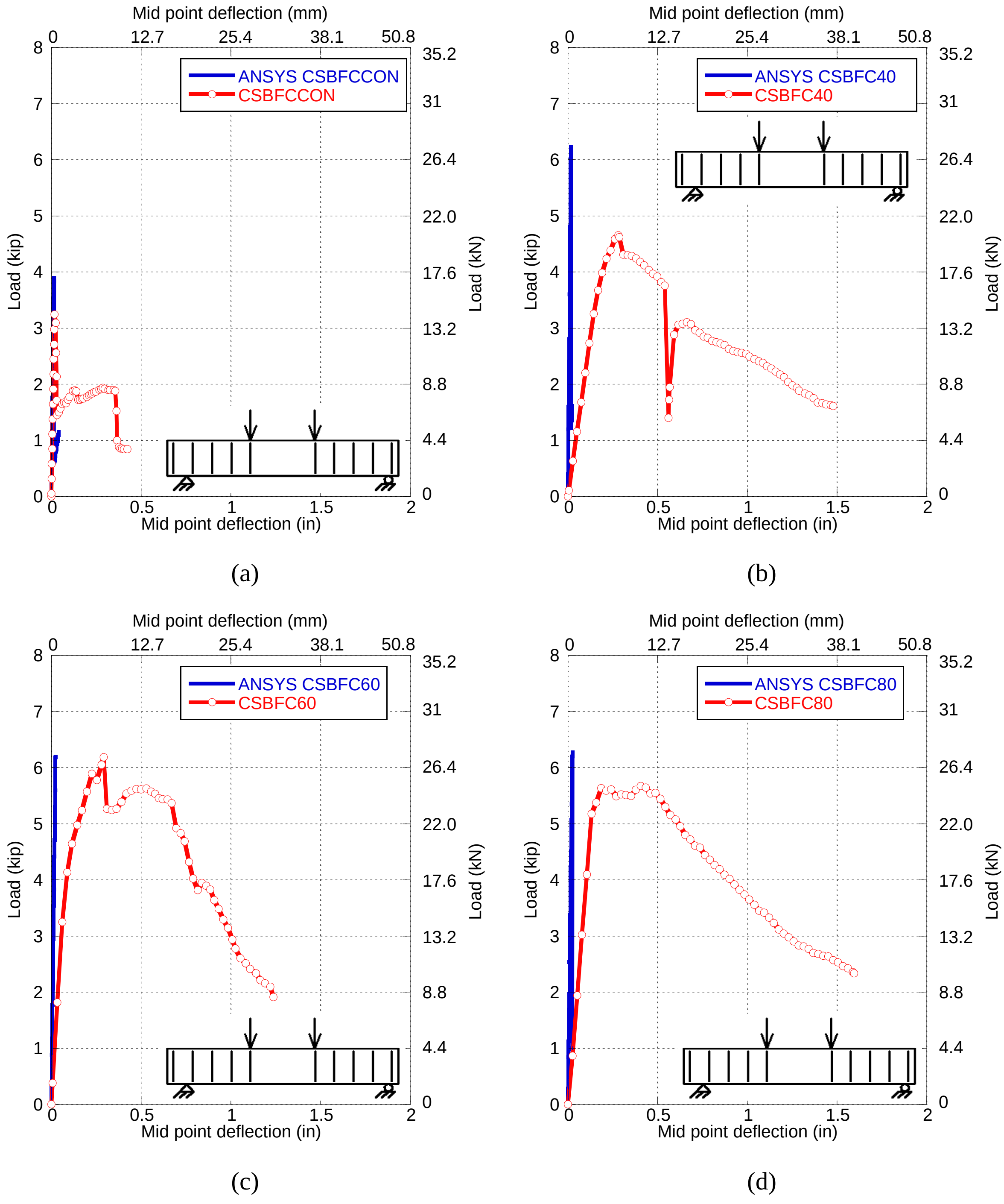
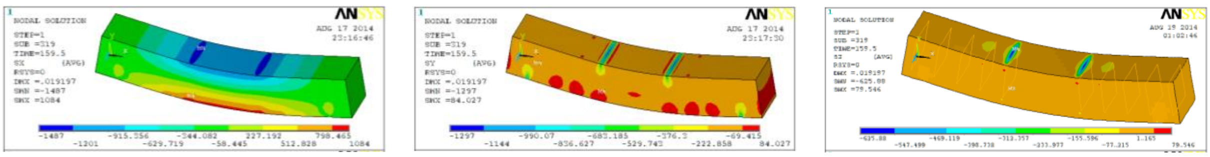
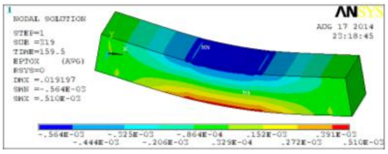


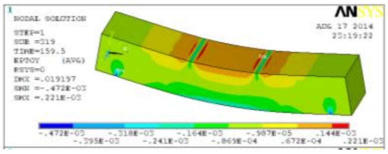
Fig. 4: Evaluation of load deflection behaviour FE and experimental FC beams a) CSBFCCON i.e. control beam b) CSBFC40 c) CSBFC60 d) CSBFC80.



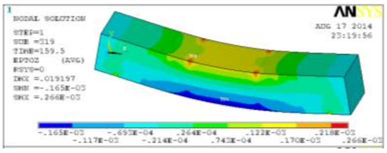
(a) (b) (c)



(d)

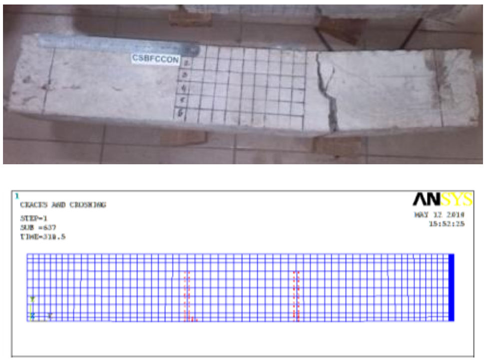


(e)

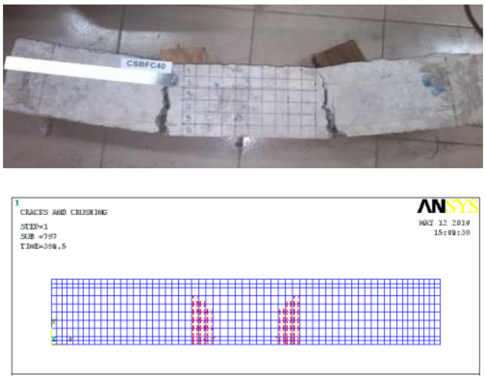


(f)

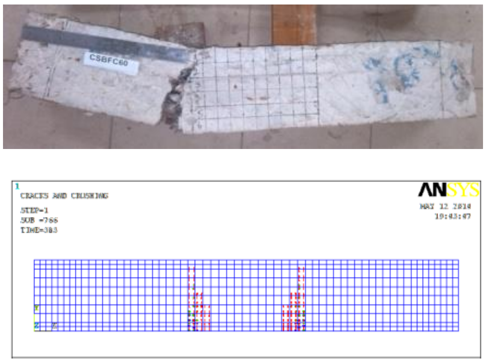
Fig. 5: (a), (b), (c) Typical stress and (d), (e), (f) strain contours at X, Y, Z direction respectively.



(a)



(b)



(c)

(d)

Fig. 6: Experimental and FE failure pattern (a) CSBFCCON (b) CSBFC40 (c) CSBFC60 (d) CSBFC80.

CONCLUSION

The compressive strength increases about 17.6% for SFAR 40 with respect to control specimen and ductility is increased about 5, 3.6 and 3 times for SFAR 40, 60 and 80 respectively, the tensile strength enhanced about 58%, 117.5% and 64.1% respectively and ductility increased about 15, 9.2 and 13 times respectively. The load-deflection behavior shows that the flexural strength increased about 50%, 94% and 79% for the SFAR 40, 60 and 80 respectively and ductility enhanced 3.7, 3.125 and 4 times respectively. The FE model showed similar load deflection behavior upto peak load and similar failure pattern that validates the FE modeling. This paper can provide the construction industry of Bangladesh with reliable experimental results and FE modeling of SFRC.

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EXPERIMENTAL INVESTIGATION OF THE SINGLE SHEAR, DOUBLE SHEAR AND FLEXURAL SHEAR BEHAVIOR OF STEEL FIBER REINFORCED CONCRETE (SFRC)

M. M. Islam[1]*, M. S. Khatun[1], J. F. Dola[1], M. R. U. Islam[1], M. Hussan[1] & A. Siddique[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.*
**Corresponding Author, <mashfiq7777@gmail.com>*

ABSTRACT

Experimental investigations are conducted to study the enhancement of shear capacity in single shear, double shear and flexural shear specimens made of Steel Fiber Reinforced Concrete (SFRC). In this study, low aspect ratio steel fibers are used and sufficient capacity enhancement is attained. Two different aggregate types are used, i.e. stone and brick aggregates to make plain concrete and SFRC specimens. The SFRC specimens show substantial increase in compressive, tensile, and shear capacity as well as ductility due to the presence of steel fibers. In context of Bangladesh, this study also investigates the effectiveness of locally available steel fibers. The experimental plan intended to investigate the shear capacity enhancement of three different types of beams, i.e. single shear beam, double shear beam and flexural shear beam specimens. First two types are unique specimens and not yet analyzed before in this kind of investigations. Total 8 cylinders and 12 shear specimens are cast and analyzed in this study. Steel fiber volume ratio of 1.5% is maintained to cast the SFRC specimens. Experimental tests have shown the increase in shear capacity of SFRC by 30% to 170% with significant increase in ductility as well. Finally, it can be said that the SFRC can be suitably used in structural components to increase shear capacity as well as ductility which results in reducing the risk of brittle failure of structural components during earthquake and any other natural load.

Keywords: Steel Fiber Reinforced Concrete (SFRC), Shear Capacity, Single Shear, Double Shear, Flexural Shear.

INTRODUCTION

Fiber reinforced concrete is ordinary concrete containing discontinuous, discrete fibers of short length and small diameter (Mansur and Paramasivam, 1985). Steel fiber reinforced concrete (SFRC) is a composite material whose components include the traditional constituents of port-land cement concrete (hydraulic cement, fine and coarse aggregates, and admixtures) and a dispersion of randomly oriented short discrete steel fibers (Aoude et al., 2009). Fibers in the concrete serve as crack arrestors by applying pinching forces at crack tips, thus delaying the appearance of cracks and creating a stage of slow crack propagation. In the concrete matrix, steel fiber reinforcement enhances shear resistance by transferring tensile stresses across diagonal cracks and reducing diagonal crack spacing and width, which increases aggregate interlocking (Dinh et al. 2010). The ductility of the composite is increased many fold, compared to the unreinforced matrix, with a corresponding increase in strength. Steel fibers can be substituted for conventional reinforcement, totally or partially, thus, reducing the working time for placing conventional reinforcement (reinforcing bars or welded mesh) and cutting construction costs (Meda and Plizzari, 2004). The use of steel fiber-reinforced concrete (SFRC) is increasing due to its improved material and structural behavior relative to plain concrete and even to conventionally reinforced concrete with the same steel volume fraction (Kang et al., 2011). One of the most beneficial aspects of the use of fibers in concrete structures is that non-brittle behavior after concrete cracking can be achieved with fibers (Lee et al., 2011). Adding steel fibers increases ultimate shear strength, reduces deflections, increases stiffness and transforms failure modes from brittle and dangerous shear failures into more ductile flexural failures (Yakoub, 2011). The objectives for the addition of fibers are to improve the tensile strength, flexural strength, impact strength or toughness to change the mode of failure by means of post cracking ductility to control cracking (Traina and Mansour, 1991). The main applications of steel fiber-reinforced concrete (SFRC) are in structures subjected to potentially damaging concentrated and dynamic load (Balaguru and Najm, 2004).

In this work, experimental investigation is done to evaluate the shear capacity enhancement of SFRC. To this end, three types of shear specimens, i.e. single shear, double shear and flexural shear specimens are made to investigate the effects on shear capacity due to steel fibers. Though SFRC possesses many advantages, its use in the construction industry of Bangladesh is not yet started. This work will provide reliable experimental results on shear capacity of SFRC which will surely help the construction industry to start use this engineering material.

EXPERIMENTAL PROGRAM

According to ACI-544.4R-88, enlarged-end fibers show the maximum energy absorption as well as tensile strength enhancement so that, enlarged end fibers with 1.5% volume fractions are used to cast the shear specimens. Mild steel cold drawn wires (Type V as per ASTM A 820/A 820M-06) are selected to make the steel fibers and are prepared manually in the laboratory. The fibers are slightly customized to make enlarged ends for better anchorage (Fig. 1a). The tensile strength of steel fiber is 552 MPa (80 ksi) which satisfies the minimum requirement of ASTM A 820/A 820M-06 (i.e. 345 MPa or 50 ksi) and aspect ratio is 22. Two different kinds of aggregates are used to make the SFRC shear specimens and also plain concrete specimens, i.e. stone (CS is used in the specimen designation) and brick (CB) aggregate and the effects of SFRC on these two types of concretes are investigated. Significant increase of the shear capacity is achieved using this kind of steel fibers. Three kinds of shear capacity are tested, i.e. single shear (SS), double shear (DS) and flexural shear (FS). A total of 4 compression cylinders (C), 4 split tensile cylinders (T), 4 single shear, 4 double shear and 4 flexural shear specimens are made for this study, of them half of the specimens are control (CON) specimen i.e. made of plain concrete. The dia of the cylinder specimens are 100mm (4in) and length is 200mm (8in). The dimension of the flexural shear specimen is 153x153x610 mm (6x6x24 in) and is analyzed with two-point loading and each load was placed at a distance of 153 mm (6in) from the end supports. Support is placed 76 mm (3 in) from the edge of the beam. The span between the two supports was 458mm (18 in). These beams are restrained only vertically as it was tested experimentally. The dimension of S-shaped single shear specimens is 100x100x400mm (4x4x16 in) for the main beam part and is casted with a relatively higher stiffened part monolithically, which is used apply the single shear. The dimension of double shear specimen is also 100x100x400mm (4x4x16in) and again monolithically stiffened parts are casted to apply double shear. The single shear and double shear specimens are notched at the shear critical location to avoid bending during load application. All the shear specimens are tested in a 1000kN capacity digital universal testing machine (UTM) which is a displacement control machine (Fig. 1b). The lateral strain data are measured by employing digital image correlation technique (DICT) using high definition (HD) images and high speed video clips (see also Islam et al. 2011, Islam et al. 2014a,b) and these strain data are synthesized with the load data gathered from the load cell of UTM.

Table 1: Experimental test results of cylinder specimens

Specimen ID	Comp. stress	% Increased	Ductility increased	Specimen ID	Tensile stress	% Increased	Ductility increased

	(MPa)		(times)		(Mpa)		(times)
CBCCON	26	-	-	CBTCON	5	-	-
CBCSFRC	28	8	1.2	CBTSFRC	6.5	30	1.1
CSCCON	28	-	-	CSTCON	3	-	-
CSCSFRC	34	21	1.2	CSTSFRC	8	166	2

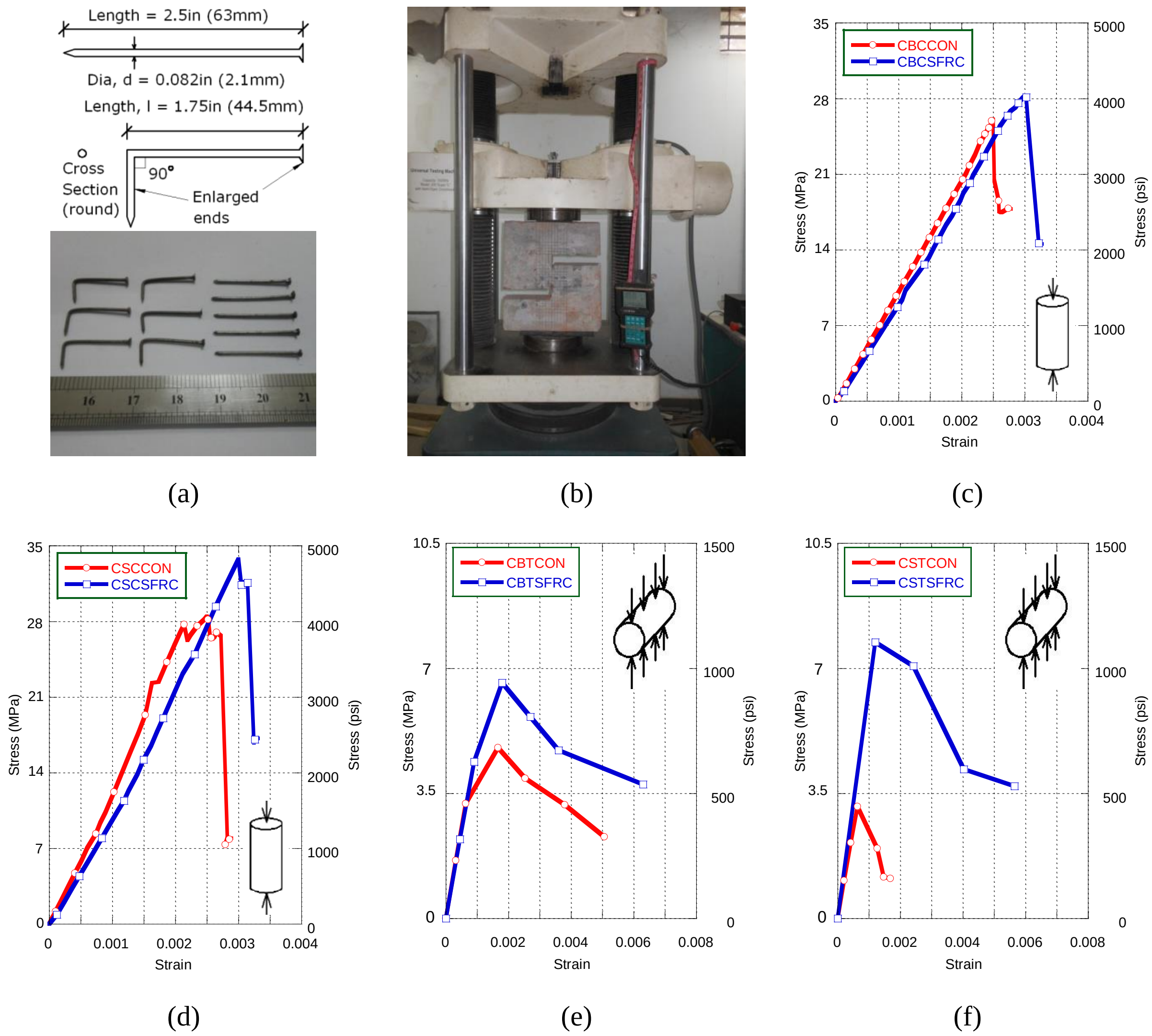


Fig. 1. (a) Geometry and image of steel fiber, (b) test setup of single shear specimen, (c), (d) compressive behavior and (e), (f) split tensile behavior of cylinders made of brick and stone concrete respectively.

Table 2: Experimental test results of shear specimens

Specimen ID	Shear stress	% Increased	Ductility increased	Specimen ID	Shear stress	% Increased	Ductility increased

	(MPa)		(times)		(MPa)		(times)
SSCBCON	3	-	-	DSCSCON	7.5	-	-
SSCBSFRC	8	167	3	DSCSSFRC	17	127	2
SSCSCON	4	-	-	FSCBCON	0.3	-	-
SSCSSFRC	8	100	6	FSCBSFRC	0.5	67	2
DSCBCON	7	-	-	FSCSCON	0.8	-	-
DSCBSFRC	15	114	1.4	FSCSSFRC	1.1	38	1.4

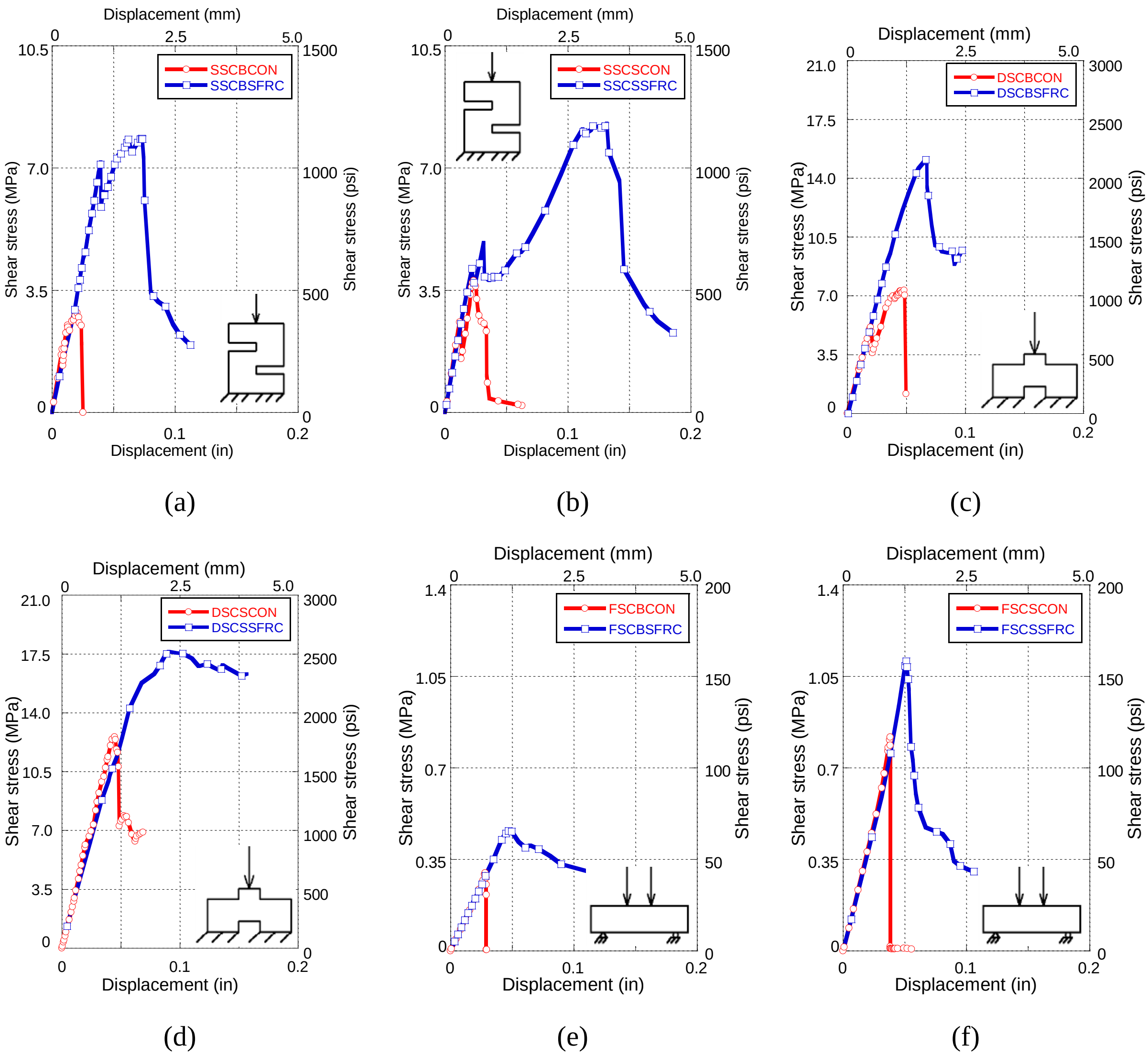
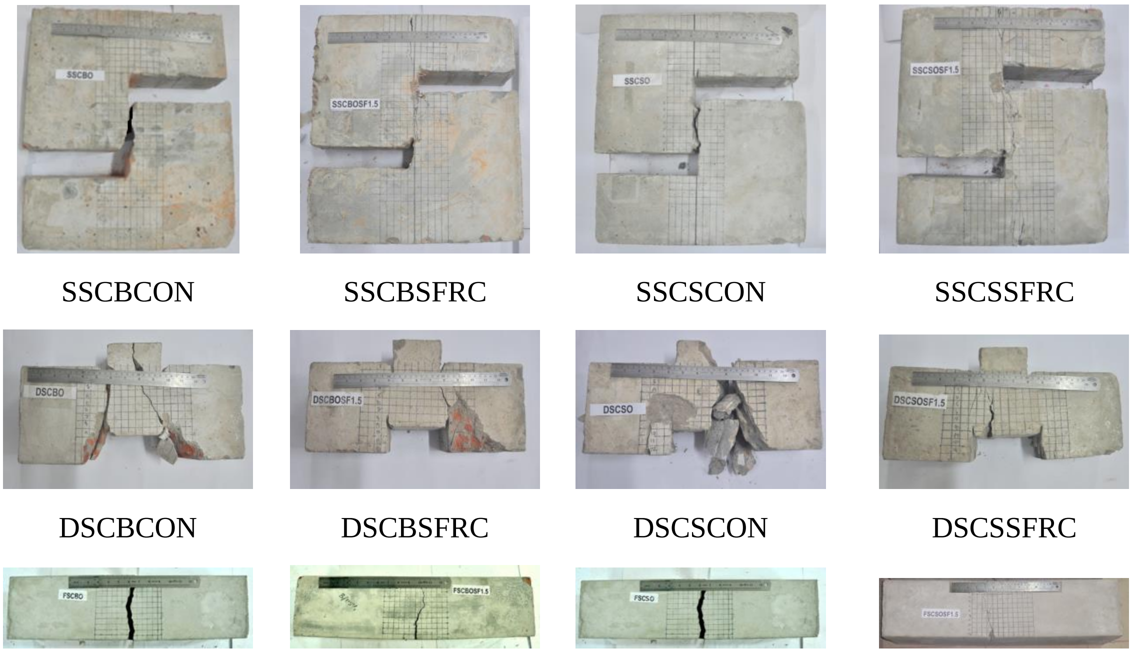


Fig. 2. Shear stress vs. displacement behavior of shear specimens (a), (b) single shear, (c), (d) double shear and (e), (f) flexural shear made of brick and stone concrete respectively.



SSCBCON SSCBSFRC SSCSCON SSCSSFRC

DSCBCON DSCBSFRC DSCSCON DSCSSFRC

FSCBCON FSCBSFRC FSCSCON FSCSSFRC

Fig. 3. Failure pattern of the shear specimens

RESULTS AND DISCUSSION

Fig. 1 (c) to (f) represents the compressive and tensile behavior of cylinder specimens made of brick and stone plain concrete and SFRC. The compressive strength is increased 8% and 21% for brick and stone SFRC respectively (Fig. 1c,d) and ductility increased 1.2 times. Again split tensile strength is increased 30% and 166% for brick and stone SFRC respectively and ductility increased 1.1 and 2 times respectively (Fig. 1e,f). These results are summarized in Table 1. Fig. 2 represents the experimental test results of the shear specimens. The single shear strength is increased 167% and 100% for brick and stone SFRC respectively compared to respective plain concretes. The double shear strength enhanced 114% and 127% and flexural shear strength enhanced 68% and 38% for brick and stone SFRC respectively. The ductility enhanced 3 and 6 times for single shear, 1.4 and 2 times for double shear and 2 and 1.4 times for flexural shear specimens made of brick and stone SFRC respectively. These results are summarized in Table 2. The failure patterns of the shear specimens are represented in Fig. 3. The plain concrete specimens showed wide cracks as well as complete separation whereas SFRC specimens showed ductility by crack bridging by steel fibers and slow cracks propagation.

CONCLUSION

The use of steel fibers in concrete construction is not yet been started in Bangladesh for the lack of reliable experimental results though SFRC possesses many advantages like improved tensile, compressive and shear strength, superior crack control, ductility etc. The compressive strength of SFRC is increased 8% to 21% and tensile strength is increased 30% to 166%. The single shear strength is increased 167% and 100% for brick and stone SFRC respectively compared to respective plain concretes. The double shear strength enhanced 114% and 127% and flexural shear strength enhanced 68% and 38% for brick and stone SFRC respectively. The ductility enhanced 3 and 6 times for single shear, 1.4 and 2 times for double shear and 2 and 1.4 times for flexural shear specimens made of brick and stone SFRC respectively with a evidence of superior crack control. This paper intends to introduce the use of SFRC in the construction industry of Bangladesh to construct safe structures against seismic loading or other deadly natural loads.

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EFFECTS ON CABLE CONFIGURATION DUE TO CHANGE IN DIFFERENT GEOMETRIC PARAMETERS OF EXTRADOSED BRIDGES

Shisir Barua[1*] & Abu Md. Jubary[1]

[1] *Department of Civil &Environmental Engineering, Uttara University, Uttara, Dhaka-1230, Bangladesh.*
**Corresponding Author, <shisir.ce007@gmail.com>*

ABSTRACT

The purpose of this paper is to carry out a review study of an extradosed bridge. The study is aimed at checking how different geometric parameters, such as span length, tower height and deck section dimensions influence the cable configuration, such as cable layout, no. of cables, cable inclination, and cable force. The super-structural information of the Third Karnaphuli Bridge located at Chittagong, Bangladesh has been considered for this study. A detailed structural modelling and analysis of a Stiff Girder Pre-stressed Post-tensioned Extradosed Bridge "Third Karnaphuli Bridge" has been carried out using CSi Bridge v15.0.0 software according to AASHTO LRFD Bridge Design Specification. The parameters included variable span lengths and deck section to determine the extradosed cable configuration. The study shows that it is possible to balance a portion of dead load stresses in addition to live load using extradosed cables but the number of cables needed to be increased. As the span length increased, cable inclination was decreased and the vertical component of cable force was also decreased.

Keywords: Third Karnaphuli Bridge, deck depth and mast height, length supported by cables, side span length, cable allowable stress, CSi Bridge v15.0.0

INTRODUCTION

Generally extradosed bridges are transitional between girders and cable-stayed bridges. Whereas cable-stayed decks have little stiffness or strength compared with the stays which carry most of the loads, extradosed decks are stiffer compared with their cable systems and the decks carry a significant proportion of the loads. The reduced cable inclination in an extradosed bridge leads to an increase in the axial load in the deck and a decrease in vertical component of force at the cable anchorages. Thus, the function of the extradosed cables is also to pre-stress the deck, not only to provide vertical support as in a cable-stayed bridge. Extradosed bridges are characterized by a low live load stress range in the stay cables. In extradosed bridges post-tensioned extradosed tendons are used above the deck anchored in low towers to balance the live load stress.

METHODOLOGY

Approach of the Study

The study has been organized so as to best describe and discuss the problem and the resulting findings. Detailed structural modeling and finite element analysis of a stiff girder Pre-stressed Post-tensioned Extradosed bridge using CSI BRIDGE v15.0.0 integrated 3D bridge design software has been precisely carried out. The extradosed bridge model included a total length of 430m with a 200m center span, two 115m side span and a centrally supported single cell deck section. The total depth of the box section was 4m at mid span and 6.75m at pier location. The deck was parabolically varied along the depth, top flange, and bottom flange and exterior webs to reduce the self-weight of the bridge. The deck was pre-stressed longitudinally by internal bonded pre-stressing tendons with twenty-seven 15.7 mm diameter strands passing through the top and bottom slabs. The minimum breaking strength of the strand was 1862 MPa. A total number of 24 cables consisting of 91, 15.7mm diameter strands were used as extradosed cables. For the Karnaphuli design, approximately 15% of vertical dead load is picked up by the cables and transmitted to the towers (Abu Saleh, 2010).

Design Specification

Deck depth and mast height

In an article published in 1988, Mathivat a French engineer proposed a constant depth deck, slender L/h from 30 and 35, a mast height so that *L/H* is equal to 15. Komiya (1999) suggested for pier embedded bridges edge with 35 slenderness in the pier support section and 55 in the main center span and, mast heights ranging from L/12 and L/8. Chio (2000) proposes project criterion using an edge for the pier support edge of L/30 and central span L/45, i.e. ha/hc equal to 1.5. He also recommends a mast height equal to L/10, so that rods tension oscillations due to live load would be delimited by 80 MPa value. Dos Santos (2006) proposed a steady deck height L/33 and mast height L/10, however, since he did not considered concrete deformation and steel relaxation effects, his proposal has a limited applicability.

Length supported by cables

Since tension rods close to the mast are powerless in a fan tension rod arrangement, Chio (2000) recommended that the first tension rod should be fixed between 0.18 and 0.25 from center span. Such value differs from Mathivat, who suggested that the first tension rod should be fixed at 0.1 from central span. According to Komiya (1999 quoted by Mermigas, 2008) the combined cost for extradosed cables and internal tension roads fixed at 0.14, 0.20 and 0.24 from central span, has a variation of approximately 2% among them and, the most cost effective arrangement is the one corresponding to the first fixed stay cable at 0.20 from main span.

Side span length

For stiff-deck extradosed bridges, Chio (2000) recommended side span lengths lower than 60% from main span, but higher than 40%. Dos Santos (2006) suggested lengths between 60% and 65% from main span. Supported in the similarity held between extradosed bridges and cantilever constructed pre-stressed box-girder bridges, Kasuga suggests side span length between 60% and 80% from main span. Previous recommendations are not of great application in extradosed bridges which are similar to cable-stayed bridges: Mermigas (2008) found that in stiff deck extradosed bridges with 140 m main span, it is not possible to use side spans which length is 50% higher than main span length, because moments are quite significant and exceed deck capacity.

Cables allowable stress in serviceability limit state (SLS)

In his proposal, Mathivat (1998) employed an allowable stress in tension rods of 0.6 *fpu*, which is a criterion adopted by the first constructed extradosed bridges (Kasuga, 2002; Ogawa et al., 1998b; Tomita et al, 1999), since rigid connection between deck and piers, together with main spans between 90 and 180 meters produced lower tension values in rods due to live load. However, the arrival of higher spans bridges beaten by slender decks produced significant tension variations due to live load. Therefore cables allowable stress is decreased by employing in some cases values lower than the limit 0.45*fpu* normally used in cable-stayed bridges, such as Kanisawa Bridge (Japan, 1998; Kikuchi & Tabata, 1998).

Loads

In the design of extradosed bridges and also for other bridge type , generally the load combinations and loading patterns including traffic loading, lane loading, superimposed loading, live load etc. are considered according to AASHTO (American Association of State Highway and Transportation Officials). Similar to cable-stayed bridges, the extradosed bridge is designed with service loads and allowable stresses in the stay cables and tendons. In the final stages of design, the capacity of the sections is verified at the ultimate limit state (ULS).

Temperature

There are three temperature effects that cause forces in an extradosed bridge: temperature gradient in the girder, temperature differential between the cables and girder, and a uniform temperature range applied to the entire structure. The effects of temperature gradient and temperature differential on the extradosed bridge become more significant as the stiffness of girder increased and must be considered while a temperature range mainly affects the piers.

Tendon Layout

In Mathivat's (1988) concept of the extradosed bridge, the internal cantilever tendons of the box girder are replaced by the extradosed tendons. The only tendons that are housed within the cross-section are external tendons, draped between pier diaphragms and deviators in the span, which is reasonable for precast construction. However, in many extradosed bridges constructed to date, internal cantilever and continuity tendons have been incorporated into the designs.

Extradosed Tendons

Extradosed tendons are anchored in the deck segments and are either anchored in the towers or deviated through the towers by means of saddles. They resist dead load during cantilevering, and resist dead and live load in the final condition.

MODELING AND ANALYSIS

This section presents comprehensive modelling and analysis of Pre-stressed Post-tensioned Extradosed Bridge including apply of loads, design methodology, extra dosed bridge proportions, stay cable technology, girder cross-sections, cable and tendon layout, erection and analysis. The purpose of this analysis is to explore the structural behaviour of an extradosed bridge, and determine what should be considered for its design. The Third Karnaphuli Bridge located on the Chittagong Cox's Bazar (N-1) Highway, Bangladesh is the only one extradosed bridge that has been constructed in Bangladesh. It is a newly adapted type in the context of bridge engineering technology in Bangladesh. We have considered this bridge as the basis of our modelling and analysis (see Figure 1) and all the data that we have used are quite same as The Third Karnaphuli Bridge and compared with two variation of this bridge such as mid span length and deck section variation. All the design criteria and specifications are followed according to AASHTO LRFD BRIDGE DESIGN SPECIFICATION, SI Units, 4th Edition, 2007. For the modelling and analysis we have considered to use CSI BRIDGE v 15.0.0 integrated 3d bridge design software from the advancement of SAP 2000.

Alternatives

Case (I): Deck Section variation of the Karnaphuli Bridge

The mid span depth of the third Karnaphuli Bridge was 4m and section depth at pier location was 6.75m. For comparison we have used mid span depth 3.75m and section depth at pier location 6.5m. All the other parameters have kept constant. The number of external cables required to keep the box stress, shear and flexure within the AASHTO LRFD limiting range is determined.

Case (II): Centre Span Length variation of the Karnaphuli Bridge

The centre span of the original Karnaphuli Bridge was 200m. In this case, we have varied the centre span to 225m and determined the number of extra dosed cables required to balance the additional mass due to the extension of the span length. The end span has kept constant to 115m as Karnaphuli Bridge. Comparison of the results from analysis is shown in Table 1, Table 2 and Table 3.

Table 1 : Shear Comparison at Critical Sections

Station	**KARNAPHULI BRIDGE**		**CASE 1**		**CASE 2**	
m	V2	DC Ratio	V2	DC Ratio	V2	DC Ratio
	KN	Unitless	KN	Unitless	KN	Unitless
0	-35893.3	0.0513	-36038.4	0.057511	-22759.905	0.124973
57.5	19383.42	0.722451	18527.06	0.742426	7527.233	0.201589
115	-70634.8	0.464412	-68927.8	0.479876	-65186.587	0.428596
215	-1375.198	0.069571	683.01	0.065518	-16429.298	0.498356
315	74728.605	0.486357	-57746.5	0.397186	24461.479	0.290932
372.5	-20228.42	0.794987	-17540	0.734179	-5859.192	0.050465
430	34845.67	0.170159	32070.89	0.147234	-11798.184	0.584584

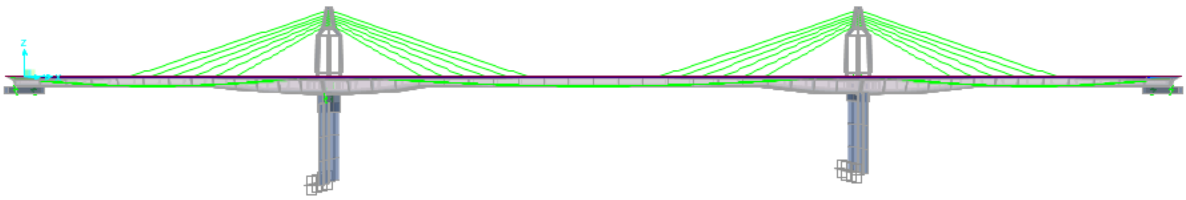

Figure 1: 2D View of the Completed Extradosed Bridge Model

Comparision of Results among Third Karnaphuli Bridge and other two Alternative Cases

Table 2 : Box Stress comparison at Critical Sections

		KARNAPHULI	**CASE 1**	**CASE 2**		
		BRIDGE				
Station	Point	Stress Max	Stress Max	Stress Max	Tens. Limit	Comp. Limit
mm		N/mm2	N/mm2	N/mm2	N/mm2	N/mm2
0	Top Right	-11.718	-11.958	-12.001	8.944	-72
0	Bottom Center	3.589	3.787	3.677	8.944	-72
57.5	Top Center	-0.874	0.46	2.031	8.944	-72
57.5	Bottom Right	-28.281	-29.58	-38.632	8.944	-72
115	Top Right	1.998	1.955	-1.683	8.944	-72
115	Bottom Center	-15.691	-15.93	-13.034	8.944	-72
215	Top Right	-32.888	-33.758	-29.178	8.944	-72
215	Bottom Left	8.545	8.913	6.263	8.944	-72
315	Top Right	3.2	3.172	-5.213	8.944	-72
315	Bottom Left	-15.73	-15.965	-8.59	8.944	-72
372.5	Top Left	3.167	4.604	6.376	8.944	-72
372.5	Bottom Right	-23.583	-24.585	-34.062	8.944	-72
430	Top Right	-7.773	-7.921	-7.632	8.944	-72
430	Bottom Left	2.387	2.516	0.509	8.944	-72

Table 3 : Moment Comparison at Critical sections

Distance	**Item Type**	**V2**	**M3**	**V2**	**M3**	**Distance**	**V2**	**M3**
m		KN	KN-m	KN	KN-m	m	KN	KN-m
0	Max	-1588.639	351859.3844	-1733.918	322154.9	0	10019.26	360502.5
0	Min	-4051.945	351810.678	-4189.812	322102.1	0	7504.375	360495.2
57.5	Max	16495.616	-321807.81	15627.139	-318218	57.5	12333.16	-487416
57.5	Min	14489.237	-382424.4	13622.526	-376178	57.5	10326.39	-552733
115	Max	62212.085	-735293.92	60416.319	-672822	115	53778.73	-378505
115	Min	59972.653	-779594.49	58182.128	-717447	115	51509.34	-430306
215	Max	607.288	530822.2357	606.478	502408.4	227.5	355.96	505189.1
215	Min	-1465.533	477561.7332	-1464.425	451139.6	227.5	-1730.26	444258.4

315	Max	74382.175	-939644.01	72695.58	-889796	340	68878.48	-620505
315	Min	72085.677	-986505.4	70409.12	-934892	340	66577	-679102
372.5	Max	-12024.47	-271552.326	-11317.759	-277559	397.5	-7421.91	-437992
372.5	Min	-14057.89	-339747.13	-13349.66	-344244	397.5	-9454.56	-513218
430	Max	13553.622	233434.7888	13453.193	213438.1	455	2249.646	237292.4
430	Min	10909.625	233387.9783	10808.622	213394.9	455	-474.902	237285.1

RESULTS AND DISCUSSION

In this section, we have done structural analysis of The Third Karnaphuli Bridge 430m long Pre-stressed Post-tensioned Extradosed Bridge consisting of 2, 115m side spans and 200m centre span using CSI BRIDGE v 15.0.0 integrated 3D bridge design software. We have varied different aspects of that bridge model such as box section variation and centre span length variation to make comparison with the original bridge. The analysis is mainly concerned about the determination of number of extradosed cables, layout of cables, cable inclination, number of strands used in the cables, diameter of tendons, checking of box stresses, shear and flexure according to AASHTO LRFD BRIDGE DESIGN SPECIFICATION, SI Units,4th Edition, 2007. In case of Extradosed bridges the box stresses, box shear and flexure checks largely depends on the layout of cables, no. of cables, cable inclination, cable variation along deck section and cable force. Here, we have analysed three different bridge models with variable cable configuration and the results regarding how the variation affects the box stresses, box shear and flexure are shown in Table 4.

Table 4 : Final Summary of Extradosed Cables

Case	**End Span Length (m)**	**Mid Span Length (m)**	**Total Length (m)**	**Deck Section Depth (m)**	**No. Of Extradosed Cables**	**Cables No.**	**No. Of Strand**	**Strand Diameter (mm)**	**Cables Inclination (degree)**	**Minimum Breaking Strength of Strand (fpu)**	**Maximum Cable Stress (MPa)**
Actual						1	91	15.7	29.57	1862	0.6 fpu
Karnaphul						2	91	15.7	25.89	1862	0.6 fpu
i	2@115	200	430	4	24	3	91	15.7	23.23	1862	0.6 fpu
Bridge						4	91	15.7	21.22	1862	0.6 fpu
						5	91	15.7	19.64	1862	0.6 fpu
						6	91	15.7	18.38	1862	0.6 fpu
						1	91	15.7	29.57	1862	0.6 fpu
						2	91	15.7	25.89	1862	0.6 fpu
						3	91	15.7	23.23	1862	0.6 fpu
	2@115	200	430	3.75	26	4	91	15.7	21.22	1862	0.6 fpu

Case-1						5	91	15.7	19.64	1862	0.6 fpu
						6	91	15.7	18.38	1862	0.6 fpu
						7	91	15.7	17.21	1862	0.6 fpu
						1	11	15.7	29.57	1862	0.6 fpu
						2	11	15.7	25.89	1862	0.6 fpu
						3	11	15.7	23.23	1862	0.6 fpu
	2@11	225	455	4	32	4	11	15.7	21.22	1862	0.6 fpu
Case-2						5	11	15.7	19.64	1862	0.6 fpu
						6	11	15.7	18.38	1862	0.6 fpu
						7	11	15.7	17.21	1862	0.6 fpu
						8	11	15.7	16.2	1862	0.6 fpu

CONCLUSION

In the Case 1 study the total depth of the box section of the Karnaphuli Bridge has been varied from 4m to 3.75m at the mid span location and 6.75m to 6.5m at the pier location. From analysis it has been seen that the box stress at critical sections of the deck section has increased considerably. For this reason we have used a total number of 26 extradosed cables. 13 cables are distributed on the both side of the tower. Cables are 109H15 stays consist of 91 nos. 15.7mm diameter strand with weight of 127.53 kg/m. The minimum breaking strength of the cables is 1862Mpa (Grade270). The strands are stressed up to 0.7 times of the minimum breaking strength. From the analysis, it has been found that box stress, shear and flexure are within the **AASHTO LRFD** limiting range.

In the Case 2 study the centre span has been increased from 200m to 225m. The number of cables and cable layout of the preceding model were not sufficient to balance the extra dead load. So we have used a total number of 32 extradosed cables. 16 cables are equally distributed on the both side of the tower. Cables are 109H15 stays consist of 115 nos. 15.7mm diameter strand with weight of 127.53 kg/m. The minimum breaking strength of the cables is 1862Mpa (Grade270). The strands are stressed up to 0.7 times of the minimum breaking strength. From the analysis, it has been found that box stress, shear and flexure are within the **AASHTO LRFD** limiting range.

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EFFECTS OF HEAT TREATMENT AND ALLOYING ELEMENT (CR) ON MICROSTRUCTURES AND MECHANICAL PROPERTIES OF LOW CARBON STRUCTURAL STEEL

Md. Mehedi Hasan[1*], M.A.K. Azad[1], Md. Hamidur Rahman[1], S. I. Hossain[1] & Md. T. Islam[1]

[1] *Department of Materials and Metallurgical Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
**Corresponding Author, <mehedimme07@yahoo.com>*

ABSTRACT

Now-a-days, structural steel plays a pioneering role for the modern civilization. Like other material the structural performance of steel structures is dominated by the mechanical properties of steel. The heat treatment is widely used to improve the mechanical properties of steel. Basically the mechanical properties of steel depend on microstructure especially on grain size which can be controlled by adding alloying elements, heating and cooling. In this study, austenitic grain coarsening behaviors of carburized low carbon steel (Steel l) and low carbon steel with 0.78% Cr (Steel 2) have been investigated at three temperatures such as 950 0C, 1000 0C, 1050 0C. Grain size of carburized Steel l and Steel 2 at 950 0C, 1000 0C, 1050 0C have been found 61, 100, 114 µm and 44, 69, 81 µm, respectively. Through two heat treated process like annealing and normalizing, steel l's Yield Strength (YS) , Ultimate Tensile Strength (UTS), percentage of elongation and percentage of reduction in area have been found 263MPa, 458 MPa, 30, 65 and 280 MPa, 489 MPa, 25 , 68 and of steel 2, 311 MPa, 474M Pa, 26 , 70 and 446 MPa, 673 MPa, 17 , 51, respectively. The result shows that austenite grain size increases with increasing temperature and undissolved particles of CrC refines austenite grain size during heating and improved YS, UTS and ductility which are very attractive properties for structural use.

Keywords: Carburization, Heat treatment, Microstructure, Grain size, Mechanical properties.

INTRODUCTION

The development of high-strength low alloy steels (HSLA) included studies that considered the relationship between mechanical properties and microstructures. Steel has many practical applications in every aspects of life. The early steels were based on ferrite pearlite structure. During the last fifty years, engineers have demanded steels with higher tensile strength, together with adequate ductility and toughness. An increase in carbon content met this demand in a limited way. It increases tensile strength although sacrifices ductility and impact strength even in the heat-treated condition. Alloy addition increases the yield strength with increasing lesser amount of ductility and impact strength. Heat treated alloy steels provide high strength, high yield point, combined with appreciable ductility even in large sections. For resisting corrosion and oxidation at elevated temperatures, alloy steels are essential. The *Alloy Steels Research Committee* adopted the following definition: "Carbon steels are regarded as steels containing not more than 0.5% manganese and 0.5% silicon, all other steels being regarded as alloy steels" (American Society for Metals 1964)[1]. The principal alloying elements added to steel in widely varying amounts either singly or in complex mixtures are nickel, chromium, manganese, molybdenum, vanadium, niobium, silicon and cobalt. The observation in this paper is directed towards experiment on plain carbon steel (0.15 wt% C) and low alloy steel containing small amount of Cr which has a high weldability and toughness. The purpose of the research is to:

1. Observe the effect of alloying element on low carbon steel.

2. Visualize the effectiveness of inhibiting prior austenite grain coarsening behavior by alloy addition.
3. Heat treatment of low alloy steels at a certain cooling rate and correlate microstructures and mechanical properties of heat treated steels i.e. annealed and normalized low alloy steels.

MATERIALS AND METHODS

1. GRAIN COARSENING EXPERIMENT WITH CARBURIZATION TECHNIQUE

1.1. Specimen preparation

Two types of steel were studied where Steel 1 contains about 0.15% carbon and steel 2 contains about 0.14% carbon and 0.78% Cr. The composition of these steels are presented in Table 1. Steel 1 is plain carbon steel and steel 2 is low alloy steel containing small amount of chromium. Steel 1 is the base steel with which the structure and properties of steel 2 is compared. About 16 mm diameter and 10 mm thick specimens were machined from the bars of each of the steels in order to study the austenite grain coarsening behaviour.

1.2. Carburization and measurement of austenite grain size

Since size of the austenite grains directly affect the subsequent structure and hence the properties of steels, a study was made to determine prior austenite grain size at temperatures higher than upper critical temperature. Carburization technique was used to reveal prior austenite grain size. There are also other methods in determining prior austenite grain size like isothermal transformation technique, oxidation technique etc. But previous work showed that the isothermal technique did not work well in revealing prior austenite grain boundary of low alloy steels (Haque 1989)[2]**.** So, carburization technique was adopted to reveal prior austenite grain boundaries of steels in this work.

Table 1. Chemical Composition of Steel 1 and 2

Chemical compound	Weight percentage of Steel 1 (Plain carbon steel)	Weight percentage of Steel 2 (Cr steel)
Carbon (c)	0.15	0.14
Silicon (Si)	0.12	0.16
Manganese (Mn)	1.14	1.13
Sulphur (S)	0.024	0.018
Phosphorus (P)	0.007	0.013
Chromium (Cr)	-	0.78

The technique is based on the formation of a continuous cementite network at the austenite grain boundaries. Carbon diffuses in steel from the carburizing atmosphere forming hypereutectoid steel at the surface of the specimen during slow cooling in the furnace, continuous cementite network is formed at the austenite grain boundaries at the selected austenitizing temperatures. Subsequent etching of the furnace cooled samples revealed the cementite network formed which marked the prior austenite grain size at the selected carburizing temperatures (Clark and Varney 1962)[3]. Solid carburizing or pack carburizing technique was applied for this experiment. The steel specimens were

heated to three different austenizing temperatures such as 950°C, 1000°C and 1050°C. Before heating these specimens, they were packed in a box with carburizing mixture. Then they were placed in Blue M furnace and the furnace was switched on. After reaching the desired austenizing temperature, the specimen were held at that temperature for 90 minutes and then cooled in furnace to room temperature. The assessment of prior austenite grain size was made from direct measurement of the austenite grains in the specimens under optical microscope. The grain size was measured using the mean linear intercept method counting grain boundary intersections with the circumference of the circle in the eye piece of a microscope. The effective circumference of the circle was determined precisely by measuring its diameters with reference to a stage micrometer at the magnification used. A total at least 50 – 60 intersections were counted for each specimen. Prior austenite grain size at different austenitizing temperatures for steels 1 and 2 are shown in fig. 1.

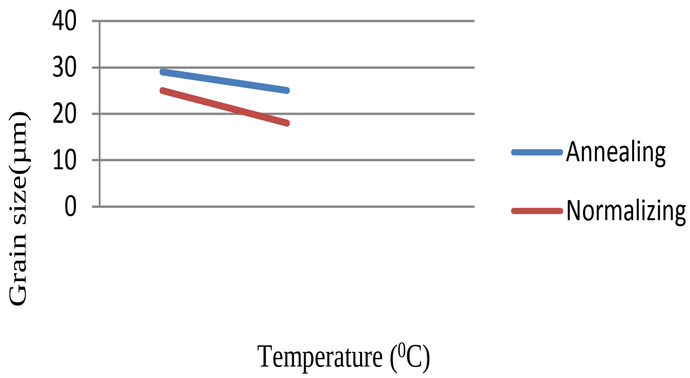


Fig.1. Variation of prior austenite grain size with temperatures of steel 1 (Plain carbon steel) and steel 2 (Cr steel).

2. HEAT TREATMENT

130 mm long specimens were cut from 16 mm diameter rod. All the specimens were annealed at 900°C for 4 hours and then cooled at furnace for obtaining homogeneous structure. A cover with a mixture of sand and coke powder was used to prevent decarburization of the specimens. After homogenization, some part of the specimen were machined out to form specimen like cylindrical shape. Then the sample was placed inside the furnace. The specimen was held at the heat treatment temperature of 910°C for plain carbon steel and 980°C for Cr steel. Holding time for both steels is 30 minutes. At the end of the holding period, the specimen was cooled in furnace which process is known as annealing and cooled in air which process is known as normalizing.

3. MECHANICAL TESTING

The heat treated 16 mm diameter bar was then machined into standard tensile specimens. The minimum diameter of the tensile specimen was about 4.00 mm and gauge length was 25 mm. The tensile specimens were then tested with a Universal Tensile Testing Machine (INSTRON). Yield strength (YS), ultimate tensile strength (UTS), percentage of elongation (% EL) and percentage of reduction in area (% RA) were measured. Dimensions of tensile specimens and location of the section for metallographic sample are shown in fig. 2. 8-10 mm small samples from the side where the sample was gripped during tensile test were cut from the fractured tensile specimens for micro examination. The samples were then mounted, polished and then etched in 2% nital. The microstructure of these samples were studied with the help of an optical microscope and photograph of these structures of each specimen were taken with 200X magnification. The ferrite grain size of steel 1 and steel 2 (annealing) were measured by 200X magnification. The ferrite grain size of steel 1 and steel 2 (normalizing) were measured by 400X magnification for better result [4]. After that the volume fraction of pearlite was observed.

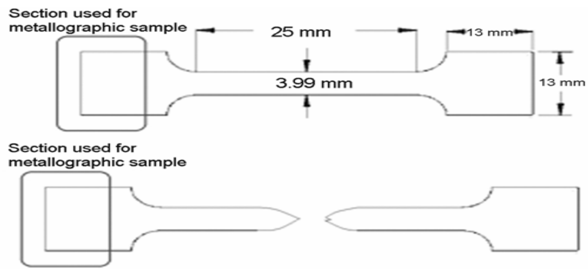
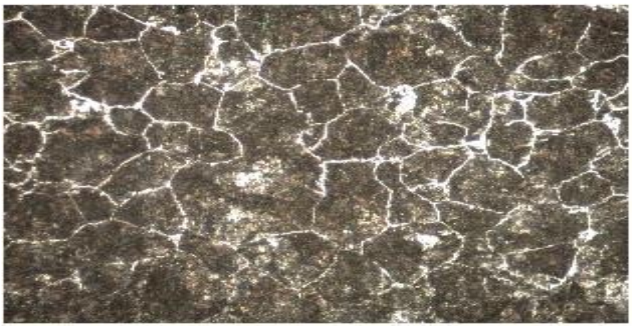


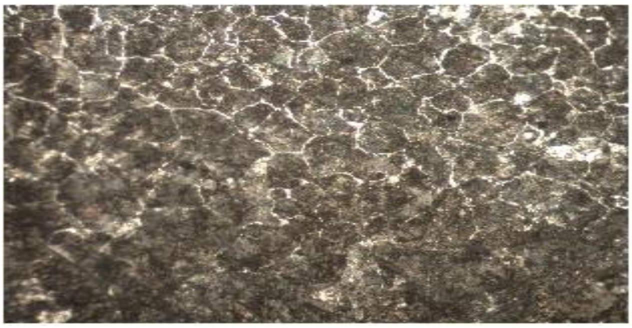
Fig. 2. Dimensions of tensile specimen and location of the section for metallographic sample.

RESULT AND DISCUSSION

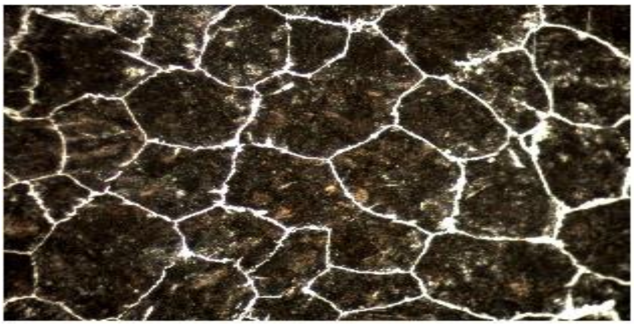
The prior austenite grain size of the steels 1 and 2 were revealed by carburization method. The austenite grain was measured by mean linear intercept method. The optical micrographs of the variations of austenite grain size of two different steels at three different temperatures (950^{O}C, 1000^{O}C and 1050^{O}C) are shown in Fig.3.



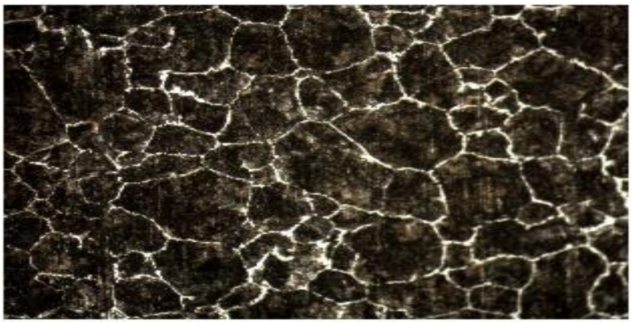
Plain carbon steel at 950^{O}C



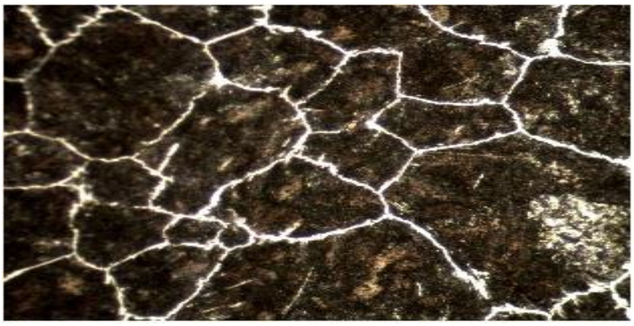
Cr steel at 950^{O}C

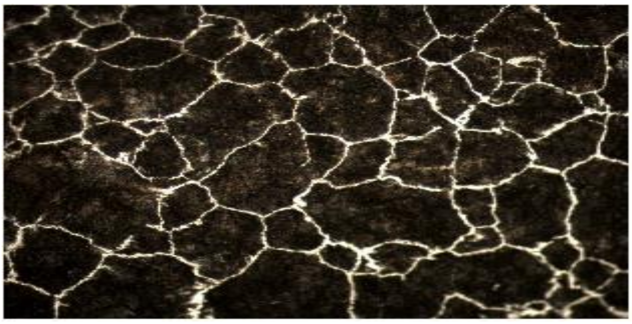


Plain carbon steel at 1000^{O}C



Cr steel at 1000^{O}C





Plain carbon steel at $1050^{O}C$ Cr steel at $1050^{O}C$

Fig. 3. Optical micrograph showing the variations of austenite grain size in plain carbon steel and Cr steel at $950^{O}C$, $1000^{O}C$ and $1050^{O}C$,200X.

It can be observed that austenite grain size increases with increasing temperature in both plain carbon steel and Cr steel. The grain size of Cr steel is smaller than the grain size of plain carbon steel at the same temperature. Cr combines with carbon and forms chromium carbide precipitates (Aver 1974)[5].

Chromium carbide particles pin the austenite grain boundaries and restrict the grain growth of austenite. For this reason, the grain of Cr steel is much finer than plain carbon steel and the rate of increasing grain size of plain carbon steel is steeper than alloy steel. The microstructure of plain carbon steel and Cr steel of annealing and normalizing process are shown in figures 4 and 5 respectively.

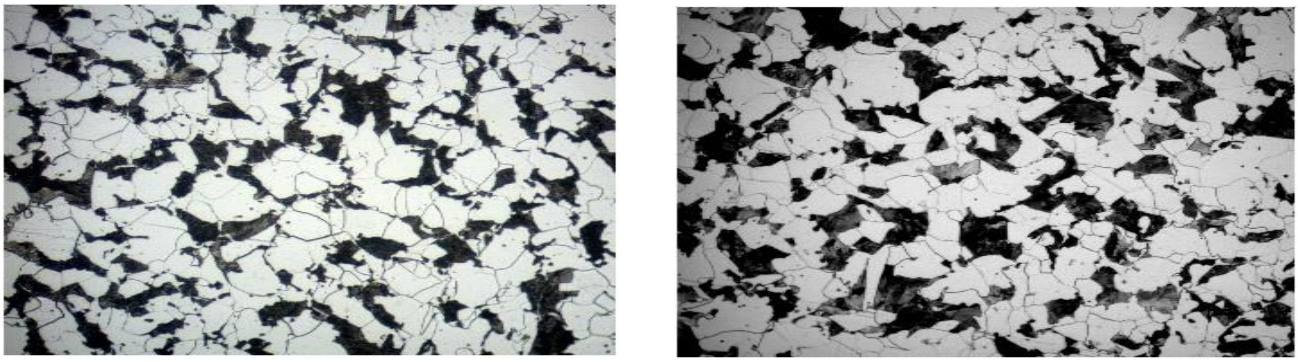


Fig. 4. Optical micrograph of plain carbon steel (Left) and Cr steel (Right) [Annealing], 200X.

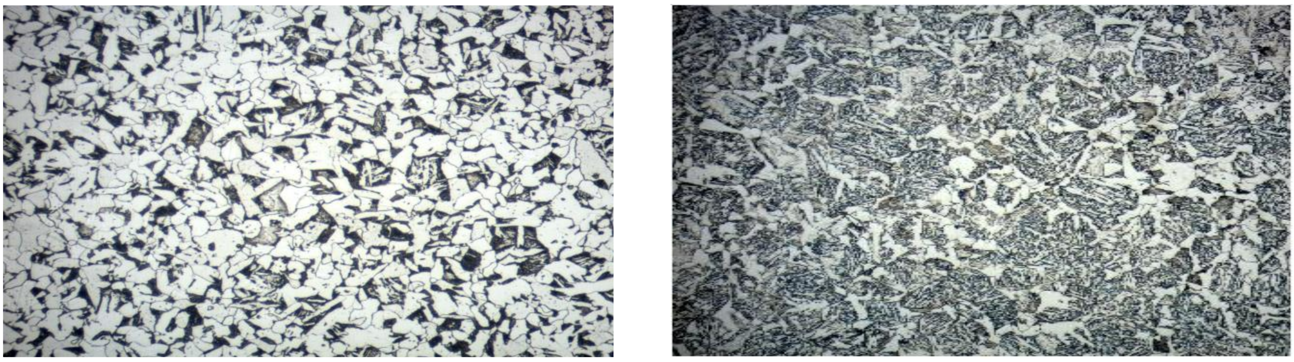


Fig. 5. Optical micrograph of plain carbon steel (Left) and Cr steel (Right) [Normalizing], 400X.

The microstructure of annealed plain carbon steel is coarse ferrite pearlite and fine ferrite pearlite in Cr steel. The microstructure of normalized plain carbon steel is fine ferrite pearlite with few amount of Widmanstatten structure. The microstructure of normalized Cr steel is fine ferrite pearlite along with higher amount of Widmanstatten structure. The ferrite grain size of plain carbon steel is coarser than Cr steel. The ferrite grain size of normalized plain carbon steel and Cr steel are finer than annealed steel. So, the grain size decreases with increasing cooling rate. Plain carbon steel does not contain any alloying element. Cr steel contains chromium which forms CrC precipitates. The finer grain size in Cr steel is due to the presence of CrC precipitate. Yield strength (YS) of plain carbon steel is smaller than the yield strength (YS) of Cr steel at both cooling rate. On the other hand, yield strength (YS) of normalized plain carbon steel and Cr steel is greater than annealed steel 1 and 2. It indicates that the yield strength is increased with alloy addition and increasing cooling rate. Ultimate tensile strength (UTS) of Cr steel is higher than the ultimate tensile strength (UTS) of plain carbon steel at the both cooling rate. The ultimate tensile strength of normalized plain carbon steel and Cr steel is greater than annealed steel 1 and 2. It indicates that the ultimate tensile strength is increased with alloy addition and increasing cooling rate. Cr steel has less percentage of elongation at both heat treated process than plain carbon steel. The percentage of elongation of normalized plain carbon steel and Cr steel is less than annealed plain carbon steel and Cr steel. It indicates that the percentage of elongation is decreased with alloy addition and increasing cooling rate. Percentage reduction in area is not smooth like percentage of elongation and ultimate tensile strength. In Cr steel, percentage reduction in area is decreased with increasing cooling rate. Variations of percentage of elongation (% EL) and yield strength (YS) are illustrated in fig.6.

Fig. 6. Comparison of (a) yield strength (YS) and (b) percentage of elongation (% EL) of steel 1 and 2.

CONCLUSION

The carburization method is satisfactory for revealing prior austenitic grain boundaries. The presence of Cr in steel 2 inhibits the further grain growth with increasing temperature by pinning effect. On the other hand, the curve of grain growth of plain carbon steel is approximately linear with increasing temperature and steeper than Cr steel. Normalizing treatment gives better result than annealing in terms of yield stress, ultimate tensile stress, percentage of elongation and finer grain size. Steel 2 shows satisfactory improvement than steel 1 with heat treatment and presence of alloying element.

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ID: SEE 058

COMPARISON OF ENGINEERING PROPERTIES OF COARSE AGGREGATES COLLECTED FROM DIFFERENT SOURCES IN BANGLADESH

Jahangir Alam[1]*, Md. Saiful Islam[1] & A.H.M. Golam Hyder[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi –6204, Bangladesh.*
**Corresponding Author, <mja31.ruet@gmail.coml>*

ABSTRACT

Concrete is one of the most widely used construction materials throughout the world. Because of the spatial variations in the properties of locally available aggregates, the properties of concrete may vary widely. These properties of aggregate vary a lot based on sources of aggregates. The specific objectives of the study are to know the properties and specifications of coarse aggregates collected from different sources in Bangladesh and to observe their spatial variation on physical properties of aggregates. Coarse aggregates have been collected from different sources, such as Sylhet, Panchagarh and Dinajpur. According to ASTM and BS specifications sieve analysis, specific gravity (OD and SSD) conditions, unit weight (compacted), aggregate impact value (AIV), aggregate crushing value (ACV), ten percent fines value (TFV), flakiness index, elongation index and Los Angles abrasion value (LAAV) test coarse aggregates have been performed in the laboratory. Considering specific gravity and unit weight of coarse aggregates, samples from Boropukuria in Dinajpur have the higher values in comparisons with other sources. On the other hand, these samples have lower aggregate crushing value and Los Angles abrasion value. But, in specific gravity for coarse aggregates, oven dry (OD) and saturated surface dry (SSD) conditions, the samples from Boropukuria in Dinajpur give higher values in compared with the other sources. On the basis of the spatial variation of aggregate properties, it has been suggested that the construction companies in Bangladesh should get a clear and definite idea of aggregates from different sources in Bangladesh and should select the best aggregate to ensure the quality and strength of concrete.

Keywords: Properties, Coarse Aggregates, Concrete, Spatial variation, Construction materials.

INTRODUCTION

Bangladesh is a developing country. Development program of a country like Bangladesh has a great emphasis on infrastructure. Construction sector is one of the prominent sectors to contribute in gross domestic product (GDP) in Bangladesh. Aggregate is mostly used as construction material in almost every construction work in Bangladesh. Aggregates are collected from different sources. In those places the rocks from which the aggregate is processed are not the same. Moreover the geological and geo-morphological processes under which the aggregate forming rocks pass, may vary in different locations. For this reason, remarkable variations on the properties of aggregate are observed according to various sources. Weather condition of the source area is one of them. Temperature, humidity and rainfall etc. varies in different seasons. These variations are very significant with the change of seasons in Bangladesh. Generally in the construction works of Bangladesh, spatial variation of aggregate is not taken into consideration. To achieve required dimensional stability, durability and strength of structures, aggregate characteristics and related engineering properties is one of the main issues needed to be addressed.

MATERIALS AND METHODS

Coarse aggregate has been collected from different sources (Figure 3.2) in Bangladesh like Zaflong at Gowainghat and Volaganj at Companiganj in Shylhet; Vozonpur at Tentulia in Panchagarh; Boropukuria at Fulbari in Dinajpur in same year. The amount of aggregate required for the test according to ASTM and BS specification was collected from the source. After collection of sample, physical characteristics have been identified and the following observations were made as shown in Table 4.1Coarse aggregates collected from different sources of Bangladesh have shown spatial variation while performing Sieve Analysis Test, Specific Gravity and Absorption Capacity test (OVD and SSD), Unit Weight and Voids (compacted), Aggregate Impact Value (AIV), Aggregate Crushing Value (ACV), Ten Percent Fines Value (TFV), Los Angles Abrasion Value (LAAV), Flakiness Index (FI) and Elongation Index (EI) test. All the standard specifications of the tests have been maintained during in the laboratory. Results of those tests are shown in tables 1. Tests results are comparing with the standard value of ACI, BS, IS, PWD (Public works department, Bangladesh) and AASHTO

(a) Zaflong in Sylhet

(b) Volaganj in Sylhe

(c) Vozonpur in Panchagarh

(d) Boropukuria in Dinajpur

Fig.1: Coarse Aggregate of Different Sources in Bangladesh.

Table 1: Physical observation of coarse aggregate

Source	Colour	Size	Shape
Zaflong in Sylhet	Gray to chockolet	Uniformly	Almost regular
Volaganj in Sylhet	Brown to gray	Mixed	Mainly regular

Vozonpur in Panchagarh	Brown and mixed	Comparatively large	Regular
Boropukuria in Dinajpur	Dark to black	Uniformly	Almost regular

Table 2: Qualitative Analysis of Engineering Properties

Properties	Test Result				ACI/AASHTO standard value
	Zaflong in Sylhet	Volaganj in Sylhet	Vozonpur in Panchagarh	Boropukuria in Dinajpur	
Specific Gravity	2.57	2.69	2.50	2.79	2.3-2.9
Absorption Capacity (%)	1.4	1.32	1.93	0.95	0.5-4.0
Unit Weight(kg/cum)	1645	1695	1674	1732	2250
Aggregate Impact Value (AIV, %)	13.49	12.48	13.86	10.50	<30
Aggregate Crushing Value(ACV,	18.72	17.50	18.53	15.06	<30
Ten Percent Fines Value (TFV, %)	13.86	14.14	13.93	14.0	<30
Flakiness Index (FI, %)	18.95	18.55	18.45	17.95	<30
Elongation Index (EI, %)	26.20	25.0	28.75	24.0	<45
Los Angles Abrasion Value (LAAV, %)	29.0	28.30	28.50	26.40	<40
Fineness Modulus(FM)	6.19	6.19	6.22	6.19	6.5-8.0

RESULTS AND DISCUSSIONS

Spatial variations of aggregate properties have been identified interpreting the test results with graphical representations. Standard values of ACI, BS, AASHTO, IS and PWD have been also compared with test results to find the quality of aggregate. Boropukuria in Dinajpur and Volaganj in Sylhet have been found as the best coarse aggregate respectively with the comparison of other sources in Bangladesh. On the basis of the whole research works brief conclusion and some major recommendations is going to be represented in the upcoming chapter.

CONCLUSION

The paper presents a conceptual framework which puts forwarded a vision for aggregates characterization. It integrates three major facets of characterization viz. the aggregate source, the crushing plant and the basic properties obtained through physical and engineering testing. The above three facets are linked with application side where the standards come in place to define suitability of the aggregate for a given application such as concrete, asphalt etc. The output of the characterization is an aggregate resource map which can provide geographically referenced database for ready utilization. It may be concluded that the study would provide results which would be highly beneficial to the construction industry in Bangladesh. This research work represent in depth analysis of aggregate properties from different sources in Bangladesh. From this study the variation of aggregate properties due to spatial variation have been observed which gives a clear idea about the quality of aggregates of that particular place. On the basis of physical observation test results and comparison with the standard value of ACI, BS, AASHTO, IS and PWD following spatial variation of coarse aggregate properties have been found.

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A STUDY OF LIGHTWEIGHT GREEN CONCRTE USING WASTE PLASTIC

M. J. Islam [1*], A. Hasnat[1], A. S. M. Sayem[1] & F. Tousif [1]

[1] *Department of Civil and Environmental Engineering, Islamic University of Technology, Gazipur - 1704, Bangladesh.*
**Corresponding Author, <mjislam@iut-dhaka.edu>*

ABSTRACT

for the conventional coarse aggregate. Polypropylene, a waste polymeric material, is produced nowadays to produce buckets, toys, etc. However, it is imperative to identify an alternative procedure for recycling them since they are non-biodegradable. To this end, recycled polypropylenes are melted at 170 0C and then cooled within molds to produce high density plastic coarse aggregate. Waste plastics as a partial replacement (10%, by volume) of conventional coarse aggregate, such as brick chips have been used in this study. Water-cement ratio of 0.48 has been selected to observe the performance of concrete mixture. The selected mix proportion is selected to be 1:1.5:3 (Cement: Fine aggregate: Coarse aggregate) by volume. The compressive and tensile strengths of concrete at 28 days curing period has been investigated. Findings from the experiments have shown that the incorporation of plastic does not results in significant reduction in compressive strength compared to the natural aggregate concretes. Furthermore, it produces lower density concrete which can be used for structural purposes.

Keywords: Polypropylene, lightweight concrete, compressive strength, tensile splitting strength.

INTRODUCTION

Light Weight Concrete (LWC) can be produced by making concrete lighter by incorporating relatively lighter aggregate in place of conventional coarse aggregate (i.e. stone chips, brick chips etc.). Light weight aggregate is basically of three types. The first type is natural ones (i.e. volcanic pumice), the second type is industrial by-products (i.e. fly ash, Lytag, foamed slag, sintered pulverized fuel ash) and the third type is artificial ones like treated with temperature, clay, shale or slate. The LWC contributes in reducing self-weight of load bearing members, such as beams, columns, slabs, foundations etc.

The Structural LWC has densities ranging from 1360 to 1920 kg/m^3 and minimum compressive strengths of 17.0 MPa. On the other hand, low-density concretes, whose density seldom exceeds 800 kg/m^3, are used chiefly as insulation. While their thermal insulation values are high, their compressive strengths are low, ranging from approximately 0.7 to 7.0 MPa (ACI Education Bulletin E1-07). In the Bangladesh National Building Code (BNBC) specification of minimum load bearing strength is 17 MPa.

Polypropylene (PP) is form of plastic, drawn up from different plastic products like bucket, jar, toys, technological apparatus etc. These products are not bio degradable, and thus, possess threats to the surrounding environment. Recycling of these plastics offer sustainable waste disposal and management with a goal towards pollution free environment. In our study, PP has been used as the replacement of coarse aggregate in concrete. PP can be used in shredded form after melting & cooling, to provide a rough surface for proper bonding between the mortar and aggregates.

Marzouk et al. **(2006)** ran research on an innovative use of consumed plastic bottle waste, Polyethylene terephthalate (PET), as sand-substitution aggregate within composite materials. A volumetric percentage of sand was replaced by the same volumetric percentage of recycled aggregates, ranging from 2% to 100%. In a similar study **Ismail and Al-Hashmi (2007)** substituted fine aggregates in concrete with waste plastics. In their research, a concrete mixture made of 20%

waste plastic had 30.5% lower flexural strength, at 28 days curing age, than the reference concrete mixture. At 28 days curing age, the lowest dry density (2223.7 kg/m^3) exceeded the range of the dry density of structural lightweight concrete. The fresh density values of waste plastic concrete mixtures tend to decrease by 5%, 7%, and 8.7% for 10%, 15%, and 20% waste plastic substitution, respectively. The reuse of thermosetting plastic waste was investigated by **Panyakapo et al (2008)**. The ratio of cement, sand, fly ash, and plastic was equal to 1.0:0.8:0.3:0.9 in an appropriate mix proportion. The results of compressive strength and dry density were 4.14 MPa and 1395 kg/m^3, respectively. In an another work **Frigione (2010)** substituted 5% of the weight of total fine aggregate (natural sand) with an equal weight of PET aggregates manufactured from the waste un-washed PET bottles (WPET). The compressive strength was determined to be 40.7 MPa with a water-cement ratio of 0.55; whereas the reference concrete (without plastic aggregates) has the compressive strength of 41.5 MPa.

Sim et al (2013) conducted research on effect of aggregate and specimen sizes on lightweight Concrete. At their research, artificially expanded clay granules were used as the lightweight aggregates. **Rahman et al. (2012)** investigated on the incorporation of waste polymer materials, especially expanded polystyrene (EPS) based packaging materials, to the concrete which makes the material very light weight. Concrete with modified EPS showed lighter properties than that of HDPE and tire modified concrete. The result shows that the inclusion of waste polymer materials decreases compressive strength, density, porosity and water absorption characteristics. The mechanical behaviour of concrete with recycled Polyethylene Terephthalate (PET) has been studied by **Albano et al. (2009).** Portland cement, fine aggregate (river sand), coarse aggregate (crushed stone) and light-weight aggregate (recycled PET) were used in their study with varying water/cement ratio (0.50 and 0.60), PET content (10% and 20% by volume) and the particle size along with the influence of the thermal degradation of PET in the concrete when the blends were exposed to different temperatures (200, 400, 600 °C). Both w/c ratios presented lower compressive strengths. On the other hand, the flexural strength of concrete-PET when exposed to a heat source was strongly dependent on the temperature, water/cement ratio, as well as on the PET content and particle size.

From the literature review, it is evident that, higher the replacement of plastic as coarse aggregate lowers the compressive strength. Furthermore, use of water more than 50% weight of cement lower the compressive strength. Based upon the findings, a method for replacement (10% by volume) of conventional coarse aggregate by PP has been adopted for our study and the water-cement ratio used was 0.48.

MATERIALS

Various materials are used to cast concrete. The materials are chosen based on the application and purpose of the concrete. For the preparation of concrete mixture coarse aggregate, fine aggregate and binding material is the main ingredient. Stone or brick chips are widely used as coarse aggregate; while sand is used as fine aggregate, and cement is the first choice for binding material.

Binding material

In the present study, Portland Composite Cement was used as binding material. It contained 65-79% clinker, 21-35% lime stone, fly ash, blast furnace slag 21–35% and Gypsum 0-5%. Chemical Composition of the used binding material (CEM II / B-M conforming to ASTM C 595 Specification of Portland Composite Cement) is presented in Table 1.

Table 1 Chemical Composition of binding material

Chemical Composition	Unit	Test result
Calcium Oxide (CaO)	(%)	51.63

Silicon dioxide(SiO_2)	(%)	23.79
Aluminium Oxide(Al_2O_3)	(%)	8.36
Ferric Oxide(Fe_2O_3)	(%)	3.41
Sulfur trioxide(SO_3)	(%)	2.24
Magnesium Oxide(MgO)	(%)	1.67
Loss of Ignition(LOI)	(%)	3.17
Insoluble Residue(IR)	(%)	17.30

Coarse aggregates

Brick chips along with PP were used as coarse aggregate. PP was used to partially replace the brick chips as coarse aggregate. Bricks were obtained from local brick fields. They were crushed manually and then washed to remove dust and dart. The used PP was collected from a local manufacturer. The aggregates were produced from waste plastic. The used PP was crushed and sieved properly (Figure 1). To obtain their physical properties several tests were performed according to ASTM standard C127.

Figure 1 PP aggregate

Fine aggregate

Sylhet Sand was used as fine aggregate. The sand was first sieved through 4.75 mm sieve to eliminate particles greater than 4.75mm. Then it was washed to remove the dust. Relevant tests were performed according to ASTM C128 standard to get various properties.

The properties of the aggregates, used in this study, obtained from various tests are tabulated in Table 2.

METHODOLOGY

In order to prosecute the research work, some experiments have been done to analyze the outcome. The goals were to inspect the properties of the materials used and to examine the compressive strength, tensile splitting strength and failure pattern of the sample cylinder. Thus the experimental procedure includes mix design, casting and curing, slump test, density measurement and strength tests.

Table 2 Properties of the aggregates

Characteristics	Value		
	Coarse aggregate (Brick)	Coarse aggregate (PP)	Fine aggregate (Sand)
Maximum size	25mm	25mm	4.75mm
Specific gravity	2.045	0.8452	2.39
Water absorption	20.56%	0.7476%	8.9%
Fineness modulus	2.54	2.4	2.95
Unit Weight	980.295 kg/m^3	506.451 kg/m^3	1632.3 kg/m^3

Mix design

W/C ratio of 0.48 and mix proportion of 1:1.5:3 (Cement: Fine aggregate: Coarse aggregate) by volume has been selected. The resultant mix proportions of all the mixes by weight are tabulated in the Table 3.

Table 3 Mix design

Designation	w/c ratio	Water (kg)	Cement (kg)	Sand (kg)	Brick chips (kg)	PP aggregate (kg)
W48R0	0.48	4.22	6.6	7.55	12.51	0.00
W48R10	0.48	4.22	6.6	7.85	10.14	0.53

Casting and curing

Casting and Curing of concrete cylinder specimen was performed according to ASTM C192. Generally the compressive strength of concrete differs according to the age (i.e. 14, 21 & 28 days). For this project 28 days of curing was considered as standard.

Slump test

Slump test is an empirical test which measures the consistency of fresh concrete. More precisely, it measures the consistency of the concrete of a specific batch. The test was performed according to ASTM C143.

Compressive strength test

This test method covers determination of compressive strength of cylindrical concrete specimens such as molded cylinders which consists of applying a compressive axial load to molded cylinders at a rate which is within a prescribed range until failure occurs. ASTM C 39 standard specification of Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens was followed.

Tensile splitting strength test

This test method covers the determination of the splitting tensile strength of cylindrical concrete specimens, such as molded cylinders. The tests were performed according to ASTM C496 standard specification.

RESULTS & DISCUSSIONS

Data obtained from the experiments have been analyzed to assess the suitability of PP as an alternative lightweight aggregate in load bearing structures. From the findings of the experiments, comparison between concrete with PP (as coarse aggregate) and regular concrete has been made. Densities of the concrete samples were measured at surface dry condition at 28 days just before the compressive strength test. As shown in Figure 2, 7.2% reduction of density is achievable with 10% replacement (by volume) of natural aggregates with PP coarse aggregates.

The compressive strength tests were carried out after 28 days of casting. Figure 3(a) demonstrates the compressive strength test results. As observed from the figure, only 4% reduction of compressive strength was detected for Plastic Coarse Aggregate (PCA) concrete compared to Regular Concrete (RC). The probable reason for decreased compressive strength with the increment of PP aggregate in concrete is the poor bondage between PP aggregate and mortar.

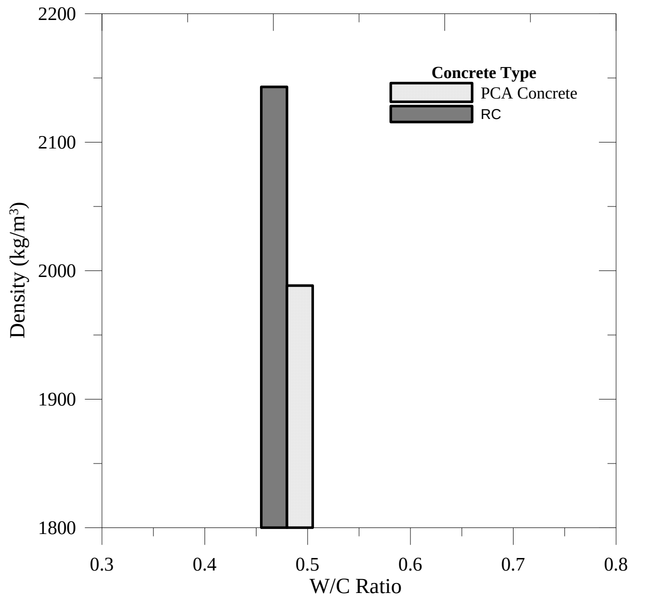
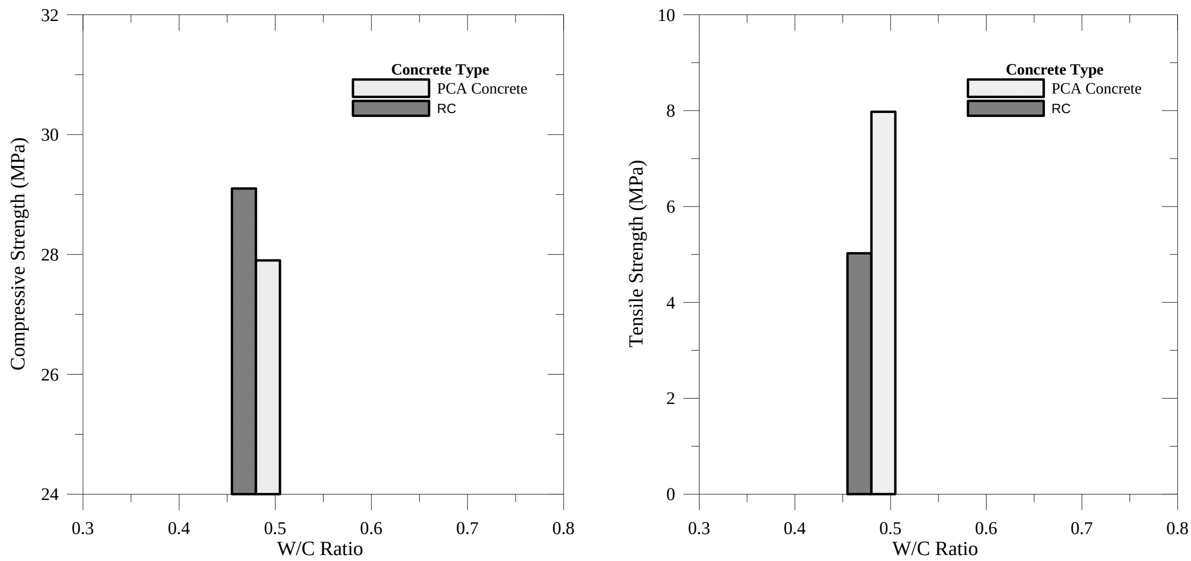


Figure 2 Density of different types of concrete.

The split tensile strength tests were also carried out after 28 days of casting. As illustrated in Figure 3(b), about 59% increase in tensile strength was observed for PCA concrete compared to regular concrete. This increase in tensile strength can be attributed to the better crack resistance of the PCA concrete.



(a) (b)

Figure 3 Comparison of strength of different types of concrete.

Table 4 summarizes the various test results for both regular and PCA concrete. It also lists the strength to density ratio. The PCA concrete has better strength to density ratio (14.05 kPa/ kg/m^3) than that of regular concrete (13.58 kPa/ kg/m^3). Slump test results indicate workability of concrete. From the test results, it was observed that for same w/c ratio, slump value was lower for PCA concrete which indicated lower workability for PCA concrete compared to regular concrete. Failure pattern of the PCA concrete specimens were also monitored. As demonstrated in Figure 4(a), a combination of cone and shear failure was detected during the compressive strength test of the PCA concrete. On the other hand, a vertical plane failure was observed during the split tensile strength test.

Table 4 Various test results for regular concrete and PCA concrete

% of PP	W/C ratio	Average Density (kg/m^3)	Slump (cm)	Average Compressive Strength (MPa)	Average Tensile Strength (MPa)	Strength to Density Ratio (kPa/ kg/m^3)	Failure pattern	
							For compression	For tension
0	0.48	2143	3.0	29.10	5.024	13.58	Cone and Shear	Along vertical plane
10	0.48	1988	0.0	27.94	7.975	14.05	Cone and Shear	Along vertical plane

(a) (b)

Figure 4 (a) Cone and shear failure during compressive strength test; (b) Vertical plane failure during split tensile strength test.

CONCLUSION

The present experimental work deals with the possibility of using the PP as a replacement of conventionally used coarse aggregate to obtain a light weight concrete mixture which can be used in load bearing structures. The experimental results lead to the following conclusions:

1. Density of the PCA concrete was found to be 1988.412 kg/m^3. Since this value is close to the ACI range of lightweight concrete the PCA concrete can be termed as lightweight concrete.
2. The compressive strength, on the other hand, was obtained as 27.94 MPa, which is distinctly higher than the mentioned standard. Henceforth, this lightweight concrete can be used for structural purposes.
3. For 10% PP replacement (by volume), 7.2% of density reduction was achieved; whereas only 4% of compressive strength reduction was monitored compare to the RC.
4. Tensile splitting strength of the PCA concrete is significantly higher (59%) than the regular concrete.

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EFFECTIVENESS OF R.C.C FRAMED SYSTEM WITH BEST ECONOMY UNDER DIFFERENT ARRANGEMENT OF SHEAR WALL LOCATION

S. Khusru[1*], M. U Chowdhury[1], S. Khatoon[1]
T. S. Preoti[1], R. Islam[1] & N. Akhter[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.*
**Corresponding Author, <shovonakhusru@yahoo.com>*

ABSTRACT

Provision of shear walls has become inevitable in reinforced concrete buildings to resist lateral forces. It is very important to determine effectiveness and perfect location of shear walls. In this paper, study of 10 storey building is analysed by different arrangement of shear walls, i.e. structure with shear walls in both directions at inner segments (case 1), in both directions at outer segments (case 2), in short direction at outer segments (case 3) and in long direction at outer segments (case 4) for determining parameters, like storey displacement due to wind and earthquake loads, column moment, etc. The results have shown that displacement is comparatively low in the direction where shear walls are provided and maximum in other direction where shear walls are not provided. But, when shear walls are provided in both directions, the storey displacements are moderately low. Similarly, for external and internal column moments, case 1 and 2 can be taken as the most favourable configuration. For economical analysis, a factor fw-∂ has been developed considering both weight of the structure and displacements. According to this factor, case 1, i.e. the building with shear walls in both directions at inner segment is the most preferable configuration.

Keywords: Reinforced Concrete, Shear wall, lateral loading.

INTRODUCTION

A shear wall structure is considered to one whose resistance to horizontal load is provided entirely by shear wall. The walls are part of a service core, or they serve as partitions between accommodations. They are usually continuous down to the base to which they are rigidly attached to form vertical cantilever. Shear walls are not only designed to resist gravity loads but they are also designed for lateral loads of earthquakes and wind. Several studies have been done on placement of shear wall and on the effect of lateral loads on it. It had been found in a research that Placing Shear wall away from centre of gravity resulted in increase in most of the members forces (Asish et.al. 2012). Also, an attempt has been made to evaluate response of 10 storey framed structure, subjected to lateral loads such as earthquake with different arrangement of Reinforced Concrete (RC) shear wall in structural plan which showed Placement of shear wall did helps a lot in restraint the lateral displacement at tip node (Toloue et.al.2013). The objective of this study are given below-

- It is an objective of this study to compare the effectiveness of structure with different arrangement of shear wall system.
- This study is done to compare different arrangement of shear wall to withstand lateral forces like earthquake and wind effectively.
- Also to determine the optimum arrangement based on serviceability of shear wall among the different arrangements studied.
- Finally a complete economical assessment of the structure will be done for minimum weight and minimum displacement in the structure with shear wall condition, so that

in future use of shear wall system in high rise structure, as a support against lateral load, can be designed more economically and can be proved as prudent.

METHODOLOGY

Here the 10 storey building is compared with the effect of lateral load on different position of shear wall and its economy.

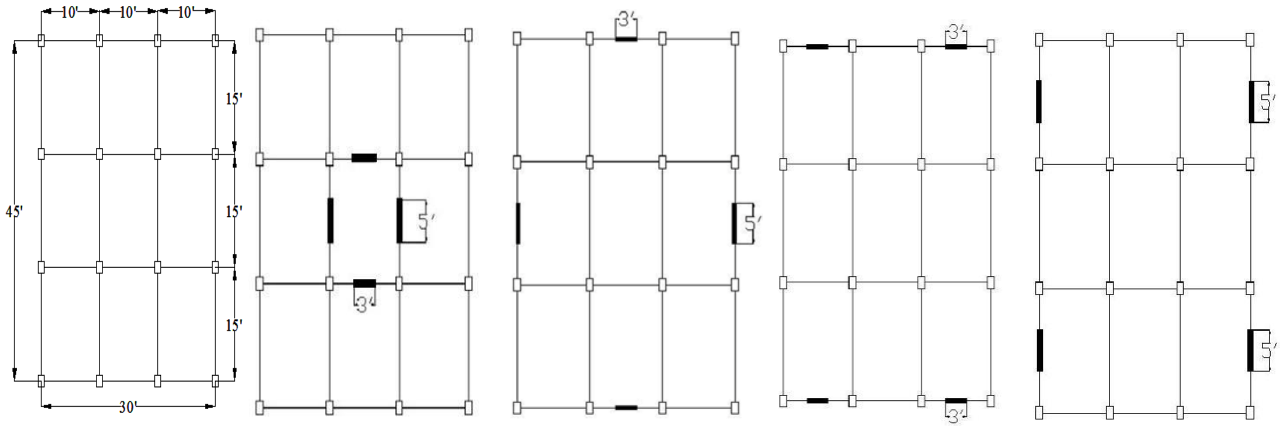
Building Modeling: The plan area of the considered 10 storied building is 1350 sqft (45'×30'). The dimension of each panel in long direction is 15' and in short direction is 10'.

Table 1: Preliminary data

Height of each storey	10 feet
Height of foundation	8 feet
Number of storey	10
Depth of slab	6 inch
Thickness of shear wall	6 inch

Case Study

Shear wall are provided considering approximately one third of the panel dimension. All the cases were designed for optimum beam and column section separately. So the design varies for each of the case



(a) **(b)** **(c)** **(d)** **(e)**

Fig. 1: (a) Layout plan, **(b)** Structure with shear wall in both directions at inner section (Case 1), **(c)** Structure with shear wall in both directions at outer segment (Case 2), **(d)** Structure with Shear wall in short direction at outer segment (Case 3), **(e)** Structure with shear wall in long direction at outer segment (Case 4)

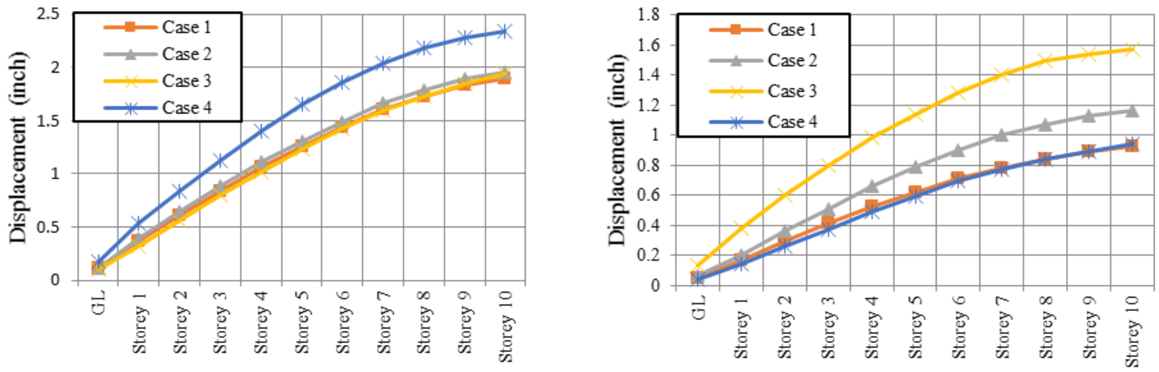
Modeling

All the buildings considered for the case study has been analyzed and designed for gravity load and lateral load. Apart from self weight of the members, 20 psf load for partition wall, 25 psf for floor finish and for live load 60 psf load is given uniformly over the whole slab according to BNBC, edition 2006. All the models considered as case study are residential buildings situated in Dhaka. Wind and earthquake load is calculated and load combinations are given as per BNBC 2006. The buildings are modeled, analyzed and designed by SAP2000V-14 software package.

RESULTS AND DISCUSSIONS

Lateral Displacement of Column for Wind Load

Lateral displacement of column in short and long direction due to wind load is different. Shear wall plays an important role in controlling displacement. Different impact of wind load on lateral displacement of column has been shown in figure 2.



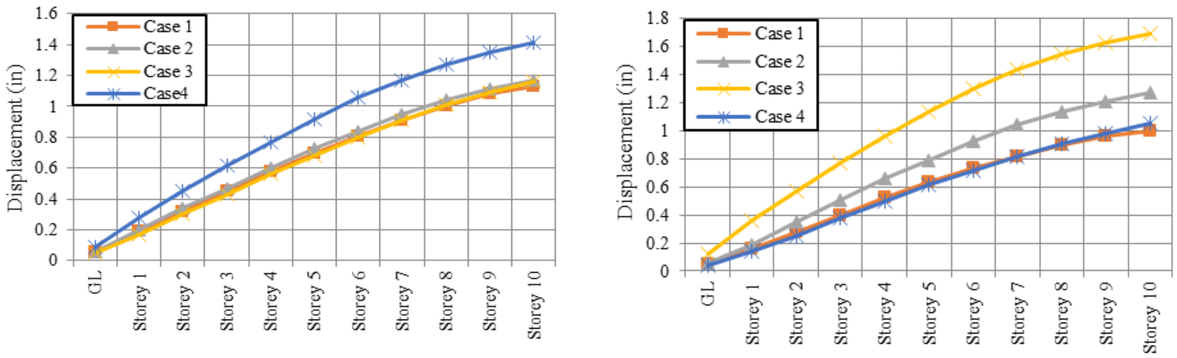
(a) In short direction for external column **(b)** In long direction for external column

Fig. 2: Lateral displacement of column due to wind load

From the above graphs, Case 1 gives the minimum value in all graphs. When shear wall is in short direction, Case 4 gives the maximum value. When shear wall is in long direction, Case 3 gives the maximum value. Case 2 shows average value of the displacement between the maximum and minimum range.

Lateral Displacement of Interior and Exterior Column for Earthquake Load

Lateral displacement for earthquake load of column in short and long direction is different. Different impact of earthquake load on displacement is presented in fig 3.



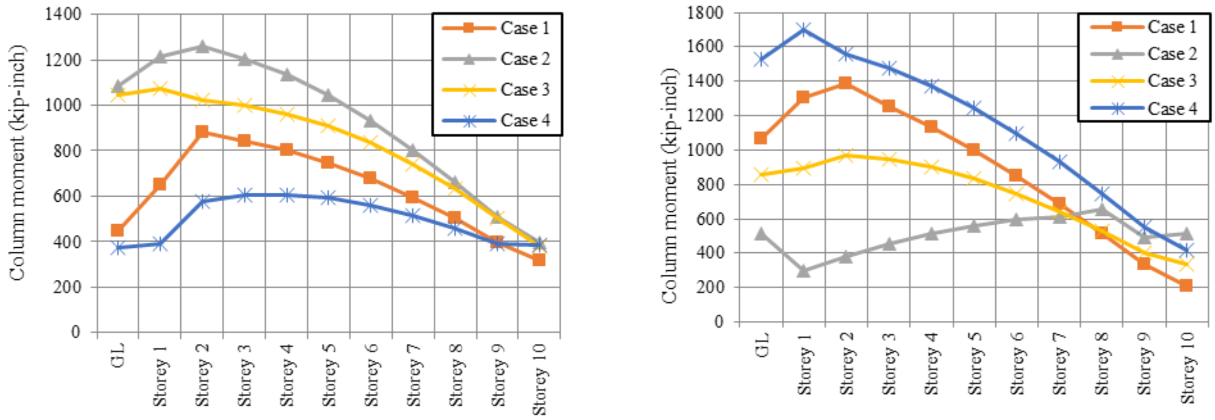
(a) In short direction for exterior column **(b)** In long direction for exterior column

Fig. 3: Lateral displacement of interior and exterior column due to earthquake load

From figure 3, Case 1 gives the minimum value in all graphs. When shear wall is in short direction, Case 4 gives the maximum value. When shear wall is in long direction, Case 3 gives the maximum value. Here also, Case 2 can be taken as optimum for displacement.

Column Bending Moment

The values of column moment in long and short direction are different due to the different arrangement of shear wall on the structure.



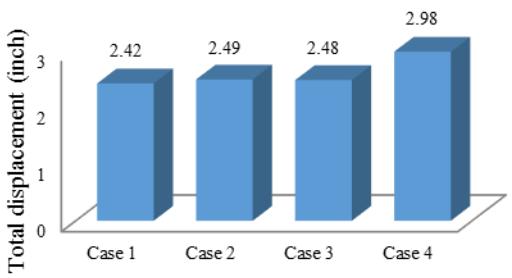
(a) column moment in short direction
(b) column moment in long direction

Fig. 4: Column moment for different storey level.

It is observed from the above graphs, Column moment changes with the position of shear wall. For short direction column moment, the highest values are observed for Case 2 whereas lowest values for Case 4 for column. For long direction Case 4 gives maximum moment for column whereas this case is gives minimum value for short direction.

Maximum storey displacement

Displacement values has been observed at top storey of the building and presented in figure 5 for all the arrangements for most unfavorable combinations of load termed as envelop which gives maximum value. Top storey displacement for Case 4 is maximum with the value of 2.98 inch.

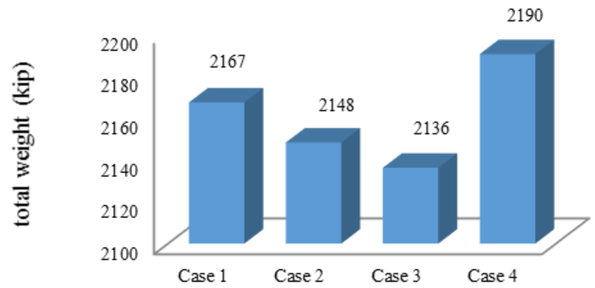


Here, Case 4 gives the maximum displacement as 2.98 inch and case 1 gives the minimum displacement with the value 2.42 inch. It is observed from the figure that the variation of displacement of Case 1, 2 and 3 are very close.

Fig. 5: Bar chart of total displacement of typical storey for different cases.

Comparison of weight for different cases

The comparison of weight among different cases is presented in figure 6 using bar chart.

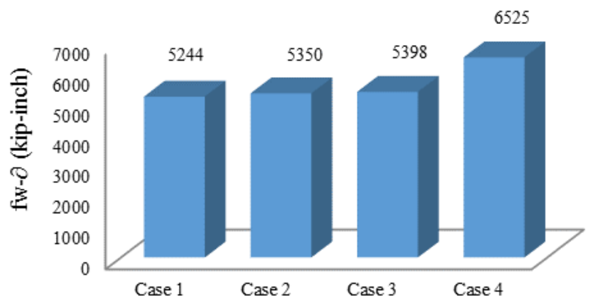


It is observed that, the maximum value of total weight is for Case 4 is 2190 kip and the minimum value is for case 3 is 2136 kip. Case 2 is in middle range.

Fig. 6: Comparison among weight of different elements for different structures

Economic evaluation of structure

For the economic analysis of different cases, the economy has been determined by multiplying the value of weight and displacement that occur under lateral load. So, a factor is established which is denoted by $f_{w-\partial}$. Here, $f_{w-\partial}$ = weight of the structure × Displacement.



It is observed from the figure 7 that Case 1, 2 and 3 have very close values of the economic factor. Case 4 gives maximum value 6525 kip-inch of the factor. Case 1 gives the lowest value of the economic factor among all the cases, which is 5244 kip-inch.

Fig. 7: Multiplying factor of total weight and displacement

CONCLUSION

The primary objective of this study is to select an efficient building structure with shear wall with maximum economy. All the buildings were designed maintaining optimization. Analyzing the results the following conclusions can be drawn

- The lateral displacement varies in short and long direction for column. In both cases minimum values have been found for case 1.
- Similarly for column moment, case 1 and 2 can be taken as most favorable, where case 1 is in the middle range . It also varies on the different arrangement of shear wall.
- For maximum top storey displacement, case 4 gives the maximum displacement and case 1 gives the minimum displacement.
- It is observed that, the maximum value of total weight is for case 4 and the minimum value is for case 3.
- In case of economical analysis, case 1 gives the lowest value of the economic factor which is 5244 kip-inch.

It can be summarized that for serviceability criteria case 1 and 3 have been found better whereas case 1 and 2 is most economical as far as economy is concerned. Considering both economy and serviceability it can be concluded that case 1 is most favorable where shear walls are provided in both short and long directions at inner segment of the structure.

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Authors wish a very happy and peaceful professional life of her.

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FINITE ELEMENT MODELING, ANALYSIS AND VALIDATION OF AXIAL CAPACITY OF RC COLUMNS MADE OF STEEL FIBER REINFORCED CONCRETE (SFRC)

M. M. Islam[1]*, R. Akhter[1], K. S. Islam[1], R. Tabassum[1], M. A. Chowdhury[1], R. Reza[1] & A. Siddique[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.*
**Corresponding Author, <mashfiq7777@gmail.com>*

ABSTRACT

The Steel fiber reinforced concrete (SFRC) which is prepared by adding steel fibers to plain concrete matrix has little effect on pre-cracking behavior in uniaxial compression with substantial enhancement in its post cracking response. This response leads to a greatly improved ductility and toughness. Steel fibers in SFRC are capable of preventing crack propagation as well as brittle failure of columns under uniaxial compression. To this end, four reinforced concrete (RC) square columns and four circular columns made of plain concrete and SFRC are cast and tested in a 1000kN capacity universal testing machine (UTM) to evaluate the axial capacity enhancement due to steel fibers. Enlarged-end fibers are used in this study. Three different aspect ratios of steel fibers (i.e. 40, 60 and 80) are used to cast the column specimens with 1.5% steel fiber volume ratio. The axial capacity is found to be increased by 6%, 12% and 21% for steel fiber aspect ratio 40, 60 and 80, respectively, in case of square columns. The enhancements in circular columns are 27%, 24% and 20% for steel fiber aspect ratio 40, 60 and 80, respectively. These columns are modelled in the finite element platform of ANSYS 11.0 and the FE outcomes and failure patterns uphold a good agreement with the experimental results.

Keywords: Steel Fiber Reinforced Concrete (SFRC), Finite Element (FE) modelling and analysis, Reinforced Concrete (RC) Column, Steel Fiber Aspect Ratio, ANSYS 11.0.

INTRODUCTION

Welcome The development of SFRC began in the early 1960s when researchers first studied the concept of using steel fibers to improve the properties of concrete (Aoude et al. 2009). Since then, the use of SFRC has gathered great interest, with research demonstrating the potential benefits that may lie in the use of the material in both structural and non-structural applications. Several researchers have shown that steel fibers can improve many of the properties of reinforced concrete (RC) including shear resistance, ductility, and crack control. The improved performance results from the ability of the randomly oriented fibers to arrest cracks and the resulting improvements in the post-cracking resistance of the concrete. In addition, some research has been carried out on the potential of using steel fibers in combination with traditional steel reinforcement. In high seismic risk regions, to improve confinement, closely spaced hoops often result in highly congested columns that may cause problems during construction. The use of SFRC in such columns may permit a reduction in the amount of transverse reinforcement, leading to improved constructability. Previous investigations on the compressive behavior of SFRC demonstrated that the toughness of SFRC increases with the product of the volume fraction and the aspect ratio of the fibers. Based on past study, experimental investigations are conducted to study the increase in axial capacities of square and circular RC columns due to steel fiber reinforced concrete (SFRC). However columns made of such concrete must have larger ductile property under axial load. The current research is motivated by this fundamental perception to evaluate the attainable axial capacity enhancement in concrete columns locally available steel fibers in Bangladesh. In this context, SFRC RC columns are made using stone concrete mixed with steel fibers of three different aspect ratios. The percentage of fiber is limited to 1.5 because SFRC with higher amount than 1.5% showed poor workability. End enlarged fibers of various lengths and aspect ratios are considered. Test results are used to establish an analytical model of the compressive stress-strain curve of SFRC, and to focus on determining compressive behaviors. Finite Element (FE) Analysis software ANSYS 11.0 is used to analyze the RC columns made of SFRC and to introduce a FE model for Steel Fiber Reinforced Concrete (SFRC) as well as plain concrete made of stone concrete. From the FE analysis a good correlation is obtained between FE models and experimental results.

MATERIALS, METHODOLOGY AND RESULTS

As the purpose of this study is to investigate the axial capacity of square and circular RC columns made of steel fiber reinforced concrete (SFRC), the columns are casted with only longitudinal rebars and no tie bars are used (Fig. 1a). Crushed stone (CS) aggregates are used to make the plain concrete and SFRC and their compressive and tensile stress-strain behaviour are shown in Fig. 1b & c. Fiber efficiency and fiber content (percentage of fiber by volume or weight and total number of fibers) are the most important governing parameters of SFRC. Fiber efficiency is controlled by the resistance of the fibers to pullout, which in turn depends on the bond strength at the fiber-matrix interface. According to ACI-544.4R-88, enlarged-end fibers show the maximum energy absorption capacity. So in this research enlarged-end fiber is selected. Pullout resistance of fiber is proportional to interfacial surface area and round fibers with smaller diameter offer more pullout resistance per unit volume than larger diameter round fibers because they have more surface area per unit volume. Thus, the greater the interfacial surface area (or the smaller the diameter), the more effectively the fibers bond in concrete matrix. To this end, enlarged-end fibers (Type: V, mild steel cold drawn wires, as per ASTM A 820/A 820M-06) of three different steel fiber aspect ratio (SFAR) i.e. 40, 60 and 80 of circular shape are prepared manually in the laboratory. The second factor which has a major effect on workability is the aspect ratio of the fibers. The workability decreases with increasing aspect ratio. For this reason steel fiber aspect ratio 40, 60 and 80 are used in this investigation with 1.5% volume ratio. According to ASTM: A 820/A 820M–06 the average tensile strength of each fiber shall not be less than 345 MPa (50 ksi). The tensile strength is found about 1,100 MPa (160 ksi) which fulfils the requirement of ASTM standard. In this case the diameters of the steel fiber is 1.18 mm, so the lengths differ for different aspect ratio. Effective lengths for the aspect ratio 40, 60 and 80 are 47.2 mm, 70.8 mm and 94.4 mm respectively and the original lengths are 67.2 mm, 90.8 mm and 114.4 mm respectively. The fibers are bent at 120^{o} at the ends to make the enlarged ends. Fig. 2 (a & b) shows the size and geometry of the steel fiber. Total 4 square columns (SC) and 4 circular columns (CC), 4 compression cylinders and 4 splitting cylinders are tested. The axial capacities are enhanced 6%, 12% and 21% for SFRC square RC columns with SFAR 40, 60 and 80 respectively compared to control column (CON) and ductility enhanced 2, 3 and 4 times respectively, again in case of SFRC circular RC columns axial capacities are enhanced 27%, 24% and 20% respectively and ductility enhanced 3, 4 and 2 times respectively. The results are summarised in Table 1 and 2. Fig. 1 b and c represents the compressive and splitting tensile behaviour of SFRC cylinders. All specimens are tested at constant rate of 0.5mm/min by a displacement control 1000kN capacity digital universal testing machine (UTM) until the failure occurred. Mild steel rebar of 60 grade (yeild strength 60 ksi or 420 Mpa) is used as longitudinal reinforcement. Lateral strain of splitting cylinders at the failure location are measured by analyzing the image histories obtained from high speed video clips and high definition (HD) images employing an image analysis technique which is called digital image correlation technique i.e. DICT (Islam et al. 2011, Islam et al. 2014). Then the load and displacement values from UTM are synthesized with the strain measurement results gathered from DICT.

Table 1: Summary of the concrete cylinder specimen

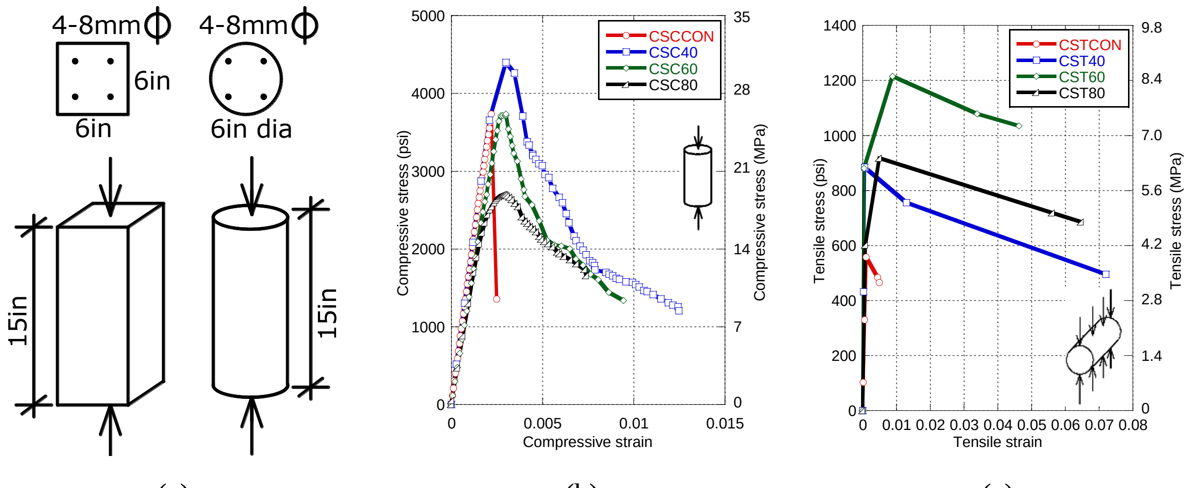
Specimen ID	Maximum compressive strength, psi (MPa)	Specimen ID	Maximum tensile Strength, psi (MPa)	SFAR
CSCCON	3741 (26)	CSTCON	560 (4)	-
CSC40	4400 (30)	CST40	885 (6)	40
CSC60	3733 (25.7)	CST60	1218 (8.5)	60
CSC80	2691 (19)	CST80	919 (6.5)	80

Table 2: Summary results of square column (SC) and circular column (CC)

Specimen designation (Sqr. Col.)	Axial Strength MPa (psi)	% incre-ment	Ductility increased (times)	Specimen designation (Cir. Col.)	Axial Strength MPa (psi)	% incre-ment	Ductility increased (times)	SFAR
CSSCCON	26 (3760)	-	-	CSCCCON	28 (4000)	-	-	-
CSSC40	28 (4000)	6	2	CSCC40	35 (5097)	27	3	40
CSSC60	29 (4228)	12	3	CSCC60	32 (4963)	24	4	60
CSSC80	32 (4563)	21	4	CSCC80	33 (4797)	20	2	80

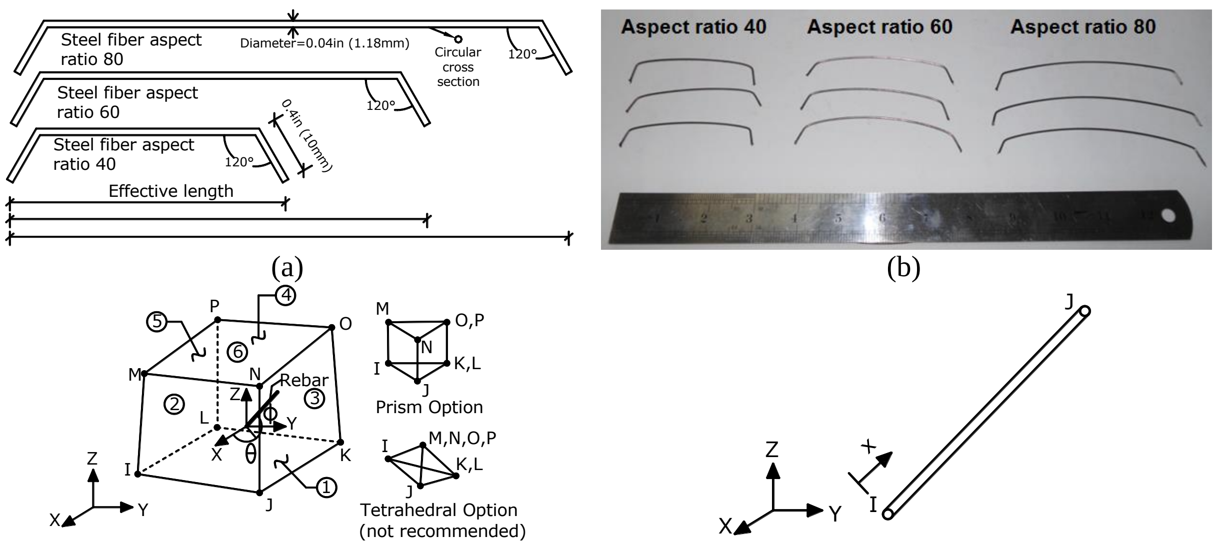
Table 3: FE input data for SOLID65 and LINK8 element

Properties for FE model	Column specimen (SOLID65)				Rebar (LINK8)
	CSSCCON, CSCCCON	CSSC40, CSCC40	CSSC60, CSCC60	CSSC80, CSCC80	
Density	2.69g/cm^3	2.77g/cm^3	2.72g/cm^3	2.74g/cm^3	7.8g/cm^3
Tensile strength	4 MPa	6 MPa	8 MPa	6.3 MPa	-
Poisson's ratio	0.3	0.325	0.325	0.325	0.3
Shear transfer co-efficient: closed crack	0.25	0.5	0.5	0.5	-
Open crack	0.3	0.3	0.3	0.3	-
Yield stress	-	-	-	-	420 MPa



(a) (b) (c)

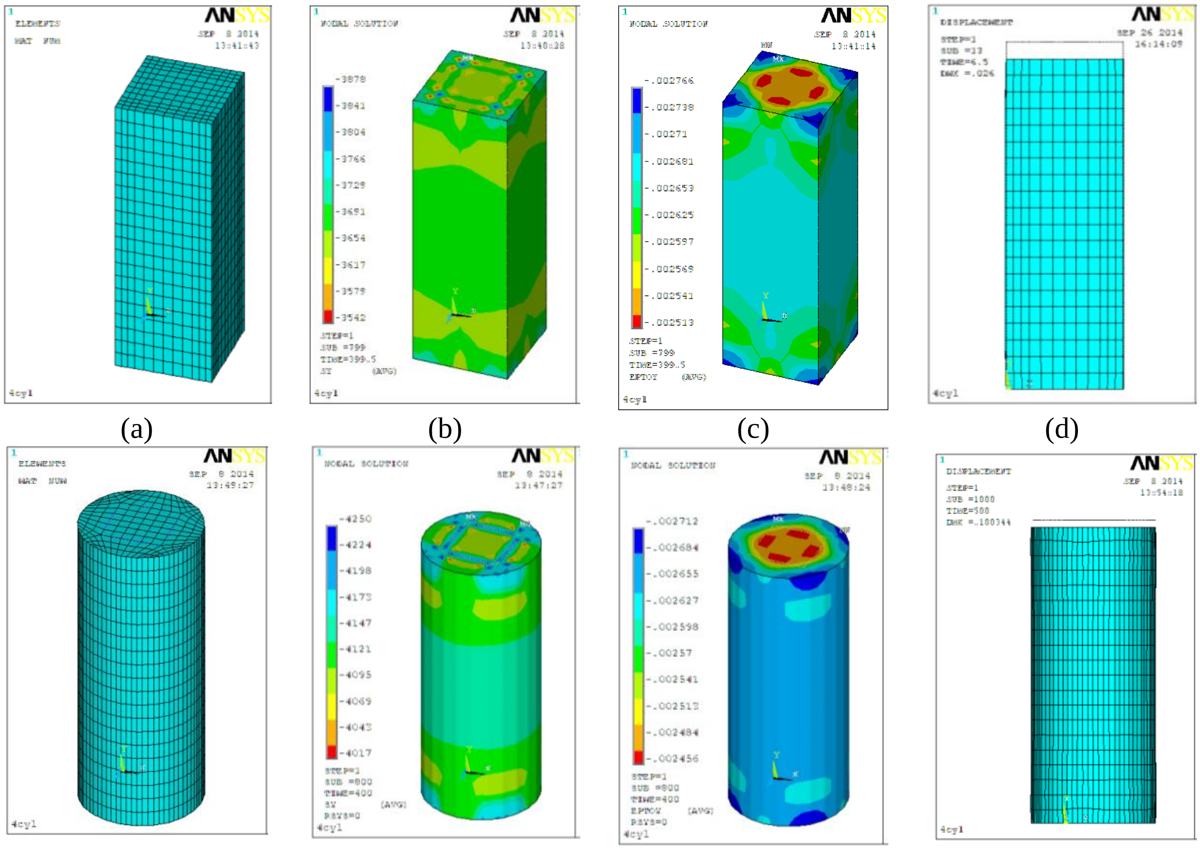
Fig. 1. Experimental results of plain concrete and SFRC (a) Reinforcement layout and column dimension as per strategy (b) compression (c) splitting tension behaviour of concrete.



(a) (b)

(c) (d)

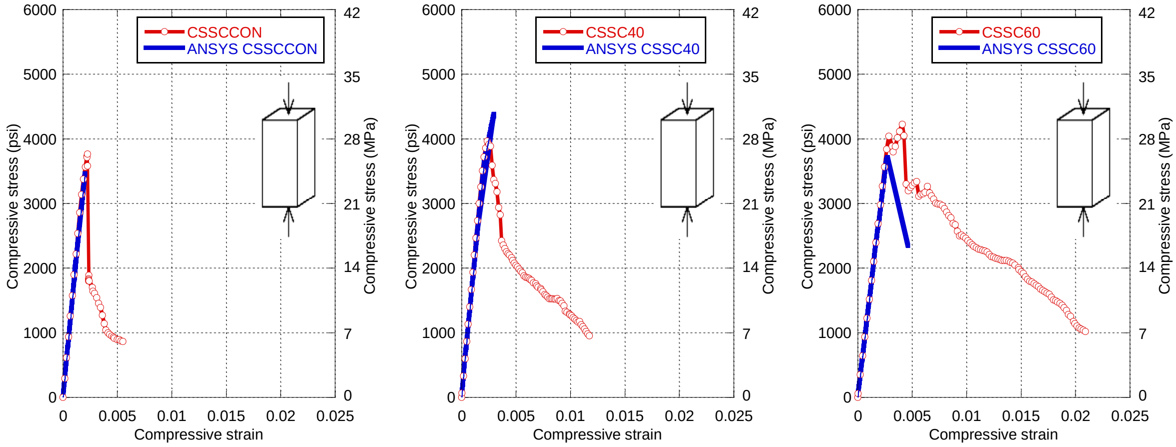
Fig. 2. (a) & (b) size, geometry & image of steel fibers of different SFAR, (c) & (d) geometry of SOLID65 and LINK8 respectively.



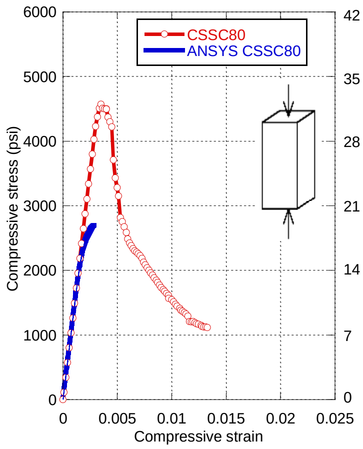
(a) (b) (c) (d)

(e) (f) (g) (h)

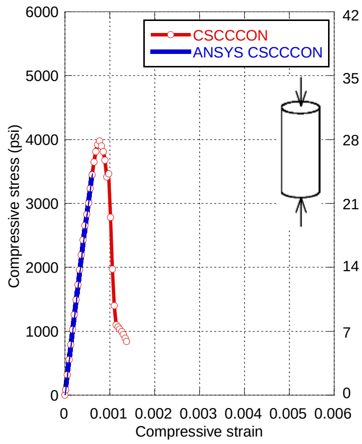
Fig. 3. Typical image of (a), (e) FE model, (b), (f) Y direction stress contour, (c), (g) Y direction strain contour, (d), (h) failure pattern of square and circular column.



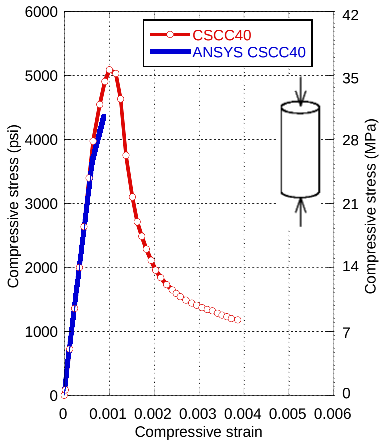
(a) (b) (c)



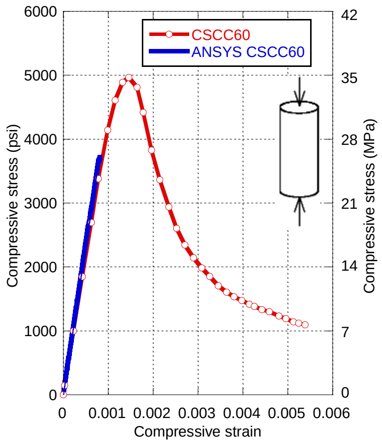
(d)



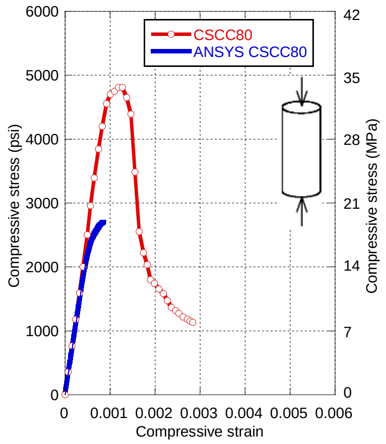
(e)



(f)



(g)

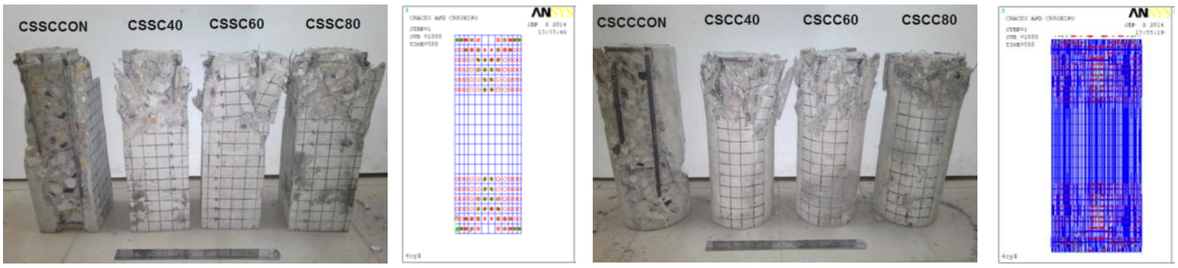


(h)

Fig. 4. Validation of FE model (a), (b), (c), (d) square columns (e), (f), (g), (h) circular columns.

FE MODELLING, ANALYSES AND VALIDATION

Plain concrete and SFRC columns are modelled using SOLID65 element in ANSYS 11.0 which is a three dimensional (3D) solid element with eight nodes and the reinforcement is modelled using LINK8 element which is a 3D spar element. Both elements having three degrees of freedoms, i.e., translation in the nodal x, y, and z directions at each node. SOLID65 is capable of plastic deformation, cracking in tension, crushing in compression and is also applicable for reinforced composites (ANSYS 2005), such as, fibreglass, SFRC etc. The geometry and node locations for SOLID65 and LINK8 elements are shown in Fig. 2c & d. For the perfect modelling of plain concrete and SFRC, ANSYS 11.0 requires data for material properties such as (i) elastic modulus, (ii) density (iii) Poisson’s ratio (iv) stress-strain behaviour (v) tensile strength (vi) shear transfer co-efficient for open and closed crack etc. Table 3 provides the properties applied in FE modelling. Loading is applied as displacement boundary condition in 500 steps followed by 2 sub-steps for each step. Fig. 3 shows the FE models, stress and strain contours in Y direction and deformed shape of the square and circular columns. Fig. 4 shows the validation of results gathered from the experimental investigation with the FE analysis by ANSYS 11.0 and demonstrates the accuracy of the FE model plain concrete as well as SFRC. FE analyses have shown conservative results in most of the cases compared to experimental result which indicate sufficient factor of safety and also ensure a reliable FE model. In the both cases, the failure patterns are found almost similar and at same locations which also validates the FE models (Fig. 5).



(a) (b) (c) (d)

Fig. 5. Failure pattern of experimental specimens and FE model (a), (b) square column, (c), (d) circular column

CONCLUSION

The axial capacities are enhanced 6%, 12% and 21% for SFRC square RC columns with SFAR 40, 60 and 80 respectively compared to control column and ductility enhanced 2, 3 and 4 times respectively again in case of SFRC circular RC columns axial capacity enhanced 27%, 24% and 20% respectively and ductility enhanced 3, 4 and 2 times respectively. There exist a good aggrement between the experimental results and FE outcomes. FE analyses have shown conservative results in most of the cases compared to experimental result which indicate sufficient factor of safety and also ensure a reliable model.

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STRENGTH EVALUATION OF MORTAR BY INCLUSION OF STONE DUST AS A CEMENT AND SAND REPLACING MATERIAL

M. T. Raihan[1*] & I. B. Muhit[2]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong- 4349, Bangladesh.*
[2] *Department of Civil Engineering, Chung-Ang University, Seoul 156-756, Korea.*
**Corresponding Author, <tanveerraihan.m2n@gmail.com>*

ABSTRACT

Mortar, a matrix of concrete which is a masonry product, composed of binder and fine aggregates. It is an essential associate in any reinforced structural construction. The strength of mortar is a special concern to the engineer because mortar is responsible to give protection in the outer part of the structure as well as at a brick joint in masonry wall system. The lack of mortar strength sometimes throws the whole structure in a great danger. Cement, sand and water are the key parameters of mortar which are dealing with the strength directly. The purpose of this research is to investigate the compressive and tensile strengths of mortar by replacing the cement and sand by stone dust. This research is focused on the comparison between fresh mortar and modified mortar. For sand replacement, the gradation and fineness modulus of stone dusts was kept similar to that of sand. Stone dusts passing by No. 200 sieve was used as cement replacing material. The portion of dusts which was retained on No. 100 sieve was separated to substitute sand and the remaining part of dusts was made finer by abrasion machine. Then the stone dusts were screened again by No. 200 sieve and the dusts passing by No. 200 was used as the cement. The compressive and tensile strengths of modified mortar were investigated by replacing of 20%, 30% and 40% of fine aggregates as well as replacing of 5% of cement by stone dusts. Several numbers of cube and briquette samples were cast with aforementioned proportions to investigate compressive and tensile strengths at 7 days and 28 days of curing. From tested results, it was found that the compressive strength of samples of 30% of sand replacing stone dust with 0% of cement replacing stone dust increases by 12% and 17% at 7 and 28 days while for the tensile strength it was increased by 8.7%. However, the mechanical properties of mortar with stone dusts as replacement of cement shows no satisfactory results.

Keywords: Mortar, Cement, Sand, Compressive Strength, Stone Dust, Replacement

INTRODUCTION

Mortar is a product composed of cement and sand. When water is mixed in with this product, the cement is activated. Whereas concrete can stand alone, mortar is used to hold together bricks, stones or other such hardscape components (Aziz, 1995).

A complete understanding of mortar and its application is huge to accomplish effective execution. When water blended with Portland cement creates pitiless, solid glue that is very unworkable, getting to be hard rapidly. Some Portland cement aids the workability and versatility of the mortar. It likewise gives early quality to the mortar and rates setting. Fine aggregate is basically sand extracted from the land or the marine environment. Fine aggregates generally consist of natural sand or crushed stone with most particles passing through a 9.5 mm sieve. For concrete sand FM range is 2.3 - 3.1 (Mobasher, 1999).

The main constituents of concrete such as sand, stone and water are mainly natural resources. Sand is the general segment of mortar which provides for its different shade, surface and cohesiveness. Sand must be free of polluting influences, for example, salts, earth or other remote materials. The three key characteristics of sand are particle shape, gradation and void ratio. Sand is mainly used as inert material to give volume in mortar for economy. It offers requisite surface area for film of cementing material to adhere and spread, prevents shrinkage and cracking of mortar. The strength of mortar or concrete is largely affected by the fine aggregates (Sharmin et al., 2006). Fine aggregate is usually sand from river (Lohani et al., 2012). The main constituents of mortar is sand are mainly natural resources. The presence of very fine materials in excessive quantities influences the performance and properties of fresh and hardened mortar or concrete.

In fresh concrete, the workability, air content and bleeding are reduced depending on the amount and composition of the very fine materials in concrete, the cement content and the grading of the sand (Popovics, 1979 ; Kalcheff, 1977 ; Malhotra, 1985). In the hardened state, the presence of fine materials can be beneficial for low strength concrete but it may have adverse effects on high strength concrete, since the shrinkage of concrete increases (Ahmed, 1989) and its durability is impaired (Popovics, 1979).

Alternative material of sand should be explored to mitigate the increasing demand of sand. A considerable amount of dust is produced at the time of stone crushing. They are often considered as a waste in the locality. Saving of natural resources and environment is the essence of any advancement (M.Veera Reddy, 2010).

Numerous attempts have been done since the ancient time and it is still continued to use the waste materials in construction work. Stone dust, fly ash, silica fume, rice husk etc are the waste materials. Exchange of normal sand by stone dust will assist both solid waste minimization and waste recovery (H.M.A.Mahzuz, 2011). Several researches have been made (A.A.M.Ahmed, 2010; Lohani et al., 2012) to discover a proper way of using the stone dust without affecting the strength of cementitious product.

For Mortar, stone dust is well appropriate to choose it as an alternative of sand. According to Masrur (2010) about 100000 cft of stone dust is generated during stone crushing which is almost equivalent to 1.6 million BDT.

With the rapid growth of contraction industries consumption of construction material is increased. Again with the industrial development waste material generation is occurring in a massive quantity. In this present work the main objective is to determine the acceptability of stone dust as replacing substance of both binding material and fine aggregate in mortar in respect of the normal strength. This study ensures the stone powder as an appropriate alternative of sand (fine aggregate) in concrete manufacturing as a building materials

MATERIALS & SPECIMENS PREPARATION

For this study we have used high strength Portland cement. The physical & chemical properties of cement are tabulated in Table 1.1

Table 1

Portland Cement Properties	
Physical Properties	
Initial Setting Time (minute)	64
Final Setting Time (minute)	121
Specific Surface Area (cm^2/gm)	3907
28 Days Compressive Strength (MPa)	22.06
Chemical Properties	
Calcium Oxide (CaO)	62.25%
Silicon Dioxide (SiO_2)	21%

Aluminium Oxide (Al_2O_3)	5.9%
Sulphur Trioxide (SO_3)	2.4%
Ferric Oxide (Fe_2O_3)	3.4%
Magnesium Oxide (MgO)	1.5%
Sodium Oxide (Na_2O)	0.2%
Potassium Oxide (K_2O)	0.45%
Loss of Ignition	1.1%

- Graded river sand (Sylhet Sand) was used to conduct the tests. The fineness modulus of the sand used was 2.8. Sand samples were washed and dried so that there should not remain any dust particle. They were free from organic chemicals & unwanted clay.
- Stone dusts were processed in two forms, one for the replacement of sand and another for the replacement of cement. For sand replacement the gradation & fineness modulus of stone dust was tried to keep similar to the sand. Stone dust retained at no. 100 sieve was selected for sand replacing stone dust. Stone dust passing by No. 200 sieve was used as cement replacing material. Stone dust was collected from nearby stone crushing plant to have exact quality in field.
- Normal drinking water was used collected from available source.
- The specific surface area of stone dust replaced for cement in mortar sample was 2529 cm^2/gm and for cement was 3907 cm^2/gm. It signifies that the size of dust particle is larger than the cement particle. This scenario also defines the negative impact of using stone dust as replacement of cement.

Mortar Sample Preparation & Curing

Cube & Briquette samples were tested in this research purpose to get some clear idea about both tensile and compressive strength. Mortar materials were mixed according to ASTM C109 standard.

The water cement ratio for mortar without stone dust was 0.41. Water cement ratio for the mortar samples with stone dust was varied from 0.41 to 0.51. Water demand increases with the increase of stone dust content in mortar.

Dimension of the cube mould for compressive strength test was 5.08 cm x 5.08 cm x 5.08 cm. Figure 1 shows the cube casting in molds.

[Fig 1] Cube Mortar Sample Casting

Standard dimension briquette molds were used for preparing briquette specimens for tensile strength test. Seven different sample types were prepared for casting. Three samples were examined replacing sand only. The percentages were 20, 30 & 40. Where one sample was casted as a replacement of

cement at the percentage was 5. Two more samples were also experimented to observe the strength while replacing both cement & sand. The mixture proportions of all specimens are tabulated in Table 2.
The replacement percentage was fixed in a proportion that, we can get some clear specification while for different dosage of stone dust. As from different previous researches it is found that, if cement is replaced with more than 5% stone dust- the strength quality is not noticeably increase. So in this case, replacing the percentage was fixed for 5%.
The cube moulds were tested after 7 & 28 days. Meanwhile, briquette samples were tested after 28 days.

Table 2

Sample Name	Details
M1	0% of Sand Replacing Stone Dust + 0% of Cement Replacing Stone Dust
M2	20% of Sand Replacing Stone Dust + 0% of Cement Replacing Stone Dust
M3	30% of Sand Replacing Stone Dust + 0% of Cement Replacing Stone Dust
M4	40% of Sand Replacing Stone Dust + 0% of Cement Replacing Stone Dust
M5	0% of Sand Replacing Stone Dust + 5% of Cement Replacing Stone Dust
M6	20% of Sand Replacing Stone Dust + 5% of Cement Replacing Stone Dust
M7	40% of Sand Replacing Stone Dust + 5% of Cement Replacing Stone Dust

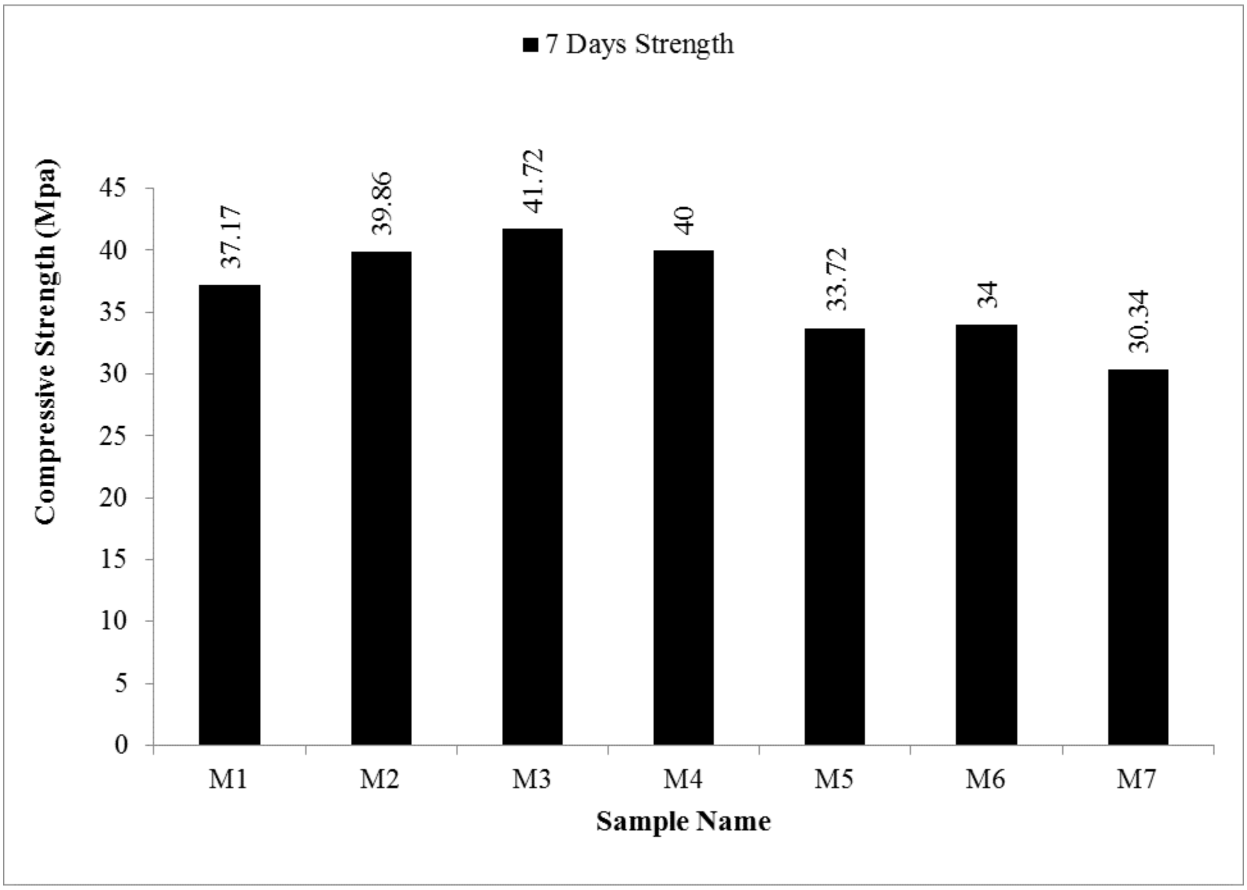
In case of sample preparation, sand & binder materials were mixed perfectly in dry condition & then according to water binder ratio, weighted amount of water was added to the homogenous mixture. Cement-Sand ratio was taken as 2.5. For both cube & briquette moulds were prepared with mould oil so that the surfaces of the moulds remain free from disturbance. A total of 21 times temping were performed on each cube mortar sample.
Compressive strength test was performed via Universal Testing Machine at a constant loading rate. In Figure 3, the arrangement of sample is shown. The average from two sample of each type was recorded for the compressive strength of each type which was tested at 7 & 28 days.

[Fig 2] Mortar Samples is in Universal Testing Machine
Tensile strength test of briquette sample was also performed with the same mixture of different types & an average of two sample of each type was recorded for the tensile strength.
Underwater curing process was followed in this experiment. The mortar samples were removed from moulds after 24 hours of casting. Then they were kept under water in a bowl and were kept

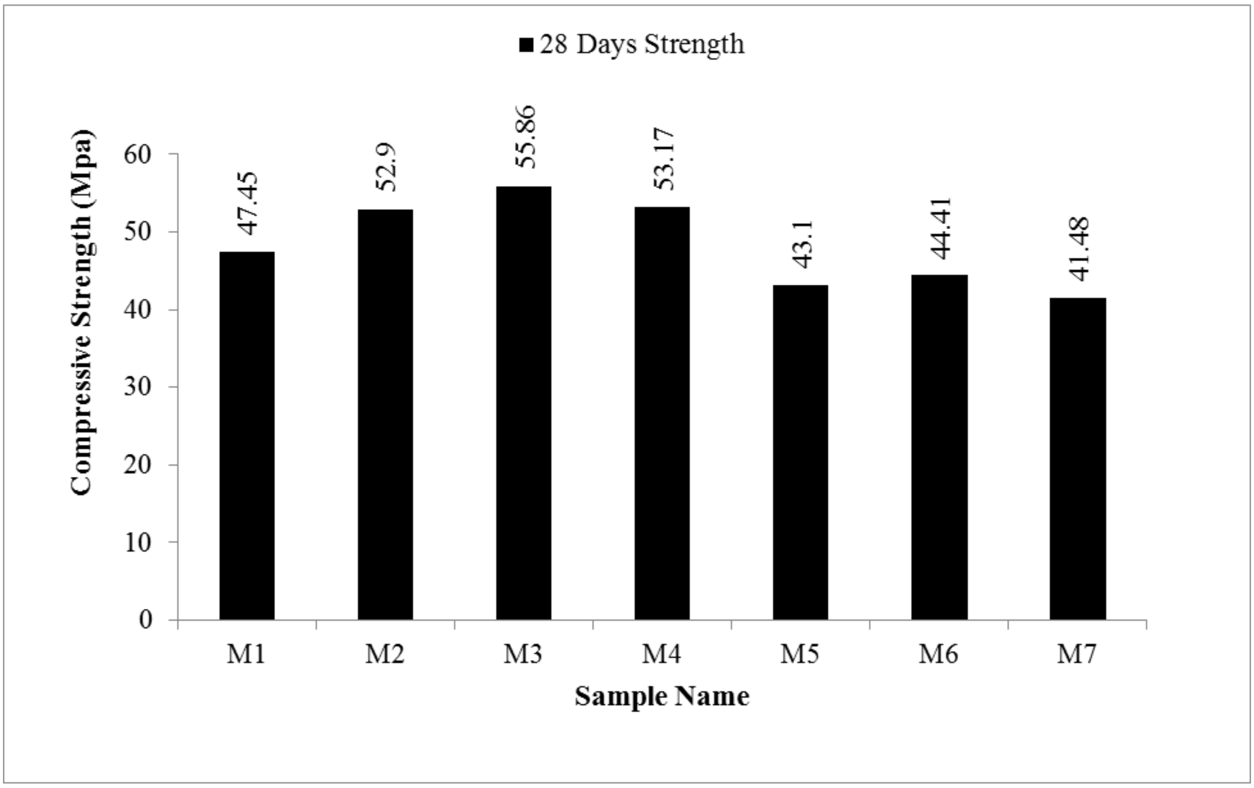
undisturbed before the time of crashing. Just before placing UTM the mortar samples were kept under sun for some period, so that they can overcome the effect of water at its surface.

RESULT & DISCUSSION

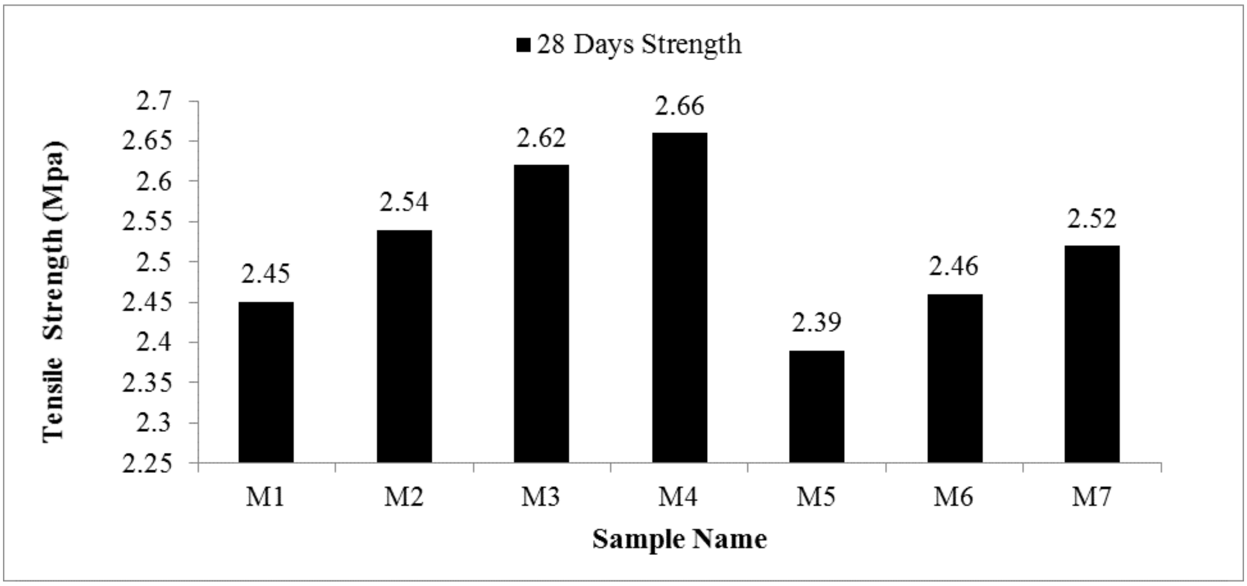
The development of compressive strength at 7 & 28 days are shown in Figure 2 & 3 respectively. The Figures shows that, the highest value of compressive strength for 7 days is 41.72 MPa & for 28 days it is 55.86 MPa. In both cases Mortar Sample M3 gives the highest value. M3 is a mixture of 30% Sand Replacing Stone Dust & 0% Cement Replacing Stone Dust. At 7 & 28 days compressive strength of M3 sample increases around 12% & 17% respectively than the control specimen (M1).
Moreover, M4 (40% of Sand Replacing Stone Dust + 0% of Cement Replacing Stone Dust) gives 2nd highest value (40 MPa for 7 days and 53.17 MPa for 28 days) of compressive strength. At 7 & 28 days compressive strength of M4 sample increases around 7.5% & 12% respectively than M1. But in case of cement replacement there was a decrease in strength compared to control specimen.
As mentioned earlier we took the 28 days tensile strength value for tensile strength determination. From Figure 4, it is shown that, the highest tensile strength value is for 40% sand replacing stone dust (M4). The value is 2.66 MPa and it is around 8.7% increased than control mortar sample M1.



[Fig. 2] 7 days Compressive Strength



[Fig. 3] 28 days Compressive Strength



[Fig. 4] 28 days Tensile Strength

CONCLUSION

According to the analysis of the whole study following conclusion can be drawn,

- We can use stone dust as a replacement of sand in case of mortar preparation which gives some good results in strength.
- Using stone dust as 30% replacement of sand gives the highest strength and after increasing the percentage the strength becomes lower.

- Using stone dust as a replacement of cement is case of mortar preparation cannot give any satisfactory result.
- Stone dust is quite appropriate to be selected as the substitution of fine aggregate but not as the replacement of cement.
- Stone dust has a potential to provide alternative to fine aggregate minimizing waste products. Thus the stone dust will introduced as a functional construction materials.

From this intensive research we can able to know that depending on the percentage of using & type of replacing stone dust may have positive or negative effect on mortar strength. We may use the favorable site of the replacement of stone dust which truly helps to make best use of some waste material and ensure some sustainable development.

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TREATMENT OF RECYCLED AGGREGATE BY IMPREGNATION OF OPC TO IMPROVE CONCRETE PROPERTIES

M. O. Imam[1], Tafsirojjaman[1], M. M. Rashid[1*]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong 4349, Bangladesh.*
**Corresponding Author, <mahabub.ce09@yahoo.com >*

ABSTRACT

The mechanical properties of cycled aggregate concrete were studied. The recycled aggregates were prepared from the old concrete structures. It was observed that the recycled aggregates are covered with loose particles preventing a good bonding between the new cement matrixes and the cycled aggregates. The old cement paste that remained on the natural aggregate was porous and cracked leading to weak mechanical properties of the recycled aggregates. In the present study, a treatment by impregnation of ordinary Portland cement solution to the recycled aggregates was applied to improve their mechanical properties. The compressive strength tests of recycled aggregates concrete were carried out at two ages of curing for two target strengths. The test results showed that the compressive strengths of recycled aggregate concretes with the given treatment increase by 21 and 14%, respectively for 7 and 27days of curing, in compared to the normal aggregate concrete for the target strength of 3000 psi while for the target strength of 4000 psi, it was found 19 % and 13 %, respectively.

Keywords: Treated recycled coarse aggregate, fine aggregate of FM 1.23, Physical properties of aggregate, Concrete, Compressive Strength.

INTROUDUCTION :

Concrete has been proved to be a leading construction material for more than a century. A report presented in 1999 to the European Commission estimated the amount of non-recycled construction waste to be 130 million t / year. The area required for landfilling this amount of waste is equivalent to the accumulation of waste, 1.3 m high, over the entire central Paris area (Symonds, 1999). When a sample from material discharged by a civil construction site is analyzed, it can be ascertained that, even with its heterogeneity, almost all of the substances in its composition are high valued and have good mechanical resistance, such as: sands, brittle stones, concretes and hardened mortars, bricks and ceramic fragments, woods and many others. All of them are potential raw materials. Therefore, civil construction waste can be real raw material deposits that could be explored. As such, means for reutilizing recycled discharged materials can be employment in pavement works, adjustment and grovel covering on soil streets, drainage works, mortar ballast execution and concrete production (B. Filho et al., 1999).

It appears that recycling construction waste is vital both in order to reduce the amount of open land needed for landfilling and to reduce depletion of raw materials. At the same time, large quantities of natural aggregates are extracted for construction every year leading to the large scale depletion of natural aggregate and the increased amounts of C&DW. The construction and demolition waste are primarily used for landfill sites which are causing significant damage to the environment and developing serious problems. The use of the recycled aggregates created from processing of construction and demolition waste in new construction has become more important over the last two decades as it conserve the non-renewable natural resource of virgin aggregates.

Several methods to improve the properties of new concrete made from recycled aggregate were reported in the literature. Sri Ravindrarajah and Tam improved the properties of new concrete by altering the water/cement ratio, adding pozzolans, and blending recycled and natural aggregates (S. Ravindrarajah & Tam,1988).These techniques, however, refer to general concrete technology and not to the improvement of the recycled aggregate itself. Montgomery treated the aggregate with a ball mill in order to remove old cement paste from natural stone. He found that the cleaner the aggregate was, the stronger was the concrete (Montgomery,1998). Winkler and Mueller milled recycled fines and used them as a cement replacement (Winkler and Mueller, 1998).A reduction of 17% in the compressive strength of the concrete, at a replacement ratio of 33%, is reported by Montgomery. Compared to natural aggregate concrete the compressive strength of recycled aggregate was decreased by 18.76%. The recycled aggragate treated with water has increased 4.93%, nitric acid by 11.88%, sulphuric acid increased by 5.38% and hydrochloric acid increased by 7.17% than the recycled aggregate (G. Murali et al.).

With a view to the above needs, aims of present study are treatment to improve material properties of the recycled aggregate (by impregnation of ordinary Portland cement solution),to improve the properties of new concrete made from recycled aggregate, to achieve the different target strength of recycled coarse aggregate and OPC coated recycled coarse aggregate using fine aggregate of FM 1.23,To compare percentage increase in 3000 psi and 4000 psi, to compare the strength of concrete of two types(3000 psi and 4000 psi).

MATERIALS AND METHODS

The Ordinary Portland Cement conforming was used for the preparation of test specimens. The natural fine aggregate (NFA) used throughout the study was natural river sand. The fine aggregate fineness modulus 1.23 was used for the preparation of test specimens.

Table-1: Determination of fineness modulus of local sand

Sieve Size	Opening (mm)	Weight of Materials Retained(mm)	% Retained	Cumulative % Retained	% Finer
No.4	4.76	0	0	0	100
No.8	2.36	1.3	0.26	0.26	99.74
No.16	1.18	2.4	0.48	0.74	99.26
No.30	0.6	4.3	0.86	1.6	98.4
No.50	0.3	164.6	32.88	34.48	65.52
No.100	0.15	259.6	51.86	86.34	13.66
Pan		68.4	13.66	100	0

Weight of sample= 500 gm.

FM=1.23

On the other hand, Two type of coarse aggregate were used for this investigation viz, recycled coarse aggregate (RCA), treated recycled coarse aggregate (TRCA). In this study, recycled coarse aggregate were collected as broken cylindrical specimen of compressive strength 3000 psi from strength of materials laboratory, Civil Engineering Department, CUET, and crushed into coarse aggregates manually, which are defined as recycled coarse aggregate. After grading, maximum size of aggregate is 20 mm (3/4 in) and evaluates the properties of the recycled coarse aggregate.

TEST SPECIMENS:

Through mixing of the materials is essential for the production of uniform concrete. The mixing should ensure that the mass becomes homogenous, uniform in color and consistency. For proper mixing vibrator are used here in preparation of cylindrical specimen (6 in. dia and 12 in. long) by ACI 211.1 method of mixed design. For preparing cylindrical specimen, here used fine aggregates FM= 1.23. The grade of concrete which we adopted were 3000 psi with water cement ratio of 0.68 4000 psi with water cement ratio of 0.57. The samples are casted using the two different aggregate. A series of test (BS 1881 : Part 110:1983 and by ASTM C 192-90a) were carried out in the laboratory, to determine the compressive strength of recycled concrete cylinder after curing of 7 days and 28 days in fresh water. The details of the specimen and their notations are given below in table-2.

Table-2:List of Test Specimens:

No.	Type of Concrete	Compressive Strength(Psi)	Curing Days	Specimens Nos.
1.	Recycled Coarse Aggregate Concrete(RCAC)	3000	7	5
			28	5
		4000	7	5
			28	5
2.	Treated Recycled Coarse Aggregate Concrete(TRCAC)	3000	7	5
			28	5
		4000	7	5
			28	5
				Total : 40

Treatments to Improve the Recycled Aggregate:

Two effects seem to have a detrimental effect on the quality of the recycled aggregates (apart from the water/cement ratio of the old cement matrix). These are (1) coating of aggregates with loose particles, which damages the bond between the new cement matrix and the recycled aggregate and (2) cracking of the old cement matrix, which decreases the mechanical strength of the recycled aggregate. The method was proposed to improve the quality of the recycled aggregates by impregnation with a solution of Ordinary Portland Cement that is intended to add a thin layer of Ordinary Portland Cement particles over the surface of the recycled aggregate. The Ordinary Portland Cement is expected to react with calcium hydroxide from the hydration of the cement to form a dense layer covering the surface of the aggregate, which, in turn, will increase its strength.

Ordinary Portland Cement Impregnation:

A solution of 10 L of water and 1 kg Ordinary Portland Cement was prepared by mixing small batches of solution in a mixer, super plasticizer (1% weight of Ordinary Portland Cement) was added to ensure proper dispersion of the Ordinary Portland Cement.

The Recycled aggregate was dried, and soaked in the Ordinary Portland Cement solution for 24 h. Weight measurements were taken before and after the Ordinary Portland Cement treatment. The saturated aggregate was then dried to ensure proper penetration of the Ordinary Portland Cement particles into the surface of the aggregate and weighted again after drying.

The amount of Ordinary Portland Cement impregnated into the surface of the Recycled aggregate can be estimated at 1–1.5% of the aggregate weight based on the weight gain after it was taken out of the Ordinary Portland Cement solution.

Figure-1 presents the images of the recycled aggregate surface before and after the Ordinary Portland Cement impregnation respectively. After impregnation, the surface is covered with a layer of Ordinary Portland Cement particles and some crumbs are still seen, though in smaller quantities.

Figure-1: Recycled Aggregate and Treated Recycled Aggregate.

RESULTS

The results indicate an improvement in the properties of recycled aggregate after treatment. The treatment effect was more significant at an early age.

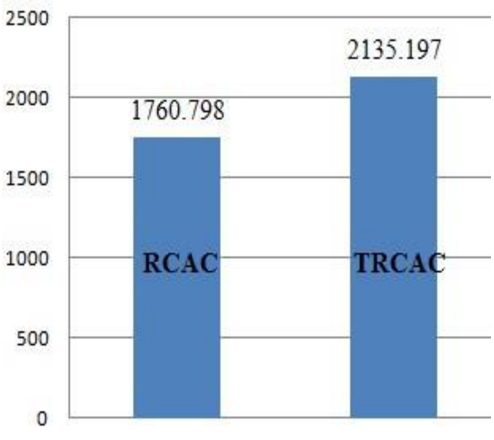


Fig 2: Comparison of compressive strength at 3000 Psi in 7 days (increase 21.26%).

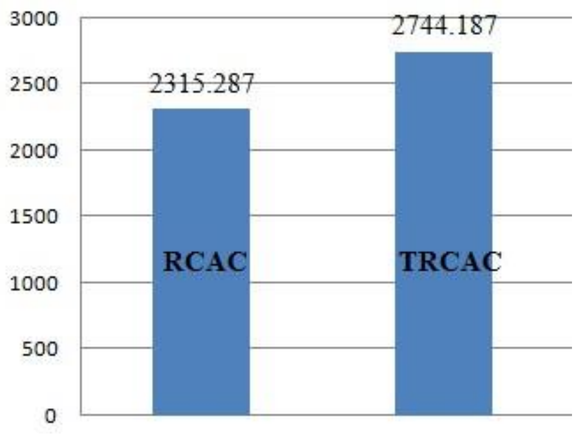
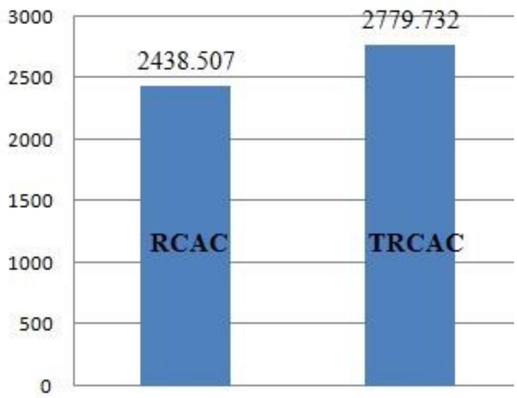


Fig 3: Comparison of compressive strength at 4000 Psi in 7 days (increase 18.52%) .



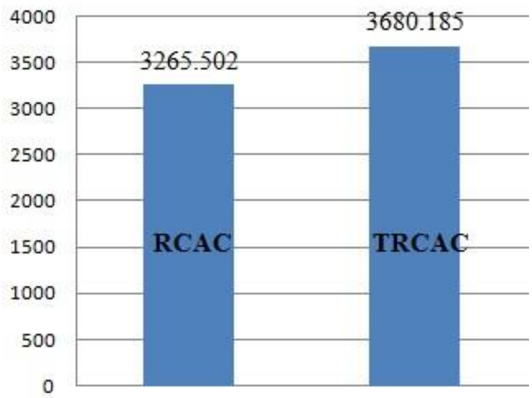


Fig 4: Comparison of compressive strength strength at 3000 Psi in 28 days (increase 13.99%).

Fig 5: Comparison of compressive at 4000 Psi in 28 days (increase 12.69%).

DISCUSSIONS AND CONCLUSION

Treatment was evaluated, with the purpose of improving the surface and physical properties of the Recycled aggregate by impregnation of the recycled aggregate with a 10% by weight ordinary Portland cement solution.

It was observed that an increase of 21.26% and 14% in the compressive strength at ages 7 and 28 days was observed after the OPC treatment for 3000 Psi concrete and an increase of 18.52% and 12.7% in the compressive strength at ages 7 and 28 days was observed after the OPC treatment for 4000 Psi concrete.

It appears that OPC impregnation improves both the interfacial transition zone between the Recycled aggregate and the new cement matrix, and the mechanical properties of the Recycled aggregate. As a result, the early strength of the new concrete increases significantly when the disparity between the properties of the Recycled aggregate and the new cement matrix is relatively small and the filler effect of the OPC is dominant. At a later age, after the cement matrix has strengthened, these effects are weaker, leading to a lesser influence on the strength. Cracking of the old cement matrix seems to have a strong influence on the properties of the Recycled aggregate.

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A STUDY ON THE STRENGTH BEHAVIOUR OF CONCRETE IN NACL ENVIRONMENT

Md Refat Ahmed Bhuiyan[1], Md Saiful Islam[1] & Md Moinul Islam[1]

[1] *Department of Civil Engg. CUET, Ctg-4349, Bangladesh.*
**Corresponding Author, <ref_ce@yahoo.com>*

ABSTRACT

As a construction material the reinforced concrete has been extensively used all over the world for a long duration. Due to its inherent durable charechteristics, structural concrete is considered as leading option for construction of many onshore and offshore structure. The structural constituents of hydrated cement matrix are very sensitive to deterioration when exposed to salt action in coastal areas where sea water contains different salt including chlorides & sulphates. This study covers the experimental investigation reflecting the effect of sodium chloride environments on the physical and mechanical strength of concrete.A total of 216 nos cubical 4 inch concrete specimens were cast and exposed to NaCl solutions of different concentration as well as in plain water over the period of 3 month. The test variables include three diffrent concrete mix (1:1.5:3, 1:2:4, 1:3:6), four different exposure environment (PW, 1T, 5T, 10T NaCl solution) and 5 diffrent exposer periods (7, 14, 28days, 3month). The specimens were tested for compressive strength after specific periods. The losses of compressive strength of the specimen are observed to lie in the range of 5% to 26%. Also the rich concrete mix (1:1.5:3) shows the minimum strength deterioration (5% to 15%) as compared to PW concrete of identical condition. The rate of deterioration is seen to increase with the increase in salt concentration of curing environment although the rate is not proportional.

Keywords: Concrete, Salt Attack, Deterioration, Compressive strength, Chlorides

1.INTRODUCTION:

The seas and oceans make up of about 80% of the total surface of the earth (Mehta, 1980). Apart from their use for navigation purposes, seas and their beds are being widely used as the space for both onshore/offshore structures throughout the world. The concrete structure constructed in sea environment has to face the sea salt loading apart from the normal mechanical loading. Sea water contains about 3.5% of salts of which the amount of NaCl is 2.7% (Mayers,1969). Chloride ions of sea salts have got detrimental effects on concrete strength and its durability. Such ions gradually penetrate into the body of concrete and form different expansive products. A chloride is known to be the most common agent for rebar corrosion in structural concrete. During construction of reinforced concrete structure in sea water environment, the harmful effects of sea salts are to be considered and the necessary measures to be taken to control the deterioration of concrete in such environment (Shetty, 1986). Concrete used as structural materials in many structures are reported to be chemically and physically durable against the external effects. Concrete and steel have been the most important building materials since the last century. Reinforced concrete structures are preferred for various kinds of structures because of their high tensile and compressive strength, mouldability for desired shapes and dimensions and cheapness. These kind of structures are practically advantageous because they are durable against adverse environmental effect and require less maintenance cost.

NaCl, the principal salts of sea water, play major roles in deterioration of structural concrete in marine exposer. Although the term “marine environment” is well understood but the complexities inherent in such an environment are not usually clear. The marine environment is not usually prevailing just over the sea but it could be deemed to be extending over the coast and the neighborhood of tidal cracks backwater estuaries(Indian Concrete Journal, 1990). Broadly it covers the area where concrete become wet with seawater and wherever the wind will carry saltwater spray which may be as far 1 km inland(Geymayr, 1980). Reinforced concrete structures located in such an environment are always subject to aggressive loading both physical and chemical in nature over their entire life span. It is therefore necessary to understand clearly the characteristics of the various aggressive agents causing distress, their nature, intensity of attack, the different zone order to get satisfactory performance of marine structure.

In most circumstances, especially in the context of seawater, the vast majority of dissolved solids are salts, and the terms *dissolved solids* and *salts* can be used interchangeably. Salinity is measured in grams of salt per kilogram of solution (g/kg), which can also be expressed as parts per thousand (ppt or ‰). Seawater contains a mixture of salts, the most abundant being sodium chloride (NaCl), or table salt. The oceans contain an average of 35 grams of salts per kilogram of seawater (35 ppt) (Mayers,1969). Sodium chloride constitutes about 85 percent of sea salts. Five more ions, sulfate (SO_4^{2-}), magnesium (Mg^{2+}), calcium (Ca^{2+}), potassium (K^+), and bicarbonate (HCO_3^-), account for the remaining salinity. These ions mostly float freely in the water. When water evaporates, the ions pair up with each other to form solid salts such as calcium sulfate (gypsum), potassium chloride, and sodium chloride. Along with these major ions, the oceans contain trace amounts of every element found in nature. Table 1(a) shows the salt contents and Table 1(b) shows the ionic concentration of salts in sea water. (Marchand,1999)

Table 1(a): Salt contents of Average Ocean water (Mayers, 1996)

Salt	Amount (gm)	% of total salt
NaCl	27.21	77.74
$MgCl_2$	3.81	10.89
$MgSO_4$	1.66	4.74
$CaSO_4$	1.26	3.60
K_2SO_4	0.86	2.46
$CaCO_3$	0.12	0.34
$MgBr_2$	0.08	0.23
Total	**35.00**	**100.00**

Table 1(b): Ion concentration of salts in sea water

Ions	Concentration (%)
Chloride(Cl^-)	2
Sodium(Na^+)	1.11
Sulphate(SO_4^{2-})	0.28
Magnesium(Mg^{2+})	0.14
Calcium(Ca^{2+})	0.05
Potassium(K^+)	0.04

From the Table 1, it is noticed that the amount of NaCl salt and its ion is governing salt water concentration of seawater.

Moreover, seawater is a dilute solution of several salts derived from weathering and erosion of continental rocks. Salinity varies from nearly zero in continental waters to about 41 parts per 1,000 in the Red Sea, a region of high evaporation, and more than 150 parts per 1,000 in the Great Salt Lake. In the main ocean, salinity averages about 35 parts per 1,000, varying between 34 and 36. The major cations, or positive ions present and their approximate abundance per 1,000 parts of water are as follows: sodium, 10.5; magnesium, 1.3; calcium, 0.4; and potassium, 0.4 parts. The major anions, or negative ions, are chloride 19 parts per 1,000, and sulfate 2.6 parts. These ions constitute a significant portion of the dissolved salts in seawater, with bromide ions, bicarbonate, silica, a variety of trace elements, and other inorganic and organic nutrients making up the remainder. The nutrients, although not abundant in comparison with the major ions, are extremely important in the biological productivity of the sea. Trace metals are of specific importance for certain organisms, but carbon, nitrogen, phosphorus, and oxygen are almost universally important to marine life. Carbon is found mainly as bicarbonate, HCO_3^-; nitrogen as nitrate, NO_3^-; and phosphorus as phosphate, PO_4^{3-}.The density of sea water is governed by its salinity of apart from the influence of temperature and pressure.

All gasses present in the atmosphere are found in the ocean among them Nitrogen (N),Oxygen (O_2) and Carbon Dioxide (CO_2) are important because of their physiological importance in the life of marine plants and animals(Marshall, 1990; Patel, 1989). In marine environment, the structural concrete in tidal zone undergoes alternate wetting and drying process due to wave and tidal action and is considered as the corrosive area(Gowda, 1981). At low and moderate concentration, chloride has relatively little disruptive effect on the volume stability of the concrete. However it destabilizes the passivating layers on concrete. At high concentration, chloride affects the stability of concrete due to formation of complex compounds including calcium choloroaluminates (Friedels Salt) together with excess soluble $CaCl_2$. Friedels salt has a property of low to medium expansion and the excess $CaCl_2$ which may leach out results is increased permeability of concrete (Ben-Yair, 1974). Chloride ions may introduce into cement pest via internal and external sources. Internal sources include mixing water, clinker, chloride-bearing aggregate. External sources may be due to the use of sodium chloride as de-icing salt and by absorption chloride from underground or sea water. The chloride ions do not form compounds with the calcium silicate hydrate even though they accelerate the hydration of the calcium silicate phases. Chloride reacts with calcium aluminate hydrate phases to form some complex chloride salts. The main objective of the study is to investigate the deterioration of concrete exposed to the effect of NaCl environment. The compressive strength of concrete in NaCl solution of different concentration were evaluated to compare the strength of concrete placed in normal and NaCl environment.

2. INVESTIGATION PROGRAM

With the specific aim stated above, the experimental investigation was carried out to study the effect of NaCl environment on the deterioration of concrete strength. The variable parameters and materials involved were as follows.

2.1 Material:

(a) Cement: Portland composite cement is generally used for concrete construction in Bangladesh. Portland composite cement (Confidence Brand) was used in making test specimen. Table 2 shows the properties of the cement used.

(b) Aggregate: Basalt, granite, quartzite, rocks are generally used as raw materials for coarse aggregate in marine construction due to their imperviousness, high strength and weather resisting characteristics. However, locally available brick khoa (broken brick chips) was used as coarse aggregate in the present study. The size of coarse aggregate was varied from ¼" to ¾" and F.M was 7.5. For marine concrete sand should be pure (SiO_2) and free from clay, organic matter and salts. Sylhet sand of FM 2.3 was used in the concrete mix.

Table 2: Chemical composition and physical properties of the Portland cement

Chemical composition (% by mass)			
SiO_2	20,36	Insoluble residue in HCl/Na_2CO_3	1,11
Al_2O_3	5,83	Insoluble residue in HCl/KOH	0,62
Fe_2O_3	2,96	CO_2	1,04
CaO	62,36	CaO, Free	0,43
MgO	1,32		
SO_3	2,80		
Na_2O	0,15	**Physical properties**	
K_2O	0,79	Residue on siev 0,09 mm / %	0,3
MnO	0,127	S_p / cm^2/g	4430
LOI	3,00	γ_s / g/cm^3	3,10
Sum:	99,70		

(c) Water: Water for making concrete should be free from deleterious chemicals. The available tap water was used for mixing of concrete.

2.2 Variable Studied:

a. Exposer condition: Four different exposer conditions was used i.e. normal water (PW), 1T NaCl solution, 5T NaCl solution and 10T NaCl solution

b. Exposer period: Four different exposer periods including 7 days, 14 days, 28 days and 90 days were used.

c. Mix ratio: Three different grades of concrete were used; rich mix (1:1.5:3), moderate mix (1:2:4) and lean mix (1:3:6).

A total of 144 Nos 4" cube specimens were cast from three different grades of concrete as per test schedule. The specimens were then demoulded after 24 hours and placed in the various NaCl environments (1T, 5T and 10T solutions). 1T means 2.7% NaCl solution and 5T and 10T solution contains respectively 5 and 10 times salts as that of 1T solution. Some specimens were also kept in normal water as control specimens to compare the results. After specific exposure periods, the specimens were tested for compressive strength and results were presented in graphical form.

3. RESULT AND DISCUSSION:

After conduction the compressive strength tests on the specimens exposed in normal and 1T, 5T, 10T NaCl solution upto 90 days, the experimental results are analyzed and presented in graphical form followed by the subsequent explanation and discussion. Fig. 1(a)-1(d) shows the compressive strength vs Exposure Period relation for different concrete specimens exposed to PW and NaCl solution of various concentration. On the other hand, Fig 2(a) to 2(d) presents the variation of compressive strength of specimen in different environmental condition. From the graph, it is seen that strength increase more sharply in between 7 to 28 days for each mix ratio and after this period it increases less rapidly. Strength of the concrete is observed to be higher for the mix ratio 1:1.5:3 and then 1:2:4 and minimum for 1:3:6 for each exposed period. Also the salt concentration of the exposure solution had a significant effect on initial strength gaining of concrete. The curves showing the increase in strength with respect to the exposure period is flat in normal water and it becomes steeper for 1T, 5T and 10T NaCl solution as the initial strength gain is higher for normal water. As the concentrations of the curing solution increases, the strength decreases although the rate of deterioration is not linear. All the curves are steep at starting and become flat with respect to the increase in concentration of NaCl solution. From these graphs it can be said that the maximum strength loss is observed for mix ratio 1:1.5:3, 1:2:4 and 1:3:4 at initial stage and reverse trend is noticed at mature stage.

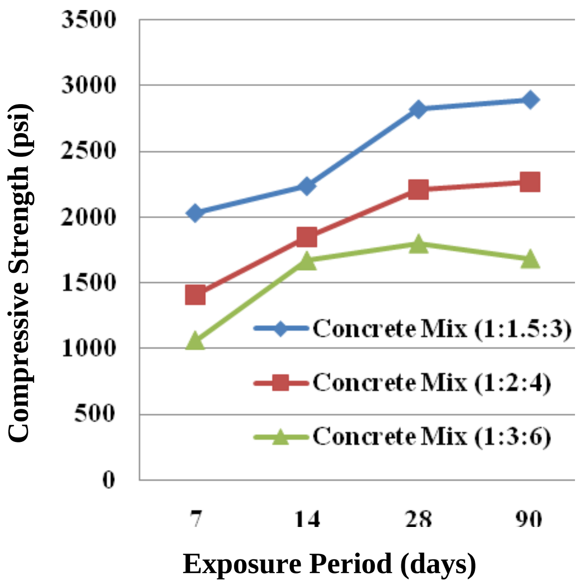

Fig. 1(a) Compressive strength vs exposure period relation for concrete exposed to PW

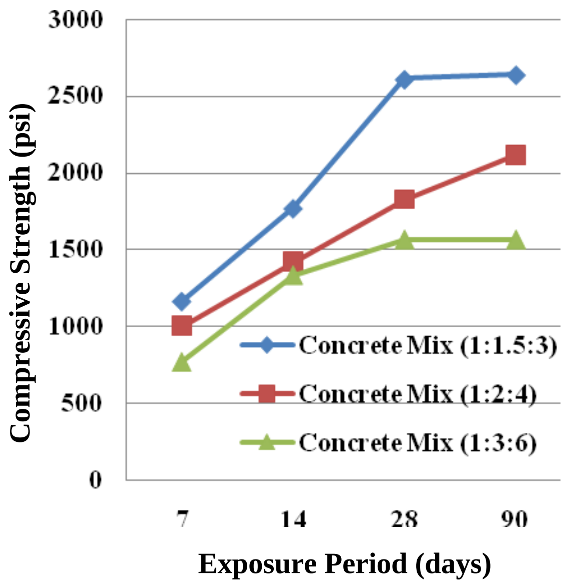

Fig. 1(b) Compressive strength vs exposure period relation for concrete exposed to 1T NaCl

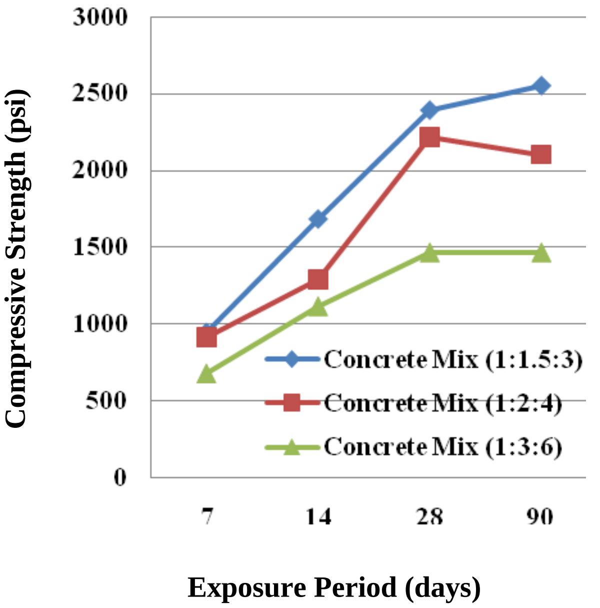

Fig. 1(c) Compressive strength vs exposure period relation for concrete exposed to 5T NaCl

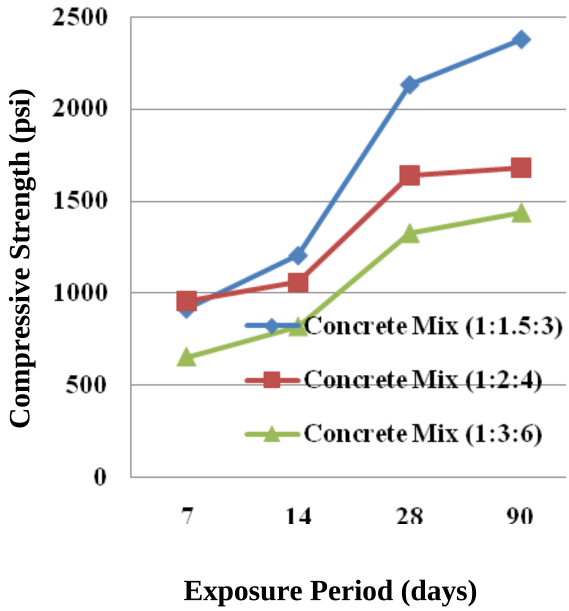

Fig. 1(d) Compressive strength vs exposure period relation for concrete exposed to 10T NaCl

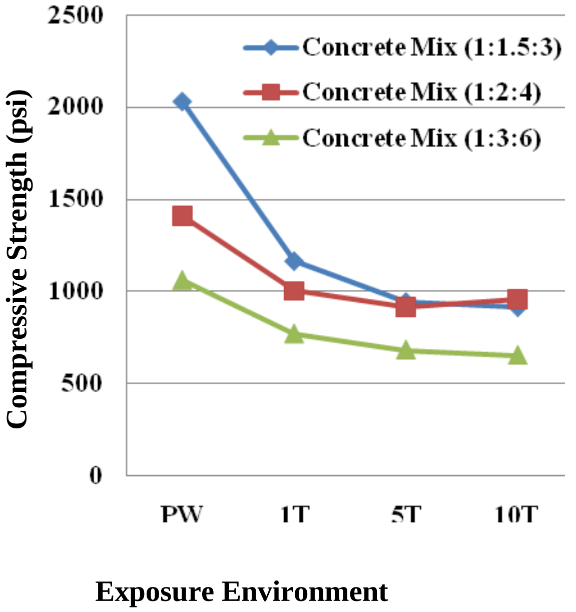

Fig. 2(a) Compressive strength vs salt concentration of environment relation (Age-7 days)

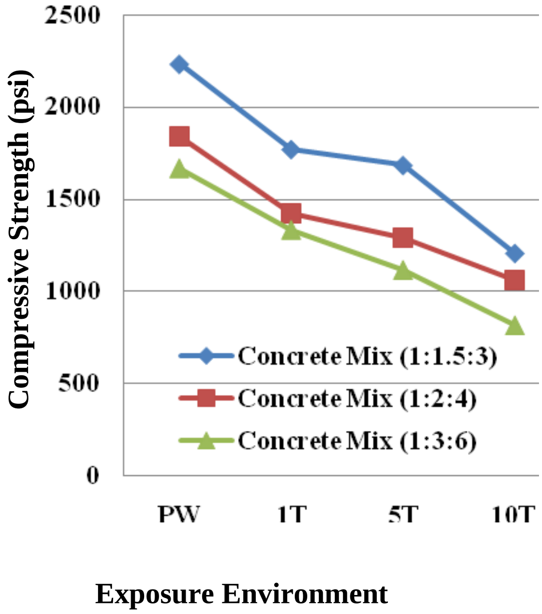

Fig. 2(b) Compressive strength vs salt concentration of environment relation (Age-14 days)

With the increase of exposure period, the curve becomes more flat which indicate that with the increase of exposure period, the deterioration effects of NaCl decreases. Fig. 3(a)-3(d) shows the relative strength vs salt concentration of environmental solution relation of the test specimen at different ages. The relative strength is determined by expressing the concrete strength of different environments as percentage of the corresponding strength of PW cured concrete. From this graph it is seen that maximum deterioration is occurred in 7 and 14 days strength of concrete ie at early ages as compared to 3 months exposure period.

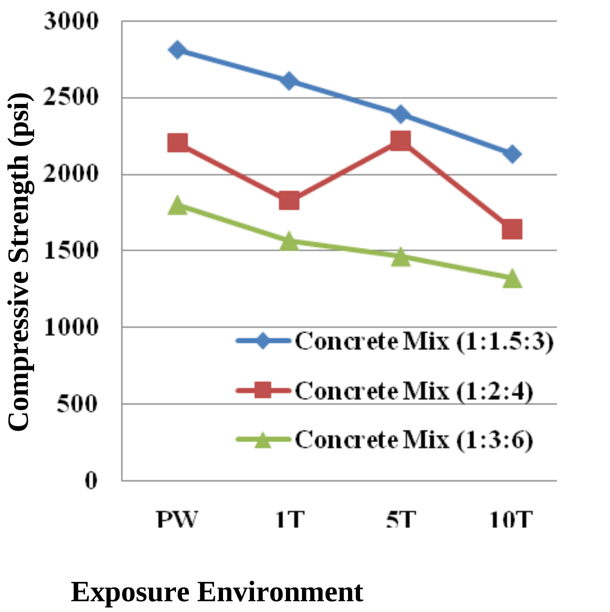

Fig. 2(c) Compressive strength vs salt concentration of environment relation (Age-28 days)

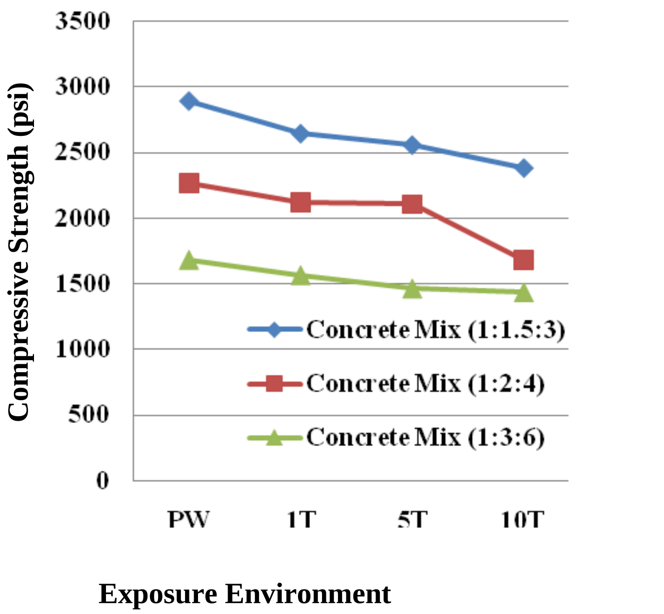

Fig. 2(d)Compressive strength vs salt concentration of environment relation (Age-90 days)

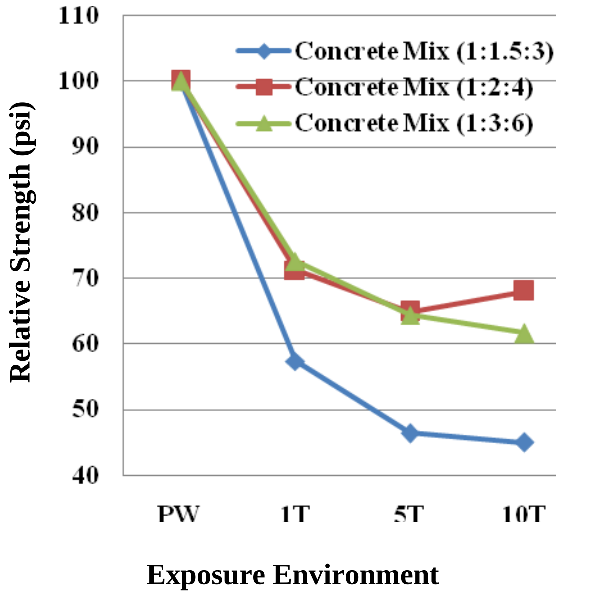

Fig. 3(a) Relative strength vs salt concentration relation (Age-7 days)

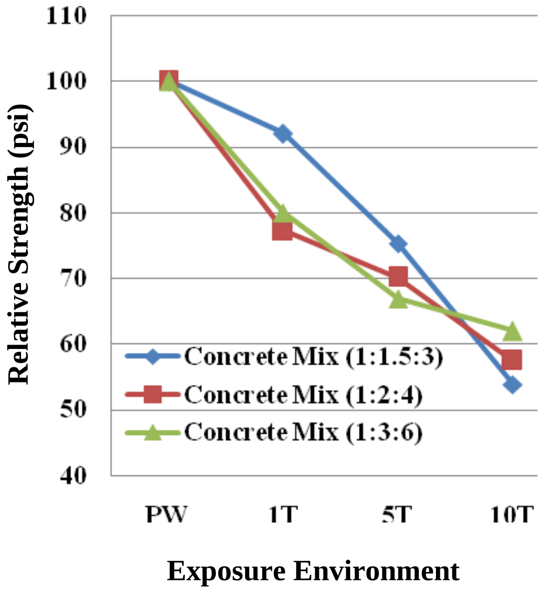

Fig. 3(b) Relative strength vs salt concentration relation (Age-14 days)

For example, for 7 days and mix ratio 1:1.5:3, the strength is reduced by 43.78% for 1T, 54.72% for 5T and 55.68% for 10T NaCl solutions as compared to the strength at 7 days in normal water. Similarly for mix ratio 1:2:4, the strength is reduced by 29.1% for 1T, 34.9% for 5T and 32.08% for 10T NaCl solution and for mix ratio 1:3:6, the strength reduction is 27.1% for 1T, 34.86% for 5T and 38.8% for 10T NaCl solution as compared to the respective strength at 7 days curing in normal water. On the other hand for 3 months curing and mix ratio 1:1.5:3, the strength is reduced by 8.57% for 1T, 11.45% for 5T and 17.18% for 10T NaCl solution as compared to the strength at 3 months in normal water.

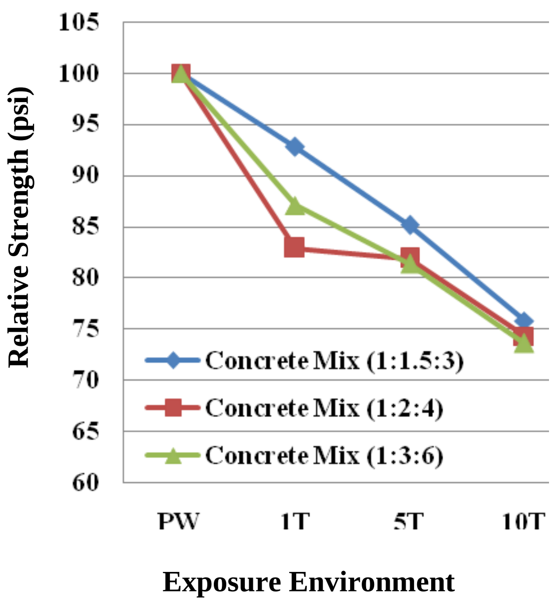

Fig. 3(c) Relative strength vs salt concentration relation (Age-28 days)

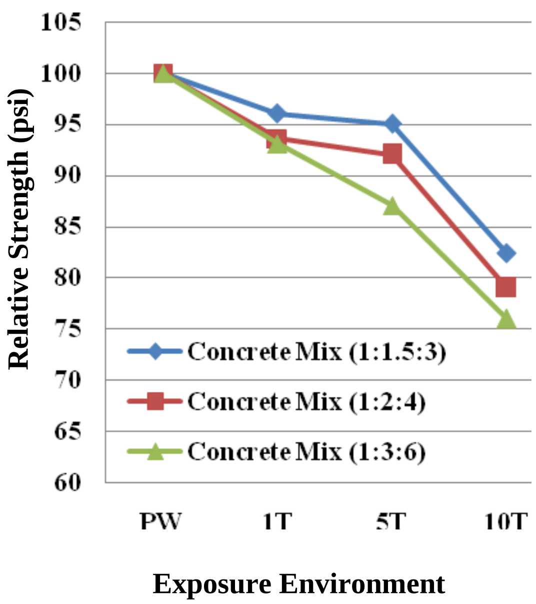

Fig. 3(d) Relative strength vs salt concentration relation (Age-28 days)

For mix ratio 1:2:4, the strength is reduced by 6.45% for 1T, 7.34% for 5T and 25.60% for 10T NaCl solution and for mix ratio 1:3:6, the strength reduction is 7.31% for 1T, 13.48% for 5T and 14.73% for 10T NaCl solution as compared to the respective strength at 3 months in normal water.

4. CONCLUSION:

On the basis of the limited number of test specimen, curing environment and test conducted on 4 inch concrete cube specimen exposed to different NaCl exposure environment, the following conclusion can be drawn:

1.The relatively rich concrete (mix ratio 1:1.5:3) shows maximum strength loss (25% to 55%) as compared to other concretes at early stage (up to 14 days) in NaCl environment.

2. After 3 months exposure in NaCl environment, the compressive strength losses are observed to lie in the range of 5%

3.The adverse effect of NaCl environment is observed as maximum in the initial stage of curing as the maximum deteri

4.Concrete gains strength at faster rate in early ages (7 to 28 days) and then slows down in the later exposure period.

5.The rate of deterioration is seen to increase with the increase in salt concentration of curing environment although the rate is not proportional.

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EFFECT OF SPECIMEN SIZES ON THE COMPRESSIVE STRENGTH OF CONCRETE

M. K. Miah[1*], M. M. Rahaman[1], M. R. Karim[1] & M. J. Mian[1]

[1] *Department of Civil Engineering, Dhaka University of Engineering & Technology, Gazipur-1700, Bangladesh.*
**Corresponding Author, <mkhasro@duet.ac.bd>*

ABSTRACT

The compressive strength of concrete is one of the most important properties in reinforced concrete design. Concrete structures are designed on the basis of 28days crushing strength of standard cylinder as well as smaller sized cylinder. 28days cylinder strength actually represents the characteristic strength of the concrete. It is mandatory to test the concrete cylinders at the age of 28 days as per almost all building code requirements. Though it is quite time consuming to wait 28 days for such tests, it is important to continue the construction work and ensure the quality control process. It has become a problem to use the strength of concrete because the sizes of control specimen vary from country to country. The objective of this research is to develop a correlation between concrete strength obtained from smaller size (ϕ100mm×200mm) to standard size (ϕ150mm×300mm) cylinders at different target concrete strength, curing age and slump values. In this regards a series of concrete cylinders were cast and tested. ACI mix design method was used to calculate the mix ratio having target strengths of 20, 24, 28, 31 and 35MPa. Two slump values of 25mm to 50mm and 75mm to 100mm were selected. The result reveal that the compressive strength of concrete obtained from ϕ100mm×200mm sizes cylinder is 98.50%, 98% and 95% to that of ϕ150mm×300mm sizes cylinders based on different slump values, strength of concrete and curing age respectively. Furthermore, smaller sizes cylinders could be used instead of standard sizes cylinders to determine the concrete compressive strength.

Keywords: Compressive strength, specimen size, slump values, curing age

INTRODUCTION

In the developed and developing countries all over the world, the major part of the construction work is done by concrete and concrete strength is important for the durability as well as longevity of the structure. The main measure of the structural quality of concrete of its compressive strength. Structural concrete elements are generally made with concrete having a compressive strength of 20MPa to 35MPa. In Bangladesh, the compressive strength of concrete is usually measured by 150mm cube and ϕ150mm×300mm standard cylinder. But the ACI Code specified that the compressive strength is measured based on the standard size cylinder of ϕ150mm×300mm. In most structural application, concrete is employed primarily to resist the compressive stress. In those cases where strength in tension or in shear is of primary importance, the compressive strength is frequently used as measure of these properties. Therefore the concrete making properties of various ingredient of mix are usually measured in terms of the compressive strength. As soon as the component of concrete have been mixed together, the cement and water react to produce a cementing gel that's bond the fine and coarse aggregates into a stone like material. The chemical reaction between cement and water exothermic reaction producing heat, is termed hydration.

Compressive strength is also used as a qualitative measure for other properties of hardened concrete like durability. The 28days strength is the most common practice employed to determine the concrete strength. To ensure the strength of hardened concrete, ϕ150mm×300mm (ϕ6"×12") cylinder is normal practice to do so. But in this case there are some problems such as placement, materials cost, labor cost and others. To overcome of these problems a number of researchers measured and compared the concrete strengths of standard and smaller sizes cylindrical specimens. Carrasquillo et al. (1981)

reported that the average ratio of compressive strength of ϕ150mm × 300mm to ϕ100mm × 200mm cylinders was 0.9 regardless of strength and test age. A contradiction to this finding was later reported by Carrasquillo and Carrasquillo (1998) which reported that compressive strength of ϕ100mm × 200mm cylinders were 7 percent lower than those of ϕ150mm×300 mm cylinders.

Nasser and Al-Manasser (1987) carried out tests to compare the compressive strength of ϕ75mm×150mm (ϕ3"×6") cylinder rodded 10, 15, 20 or 25 times in two layers with that of ϕ150mm×300mm (ϕ6"×12") cylinder rodded 25 times in three layers. They suggested that the ϕ75mm×150mm cylinder rodded 10 times in equal layer is satisfactory specimen to determine the potential strength of concrete and use greater number of specimens per test to cater wider scatter of result in a smaller size of samples. Day and Haque (1993) investigated the influence of specimen size on the measured compressive strengths of plain fly ash concretes exposed to slander and cold curing strength test were performed on three classes of air-entrained concrete using two sub-bituminous fly ashes three replacement levels. Analysis of the present experimental result presented in the literature, show that the compressive strength indicated by ϕ75mm (ϕ3") cylinder is identical that indicate by ϕ150mm (ϕ6") cylinder. Analysis suggests that this one-to-one relationship between strength of ϕ75mm (ϕ3") and ϕ150mm (ϕ6") cylinder may be valid for concrete strength up to 7250psi.

French and Mokhtarzadeh (1993) observed in their study that on average ϕ100mm × 200mm cylinders tested showed 6 percent higher strength than that of its companion of ϕ150mm × 300 mm cylinders. Aïtcin et al. (1994) reported that larger cylinder sizes gave rise to lower apparent compressive strength and that compressive strength is not sensitive to cylinder size for very high strength concrete. For comparison between compressive cube strength and compressive cylinder strength (diameter/height = 1/2), a factor of 0.8 to the cube strength is often applied for normal strength concrete (FIP-CEB, 1990). The same reference also cited a study that the cylinder/cube compressive strength ratio is not only a function of the strength grade but also of the mix design parameters. In a recent study Al Sayed (1997) reported that the ratio of 0.8 that is applied for normal strength concrete remains the same for high strength concrete.

Issa et al. (2000) found that the ratio of compressive strength of ϕ100mm×200mm sizes cylinders to that of the ϕ150mm×300mm sizes varied from 0.97 to 1.08 depending on the types of coarse aggregates used and curing days. Alaa S. M. (2003) reported that the ratio of compressive strength of ϕ150mm×300mm to ϕ100mm×200mm cylinders was 0.86 based on different high strength ready mixed concrete and age of curing of 28 days. Elfahal (2003) investigated size effect phenomenon under compression static and impact loads for both normal and high strength concrete cylinder having sizes of ϕ75mm×150mm (ϕ3"×6"), ϕ150mm×300mm (ϕ6"×12"), ϕ300mm×600mm (ϕ12"×24") and ϕ600mm×1200mm (ϕ24"×48"). This research was conducted by performing 127 compressive static and impact tests on both normal and high strength concrete and on the effect of loading rate and material strength on the size effect for structural concrete in compression.

Zia et al. (2009) reported that the ratio of compressive strength of ϕ150mm×300mm sizes cylinders to that of the ϕ100mm × 200mm sizes varied from 0.91 to 0.98 depending on the types of coarse aggregates used. In this research work results of an ongoing research on normal strength concrete were presented. A comparison of the compressive strength between ϕ100mm × 200mm and ϕ150mm × 300mm cylinders was performed. These sizes were chosen because are of most commonly used locally in the construction industry and research. The research work proposed the compressive strength factors between the different specimen sizes used during the study.

MATERIALS AND METHODS

An experimental investigation was carried out on a series of concrete cylinders. For doing so, over three hundred (300) concrete cylinder specimens were cast including the standard ones and tested. ACI mix design was used to calculate the mix ratio for the five target concrete strengths of 20, 24, 28, 31 and 35MPa and two slump values of 25-50mm and 75-100mm respectively. The two different cylinder sizes were ϕ100mm × 200mm (ϕ4"×8") and ϕ150mm × 300mm (ϕ6"×12") respectively were selected. Five water cement ratio viz. **0.680**, **0.625**, **0.570**, **0.525** and **0.480** respectively used. Mix proportion for slump values of 25mm to 50mm are **1 : 2.96 : 3.91**, **1: 2.56 : 3.56**, **1 : 2.32 : 3.27**, **1 : 2.06 : 3.02** and **1 : 1.80 : 2.76** used. Similarly mix proportion for slump values of 75mm to 100mm are **1 : 2.62 : 3.62**, **1 : 2.29 : 3.29**, **1 : 2.04 : 3.03**, **1 : 1.80 : 2.80** and **1 : 2.56 : 3.55** used.

The concrete mix consisted of ordinary Portland cement, fine aggregate and coarse aggregate. Sand was used as fine aggregate and brick chips were used as coarse aggregate. Locally available Ordinary Portland Cement brand name ″**Shah Cement**″ was used as binding material for the preparation of fresh concrete. The initial and final setting times were used 47 minutes and 4 hours 45 minutes respectively. It was free from lumps, adulteration and /or partial setting. Three cube specimens of 50.8mm size were tested as per ASTM C109-84 (1984) to get compressive strength of cement used in casting of test specimens. The standard sand recommended by ASTM C778-84 (1984) was used for making cement and mortar. The 7-days standard cube strength was 20MPa (2900psi). Unit weight of cement was 1475kg/m^3 (92pcf).

Ordinary Sylhet sand passing # 4 sieve was used as the fine aggregate in preparing the specimens. The sand was sieved to screen out foreign particles through sieve #4. The sand was washed with water and air dried before used. The unit weight was 1540kg/m^3 (96.15pcf) and specific gravity under SSD condition was 2.62. The fineness modulus of fine aggregate was determined following ASTM C 136-88 recommendations (1988) and it was 2.62. For making cylinder specimens, 20-mm down well graded hand broken brick chips was used as course aggregate. The unit weight and specific gravity under Saturated Surface Dry (SSD) condition of the brick chips were 1462.65kg/m3 (91.3pcf) and 2.68 respectively. The coarse aggregate was properly washed to remove dust and dried up before using in the preparation of concrete. Water used for casting purpose was free from impurities like oil, grease, organic and chemical matters. Water available in the laboratory is of portable quality which is supplied from deep tube well of DUET and is known to have no unusual impurities.

The cylindrical mould placed on firm and levelled floor on concrete laboratory. Lubricating oil was used to smear the inside of the moulds for its easy removal after hardening of the concrete. Fresh concrete was prepared as per designed mix in a drum type mixture machine. Immediately after unloading from mixture machine, the fresh concrete was placed in the mould in three layers and compacted by using ϕ16mm (ϕ5/8") diameter and 600mm (24") in length tampering rod with hemispherical tip for the standard cylinders. For the cylinders smaller than the standard ones, tampering rod having diameter of ϕ10mm (ϕ3/8") and length of 300mm was used. In all cases the number of blows were 25 per layer. Proper compaction was ensured over the cross-section of the mould through uniform.

The curing conditions for the test cylinders are specified by ASTM C 192-90a. The cylindrical moulds were removed after 24-hours and shift the concrete cylinders carefully to the place of curing i.e. the water bath and stored for the curing period at 7days, 14days, 21days and 28days designed accordingly. The top and bottom surfaces of the cylinders were finished and smooth enough for testing, and so requires further preparation. ASTM C 617-87 requires the end surface to be plane within 0.05mm (0.002 in), a tolerance which applies also to the platens of the testing machine. The both surfaces of the cylinder was smoothed by the grinding machine. After the specimens have been cured for the proper length of time in the water bath then it removed for smoothening the surfaces then it placed under the universal testing machine in order to determining compressive strength of cylinder. The compressive strengths of all sizes of cylinders are determined according to ASTM C 39-86. The compressive load was applied at a constant load rate of 0.15 to 0.34 MPa/s (20 to 50 psi/s) for hydraulically operated machines having capacity of 1000kN.

RESULTS AND DISCUSSIONS

Testing of harden concrete play an important role in controlling and confirming the quality of cement concrete work. Two different sizes of cylinders viz. ϕ100mm× 200mm (**smaller size**) and ϕ150mm×300mm (**standard size**) were cast and tested after curing the specimen accordingly. About

300 cylinders in 10 batches having different mix ratios were cast. The batches were defines in term of concrete strength, mix proportion and slump value of concrete. Five different water cement ratios and ten mix proportions with respect to two slump values were selected for the experimental investigation.

The test results were categorised for making correlation between the results of smaller and standard sizes cylinders based on slump values (25mm-50mm and 75mm-100mm) considered during design the mix ratios. Fig. 1 indicates the relation between smaller and standard sizes cylinder for slump value 25-50mm. The abscissa defines the concrete compressive strength of smaller sized cylinders (ϕ100mm× 200mm) and ordinate defines the compressive strength of standard sized cylinders (ϕ150mm× 300mm). The relation includes all the test results of concrete cylinders corresponding to the target strength of 20, 24, 28, 31 and 35MPa at curing ages of 7, 14, 21, 28 and 60days. Fig. 2 is the same relation for slump value 50-100mm. Regression lines are plotted for both the relations and coefficient of variances also computed to observe the correlation among the plotted data. The coefficient of variances in Fig. 1 and Fig. 2 are shown as 0.9822 and 0.9859 which are very close to unity. Theses closeness of variance to unity indicate the concrete compressive strength of smaller sized cylinders is almost similar to the compressive strength of standard sized cylinders. To overcome the much materials for the casting of heavy and weighted ϕ150mm× 300mm sized cylinder, smaller one of ϕ100mm× 200mm sized cylinder may be well accepted for measuring the concrete strength at different curing ages.

Fig. 3 and Fig. 4 show the typical relations of relation between smaller and standard sizes cylinder for all slump values under target strength of 20MPa and 28MPa at curing ages of 7, 14, 21, 28 and 60days respectively. The coefficient of variances in Fig. 3 and Fig. 4 are observed as 0.9708 and 0.9758 respectively. The plotted data are agree with each other means the deviation among the plotted individual results is insignificant. So, it can be concluded that the smaller sized cylinder crushing results may by used for predicted the concrete strength of ϕ150mm× 300mm sized cylinder at different curing ages.

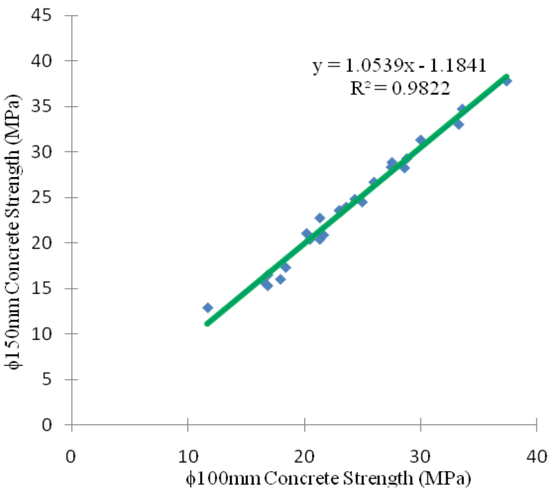

Fig. 1 Relation between smaller and standard sizes cylinder for slump value 25-50mm

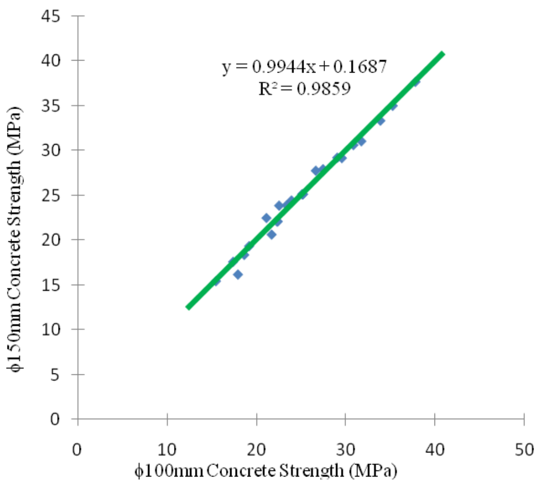

Fig. 2 Relation between smaller and standard sizes cylinder for slump value 50-100mm

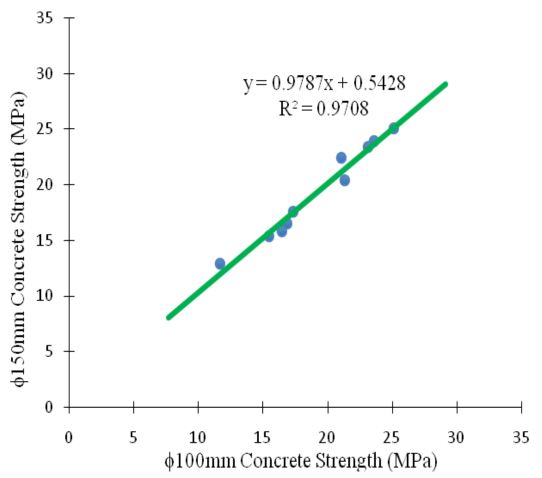

Fig. 3 Relation between smaller and standard sizes cylinder having target strength of 20MPa

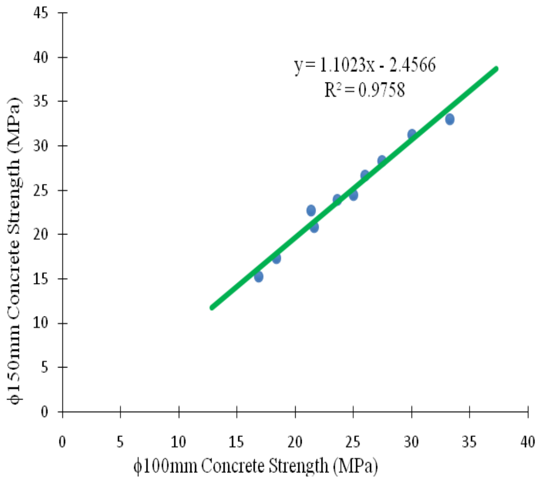

Fig. 4 Relation between smaller and standard sizes cylinder having target strength of 28MPa

CONCLUSION

The test results reveal that the compressive strength of concrete obtained from smaller sizes cylinder is about 98%, 95% and 98.5% of standard ones based on different assumed concrete strength, curing age and slump values respectively. Therefore, it can be concluded that the effect of specimen sizes on concrete compressive strength is not so significant varies from 2% to 5% and on average of 3.5% in between standard and smaller sizes cylinders. Furthermore, the conclusion on smaller sizes cylinders could be used to determine the concrete compressive strength instead of standard sizes cylinders.

ACKNOWLEDGMENTS

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SEISMIC DAMAGE ESTIMATION FOR SYLHET CITY CORPORATION AREA USING RADIUS

R. K. Mazumder[1*] & A. R. Bhuiyan[1]

[1] *Institute of Earthquake Engineering Research, Chittagong University of Engineering and Technology, Chittagong 4349, Bangladesh.*
**Corresponding Author, <rkmazumder@cuet.ac.bd>*

ABSTRACT

In the proposed seismic draft code, Sylhet city is situated in the zone number 4 with a Peak Ground Acceleration (PGA) value 0.36 g. The city and its surroundings areas have a long history of Earthquakes. Frequently occurring recent small to moderate size earthquakes surroundings this region make us aware about the future risk. The present study aims to identify the risk status of the metropolitan area using Risk Assessment tools for Diagnosis of Urban areas against Seismic disasters (RADIUS). The primary database was collected from the Comprehensive Disaster Management Program (CDMP).

Keywords: Sylhet, RADIUS, Seismic Damage

INTRODUCTION

Earthquake is the most deadly natural phenomena that strike without any warning. Even a small size earthquake can cause severe damage to the vulnerable structures. Many of the earthquake risk zones coincide with the densely populated urban areas. Bangladesh is one of the densely populated countries which is situated in southern part of Asia. The country is located close to the boundary of Eurasian plate in the north and Indo-Australian plate in the east. The country and its surrounding areas have a long history of earthquakes and north eastern part of the country is considered as most earthquake prone region. Sylhet is the largest urban settlement in the north eastern part, which is on the eastern part of the Dauki fault. The Dauki fault ruptured in AD 1548 and the 1897 great Indian earthquake ($M \geq 8.0$; Yeats et al., 1997) was caused by the rupture of the Dauki fault (Morino et al., 2014). There has no evidence of moderate to large earthquake after this event meaning that possibility of huge energy can be stored near this region. At present Sylhet City Corporation (SCC) area has a population about 500,000 people. It is important to understand the possibility of future loss for different earthquake scenario. In this study, Geographic Information System (GIS) based risk assessment tool RADIUS has been used to estimate probable seismic damage for the Sylhet city corporation area considering different earthquake scenario. This study aims to facilitate preliminary estimation of earthquake damage so as to raise awareness of earthquake risks in the city. This tool cannot be considered for accurate engineering analysis.

RADIUS TOOL

Risk Assessment tools for Diagnosis of Urban areas against Seismic disasters (RADIUS) was developed in 1996 under the support of International Decade for Natural Disaster Reduction (IDNDR 1990-2000) of the United Nations to promote seismic disaster risk reduction activities in urban areas, particularly in developing countries (RADIUS, 2000). This is a simple seismic damage assessment tool. The main objective of using RADIUS tool is to raise awareness and understanding of earthquake disasters and methodology of damage estimation in the earthquake prone areas.

STUDY AREA

Sylhet City Corporation (SCC) is the main center of Sylhet division and is located at 24° 32′ 0″ N, 91° 52′ 0″ E, on the northern bank of the Surma River. The total area of Sylhet district is about 3,490 sq. km. The physiographic of Sylhet comprises mainly of hill soils, encompassing a few large depressions. The SCC has a total area of 27 sq. km. It has a population of 479,837 and 27 administartive wards. In proposed BNBC draft code, this region falls within the most earthquake prone zone with PGA value 0.36g.

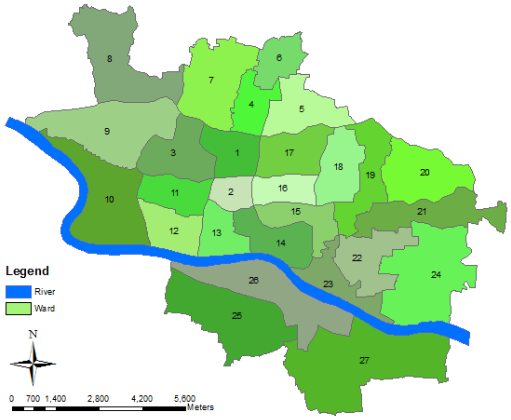


Figure 1: Ward map for Sylhet City Corporation area

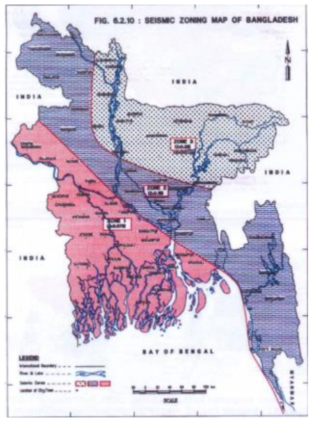


Figure 2: Seismic Zoning Map of Bangladesh (BNBC, 1993)

METHODOLOGY

The GIS based RADIUS has three basic stages named basic input parameters, constant data and generated outputs. RADIUS results can be represented in GIS maps to show the distribution of damage states. Figure 3 shows the input parameters and constant data that required estimating seismic damage for an urban area.

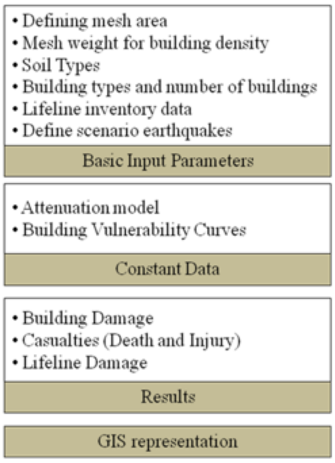


Figure 3: RADIUS approach for damage estimation.

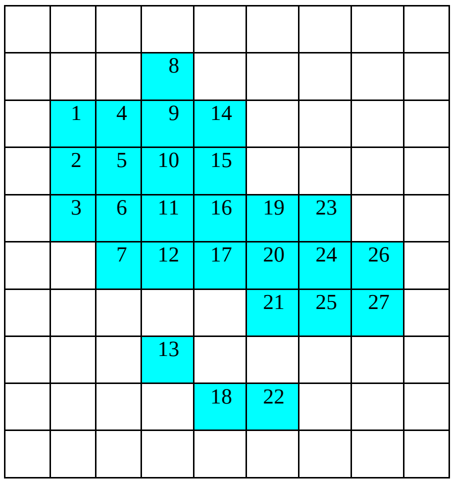
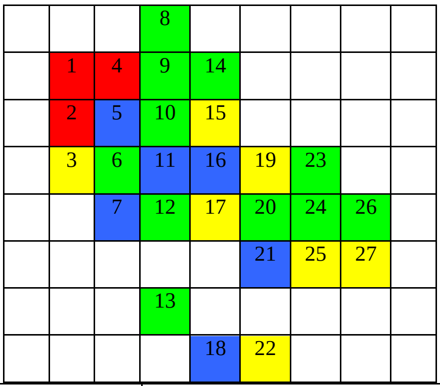


Figure 4: Mesh map generated for SCC area

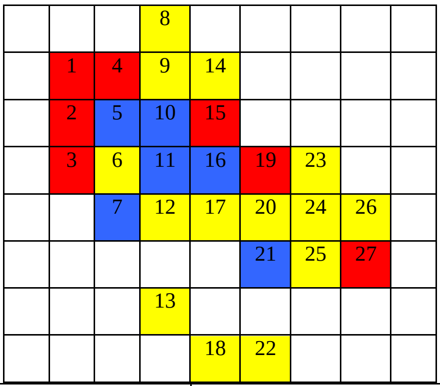
Define study area and Mesh generation

The study area needs to subdivide into equal sized square grids in order to carry out damage analysis. Sylhet city corporation area has been divided into 27 equal numbers of square grids where each grid size is defined 1 square kilometre. Then the input values are assigned to each of the grids to obtain spatially distributed outputs in RADIUS. Figure 4 shows shape of SCC area by meshes in RADIUS.



Color ID		Number of population
a		7966 - 13468
b		13469 - 18969
c		18970 - 24470
d		24471 - 29972

Figure 5: Distribution of total population



Color ID		Number of buildings
a		1353 - 1692
b		1692 - 2030
c		2030 - 2368
d		2368 - 2708

Figure 6: Distribution of buildings

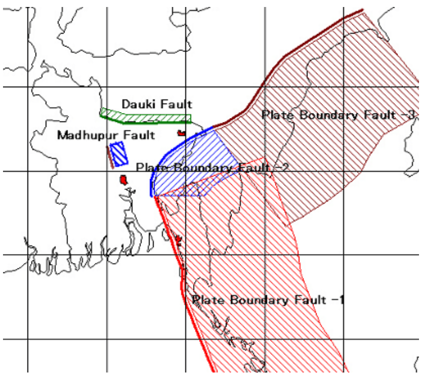
Demographic and Building Information

Distributed total population (in night-time) of the study area is shown in Figure 5. The RADIUS needs to input building types and number of buildings for mesh by mesh. RADIUS tools calculate night-time population and day-time population by predefined factors in the tool. For this calculation the tool consider respective mesh wight for relative population-building density and duration of day-night hours of a specific day. Total population for the study area is 479,837 and most of the mesh contains high ranges of population. During peak hour day-time population in the area is 284,177 whereas number of population is the same as total population during mid-night. The higher density area in mid-day indicates the concentration of commercial area. Figure 6 shows distribution of buildings map for study area.

Soil Type and Life line inventory

Five ground classifications have been adopted in the RADIUS tool based on the surface soil. They are termed as hard rock, soft rock, medium soil, soft soil and unknown soil. Soil type distribution map was generated based on recent study done by Comprehensive Disaster Management Program (CDMP, 2009b). Lifeline inventory includes road network, bridges, tunnels, electrical and telecommunication transmission towers, sub-stations, water network, gas network etc. These data were taken from CDMP 2009b study.

Earthquake Scenarios



Case	Description	M_w	Depth to top of fault (km)	Dip Angle	Fault type
1	Dauki Fault	8.0	3	60°	Reverse
2	PDF-3	8.3	3	30°	Reverse
3	beneath city	6.0	15	90°	Reverse

Figure 7: Existing Seismic faults for Bangladesh (CDMP, 2009b)

Figure 8: Earthquake Scenario Parameters for SCC area

In recent years, Ministry of Food and Disaster Management has taken up an initiative to evaluate probable seismic risk under CDMP. In this framework five earthquake scenarios were proposed based on seismic hazard assessment study carried out by OYO International Corporation. Each scenario was set as a maximum possible earthquake occurring within a fault zone, and there are five major fault zones (as shown in Figure 7), i.e. Madhupur fault (MF), Dauki Fault (DF), Plate Boundary Fault -1 (PBF-1), Plate Boundary Fault -2 (PBF-2) and Plate Boundary Fault -3 (PBF-3). In addition to five scenarios, a special earthquake scenario beneath specific city was also recommended. Earthquake scenario parameters for SCC area are given in Figure 8.

Constant Data

Two types of seismic intensity scales are used in RADIUS. These are Modified Mercalli Intensity (MMI) and Peak Ground Acceleration (PGA). PGA converted to MMI using the empirical formula of Trifunac and Brady (1975). The MMI will be calculated by the following formula MMI= $[\log_{10}(PGA \times 980)]/0.3 - 0.014$ where PGA is expressed in terms of gravitational acceleration g. Three widely used attenuation equations are used for RADIUS study (see table 1).

Table 1: Attenuation equations

ID	Source	Attenuation Equation
1	Joyner & Boore (1981)	$PGA = 10^{(0.249 X M-Log(D)-0.00255 X D-1.02}$, $D=(E^2+7.3^2)^{0.5}$
2	Campbell (1981)	PGA = 0.0185 X Exp(1.28*M)*D^(-1.75), D=E+0.147 X Exp(0.732 X M)
3	Fukushima & Tanaka (1990)	PGA = $(_{10}$ (0.41 X M - LOG10 (R + 0.032 X 10 $^{(0.41 X M)}$) - 0.0034 X R + 1.30))/980
E = Epicentral distance, R = Hypocentral distance, M = Earthquake Magnitude		

Table 2 represents the building types used in RADIUS program. Figure 9 and Figure 10 show physical vulnerability curves for buildings and lifeline, respectively.

Table 2: Classifications of building types

RES1	Residential 1	Informal construction: mainly slums and row housing made from unfired bricks, mud mortar, loosely tied walls and roofs
RES2	Residential 2	Unreinforced Masonry (URM) - Reinforced Concrete (RC) composite construction: substandard construction, not complying with the local building code provisions (height up to 3 stories).
RES3	Residential 3	URM - RC composite construction: old, deteriorated construction, not complying with the latest building code provisions (height of 4–6 stories).
RES4	Residential 4	Engineered reinforced concrete construction: newly constructed multistory buildings for residential and commercial purposes.
EDU1	Educational 1	School buildings, up to 2 stories: generally, the percentage of this type of building should be very low.
EDU2	Educational 2	School buildings higher than 2 stories: office buildings should also be included in this class; generally, the percentage of this type of buildings should be very low.
MED1	Medical 1	Low to medium rise hospitals: generally, the percentage of this type of building should be very low.

MED2	Medical 2	High rise hospitals: generally, the percentage of this type of building should be very low. Commercial Shopping centers. Industrial facilities.
COM	Commercial	Shopping Centers
IND	Industrial	Industrial facilities, both low and high risk

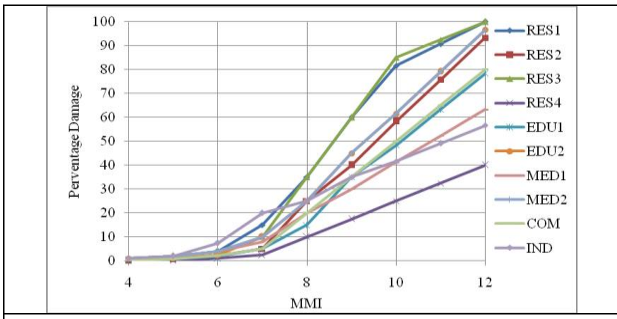


Figure 9: Physical vulnerablity curves for buildings.

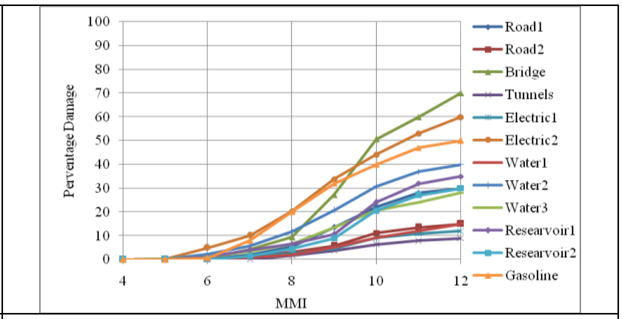


Figure 10: Physical vulnerablity curves for lifeline.

RESULTS

In this study, earthquake scenarios generated from Dauki fault and beneath the city were considered. The time of occurrence were taken as 12pm (mid-day) and 12 am (mid-night) in order to develop damage scenarios for day and night (respectively). Figure 11 and Figure 12 show the results obtained from RADIUS analyses for earthquake scenarios generated from Dauki fault and beneath the city area. It has been revealed that an earthquake generated by Dauki fault can create worse damage scenario. Significant losses found for the electrical sub stations, gas network, sewerage system and major bridges among the lifeline parameters for both scenarios.

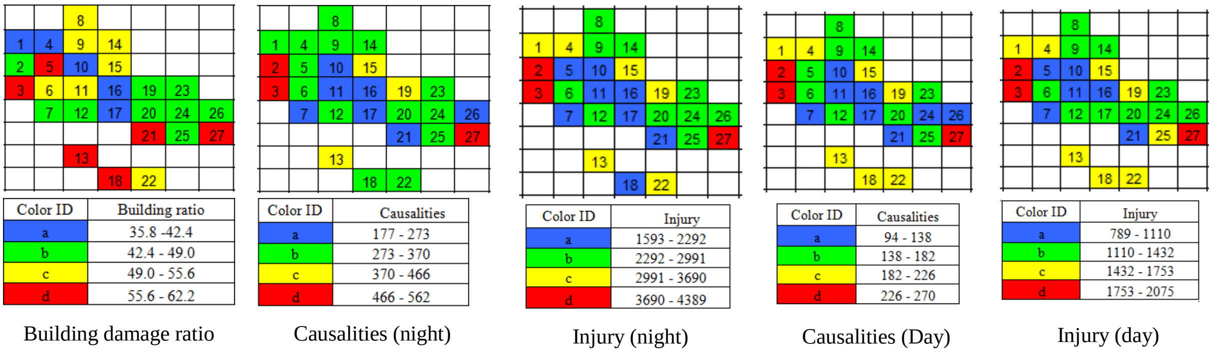


Figure 11: RADIUS output for earthquake scenario generated from Dauki fault

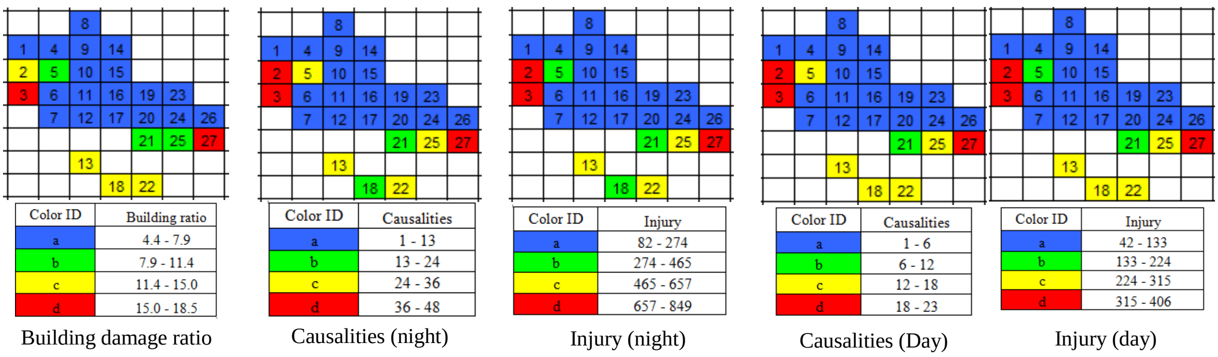


Figure 12: RADIUS output for earthquake scenario beneath the city.

CONCLUSION

The program was developed with simplified methodologies, pursuing speed and ease-to-use and its functions accuracy are limited. However, this is a very effective tool for earthquake emergency planning for an urban community.

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SEISMIC PERFORMANCE EVALUATION OF SEISMICALLY ISOLATED REINFORCED CONCRETE BUILDING: A CASE STUDY

M. A. Hasan[1*] & A. R. Bhuiyan[2]

[1] Department of Disaster & Environmental Engineering, CUET, Chittagong, Bangladesh.
[2] Department of Civil Engineering, CUET, Chittagong, Bangladesh.
*Corresponding Author, <hasanrazu0601056@gmail.com>

ABSTRACT

Damage of existing structures due to the frequent occurrence of earthquakes around the world has highlighted the need for studying the seismic performance of existing structures especially for the emergency services structures. For the safety and intensive care of the people the hospital building structures must remain functioning at all times even after major earthquake. But most of the hospital building constructed before the implementation of BNBC-1993 did not consider the load from earthquake. Bangladesh national building code has been revised recently to consider the greater effect of earthquake than previous version. So the adequacy of hospital buildings against earthquake loads constructed after 1993 should be checked with the revised code. The objective of this research is to evaluate the seismic performance of an existing hospital building. In this study the Chittagong Medical College Hospital Building (CMCHB) was considered. Two analytical models of the hospital building were generated in structural software SAP 2000 for comparison: existing CMCHB and base isolated CMCHB. After that static and dynamic analysis of the two structural systems were performed to compute the structural responses. The analytical results show that story drift as a considered response reduced 68 % in case of base isolated hospital building. It was also estimated that reinforcement requirement in column in the base isolated structure is reduced by 37.11 %.

Keywords: Base Isolation, Hospital Building, Response Spectrum and Seismic Performance.

INTRODUCTION

Earthquake is one of the most common natural hazards which lead serious damage in the building. The building constructed without any lateral load resisting features is very vulnerable during an earthquake. An earthquake can easily create damage to any vulnerable structures and able to stop its operational system. Most of the existing building in Bangladesh had been constructed before 1993 without any earthquake resisting features. It is important to evaluate the performance of existing building especially emergency services like hospital building. Hospital buildings need to be operational to serve society during a disaster to reduce its impact. In past earthquakes, it has been observed that casualties have been significantly increased due to the collapse of hospital building. For the hospital buildings, a notable example could be found in 2001 Bhuj Earthquake where a 281 bed hospital building was collapsed, as a result medical service stopped immediately after the earthquakes. For this reason the injured people died due to lack of medical treatment. To avoid such kind of situation most of the building codes emphasise more for lifeline services (hospital, medical clinic, fire service etc) in seismic design. The Bangladesh National Building Code (BNBC, 1993) recommended for considering 25 % increases in seismic load calculation. The other building codes also suggested similar importance for designing a hospital building. Euro Code 8 (EC 8), Australian Building Code (AS11704) and New Zealand building code (NZS 4203) recommended greater importance by 75%, 20% and 30% respectively.

However, it is now important to strengthening the existing hospital building so as to ensure the structural safety during moderate to large earthquake. One of the widely used and possible techniques is base isolation that could be implemented in Bangladesh. A significant amount of past and recent research in the area of base isolation has focused on the use of elastomeric bearing such as HDRB and LRB. Bhuiyan et. al (2009) covered experimental tests, analytical models, and non-linear behavior of HDRB. Although it is a relatively recent technology, seismic isolation for multistoried buildings has been well evaluated and reviewed (Baratta and Conbi 2004). In this technique, the building structure is decoupled from the horizontal stiffness between the structure and foundation. Rubber bearing is most commonly used isolation device for this purpose. A rubber bearing typically consists of alternating lamination of thin rubber layers and steel plates (shims), bonded together to provide vertical rigidity and horizontal flexibility. Two steel plates are attached on the top and bottom of the bearing for connecting with superstructure and substructure respectively. Vertical rigidity assures the isolator will support the weight of the structure, while the horizontal flexibility converts destructive horizontal shaking into gentle movement. This type of application can be proposed for the considered Chittagong medical college hospital building to increases its seismic capacity.

SEISMICITY OF CHITTAGONG

Bangladesh lies in a moderate earthquake prone region and historical events point to major earthquakes within or close to the country. According to Geological Survey of Bangladesh, the country has experienced at least 465 earthquakes of minor-to-moderate magnitudes between 1971 and 2006. The collision of Indian plate, moving northeast within the Eurasian plate is the cause of frequent earthquakes in the region comprising Bangladesh, North-East India, Nepal and Myanmar. The 1762 Chittagong earthquake having magnitude of M=7.5 have caused major landmass changes in the coastline. In recent years, small to moderate earthquakes are regularly occurring (Al-Hussaini, 2005) with epicenters in neighboring India and Burma (some within the country) which are being felt in many parts of the country, particularly in the southeast Chittagong region. Some of these earthquakes have caused severe damage. Seismic experts consider recent repeated earthquakes of low to medium magnitude as an advance warning for a massive, and potentially disastrous earthquake in the near future, as these tremors fail to release the majority of the stress that accumulates within fault rupture zones (Bolt, 2005; DPF, 2003). Chittagong City is located in the moderate seismic region in the seismic zonation map in BNBC 1993. The 1993 Bangladesh national building code has adopted a seismic zoning map (Ali and Choudhury, 1994) consisting of three seismic zones, with zone coefficients of 0.25 (Zone 3 in the north and northeast), 0.15 (Zone 2 in the middle, north-west and south-east) and 0.075 (Zone 1 in the south west). This zoning map is based on peak ground accelerations estimated by Hattori (1979) for a return period of 200 years.

DESCRIPTION OF THE HOSPITAL BUILDING

The Chittagong medical college hospital is one of the government medical hospital in Bangladesh located in Chittagong City (moderate seismic zone). The building is ten storeys with single basement and approximate floor area of 125840 ft^2 (13550 m^2). The typical bay size is 6.096 m X 6.096 m with a floor to floor height is 3.048 m. The unit weight for the concrete and brick is 23.56 KN/m^3 and 19 KN/m^3 respectively. Compressive strength and modulus of elasticity for the concrete are 21 MPa and 200 KN/mm^2 respectively. The geometric dimensions of the structural members are shown in the table 1 and table 2.

Table 1: Beam and Column

Beam						Column		
Beam	Flange	Flange	Web	Web ht	Rectangular	Col	Width X	Circular

	width (mm)	h_t (mm)	width (mm)	(mm)	(width X depth)	umn	depth (mm X mm)	column Diameter (mm)
B1	600	150	300	350		C1	625 X 750	
B2	600	150	450	300		C2	625 X 625	
B3	500	150	450	300		C3	500 X 750	
B4	450	150	300	300		C4	300 X 600	
B5	750	150	600	300		C5		500
CB					600 X 250	C6		300
GB					300 X 750			

Table 2: Thickness of floor and foundation

Story level	Floor	Thickness (mm)
1-10	T_1	150
1-10	T_2	175
1-10	T_3	200
Roof	T_r	150
Mat Foundation	T_m	900

MODELLING OF THE HOSPITAL BUILDING

The structural software SAP 2000 has been used for creating structural model of the hospital building. Beams and columns are modelled as frame element. The floors, shear wall and mat are modelled as shell element. Compression and tension springs are used to show the connection between mat and soil. Lead rubber bearing has been modelled as link element. The link elements (LRB) are placed below the ground floor and top of the basement column.

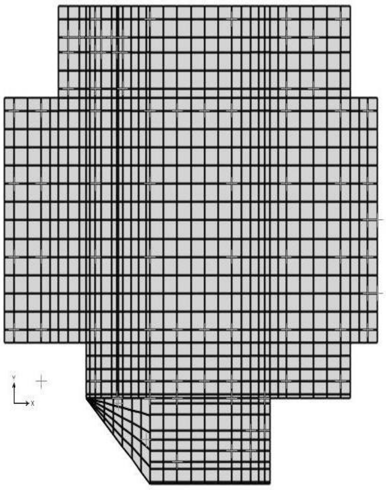

Figure 1: Mat layout

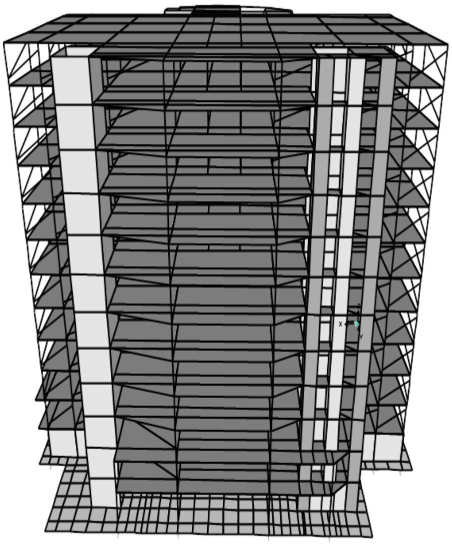

Figure 2: 3D view of the hospital building

MODELLING OF INFILL WALL

There are many way to model masonry infill wall. Stafford-Smith and Carter (1969), Main stone (1971), and others derived complex expressions to estimate the equivalent strut width, a, that consider the parameters like the length of contract between the column/beam and the infill, as well as the relative stiffness of the infill to structure. Expressions used here have been adopted from Main-stone (1971) and Stafford-Smith and Carter (1969), for their constantly accurate prediction of in-filled frame in plane behaviour when compared to the experimental results. The equivalent strut width 'a' depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness evaluated using Eq. 1(Stafford-Smith and Carter, 1969):

$$\lambda_1 H = H\, [(E_m\, t \sin 2\theta) / (4\, E_c\, I_{col}\, h_w)]^{1/4} \qquad (1)$$

Where, t is the thickness of masonry wall. Using this expression, Main stone (1971) considers the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel was calculated as shown in Eq. 2

$$a = 0.175 D\, (\lambda_1 H)^{-0.4} \quad (2)$$

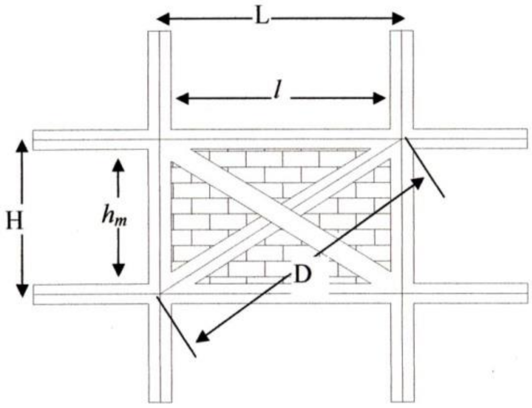

Figure 3: Strut geometry

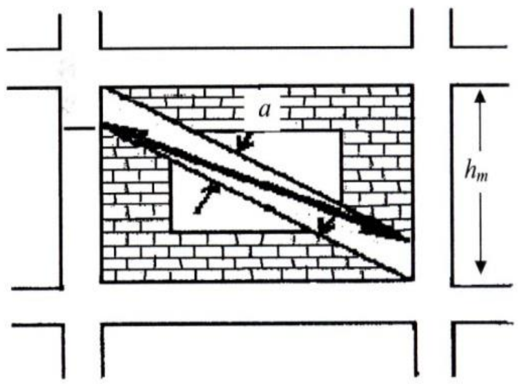

Figure 4: Placement of strut

In case of opening present with existing infill damage the equivalent strut need to be modified using

$$A_{red} = a\,(R_1)_i\,(R_2)_i \tag{3}$$

Where,

$(R_1)_i$ = reduction factor for in-plane evaluation due to presence of openings

$(R_2)_i$ = reduction factor due to existing infill damage

The infill forces are assumed to be resisted by the columns and the strut are places accordingly. The strut is pinned connected to the column at a distance l_{column} from the face of the beam. The distance is defined in Equations (4) and (5) and is calculated using the strut width (a) without any reduction factors.

$$L_{column} = a / \cos\theta_{column} \tag{4}$$

$$\tan\theta_{column} = \{h_m - (a / \cos\theta_{column})\}/l \tag{5}$$

Using this convention, the strut force is directly applied to the column at the edge of its equivalent strut width.

The existence of infill wall has been modelled by equivalent strut model. The stiffness contribution of infill wall has been calculated using regarding formula of Stafford and Smith (Stafford-Smith and Carter 1969). Necessary fractial values were considered for reduction of the stiffness for presence of opening in the infill wall. The strut is placed with an eccentricity that depends on length of the infill panel. The crushing strength is also calculated. Equivalent struts were modelled as a two point beam element with pin connection at the both end.

MODELLING OF BASE ISOLATION

It is recognized that the isolation bearing has generally nonlinear inelastic hysteretic property. Three different types of lead core rubber bearing were analytically designed for the considered hospital building using the design equations (JRA, 2004). For the designing approach the rubber material were considered as G8 and G10 type. The rubber shear modulus (G) and bulk modulus (K) are considered 1 MPa ($0.5 < G < 2.5$MPa) and 1500 MPa (1000MPa $< K < 2500$) respectively. The force displacement curve is used as bi-linear and the shear strain of the bearings are used here 175%. The shear strain of rubber bearing for building used in USA and Japan are 100 % and 200 % respectively. The properties of isolation devices are shown in Table 3.

Table 3. Properties of isolation system

Isolation system type number	Size (mm X mm)	Equivalent linear properties			Parameters in Non linear analysis				
		Effective Stiffness (KN/mm)	Effective damping	Vertical stiffness (KN/mm)	K_1 (KN/mm)	K_2 (KN/mm)	e = (K1/K2)	F_y (KN)	Q_d (KN)
LCRB_1	1050 X 1050	11.25	20%	2415	57.52	8.85	0.153	2756	590
LCRB_2	850 X 850	7	20%	1454	38.5	5.92	0.153	1714	262
LCRB_3	400 X 400	1.86	20%	444	8.26	1.26	0.153	457	147

The elastomeric devices (LRB) nonlinear properties are assigned in two horizontal shear directions and only linear elastic behaviour is accommodated in remaining axial direction.

STRUCTURAL ANALYSIS

In this method the inertia forces are determined as static force with the use of empirical formulas. To adequately represent the dynamic behaviour of the structures, the method is highly recommended for regular structures with uniform distribution of mass and stiffness as well as uniform shape and statical system. However, it can be applied to irregular ones with some limitations. The Bangladesh National Building Code (BNBC 1993) suggests to use equation (6) to calculate the vertical distribution of earthquake load.

$$V = ZICW / R \quad (6)$$

Where Z, is the seismic zoning co-efficient, I is the structural importance co-efficient and R is response modification factor. W is seismic load which includes the dead load with 25 % of live load in case of presence of parking type construction practice. Earthquake load is considered in both directions. The BNBC-1993 did not provide any guideline for calculating earthquake loads of isolated structure. The Uniform Building Code (UBC 1994) provided the guidelines for calculating the earthquake loads of isolated structure.

According to the UBC 1994, the design forces that the super structure and the elements below the isolation interface are to be designed for based on the design displacement, D. Elements below the isolation (working stress design) are calculated using formula

$$V_b = K_{max} D / 1.5 \quad (7)$$

Elements above the isolation system are calculated using the formula

$$V_s = K_{max} D / R_{WI} \quad (8)$$

The R_{WI} is a design force reduction factor (ductility factor) ranging from 1.5 to 3.0. In all cases V_S should not be less than

a) The seismic force required by the UBC provisions for a fixed-base structure.
b) The base shears corresponding to design wind load.
c) The lateral force required to fully active the isolation system, i.e., the yield load of a lead plug bearing or slip threshold of a sliding bearing system.

According to the UBC 1994, the lateral force at level x, denoted by $F_{x,}$ is computed from base shear, V_s, by

$$F_x = V_s (h_x w_x) / (w_1 h_1 + w_2 h_2 + \ldots\ldots\ldots\ldots + w_n h_n) \tag{9}$$

It is useful at this point to evaluate this code requirement implies in terms of a seismic base shear coefficient, $C_{s.}$

$$C_s == V_s / W = (N\, Z\, S / B\, T\, R_{WI}) \tag{10}$$

In this equation 'T' is the fundamental period of the isolated structure which is in between 2-3 sec. B is the damping reduction factor which depends on the damping of the isolation system. For 20 % damping the UBC 1994 suggested the value 1.5 for damping reduction factor and R_{WI} is the response reduction factor.
But in case of fixed base structure the term 'T' is the fundamental time period of the fixed base structure and the response modification factor which depends on lateral forces resistant frame and the maximum value of 12.

Response spectrum analysis is used for analysing the performance of structures under earthquake motions. The method assumes a single degree of freedom system to be excited by a ground motion in order to obtain the response spectrum curves for peak displacement, peak velocity or peak acceleration. Thus once the natural period of the structure is known then the response spectrum curves helps in estimating the peak responses of such structure. These estimated values are considered as the basis for calculating the earthquake forces to be resisted through earthquake resistant design stages. In order to perform RS analysis, important parameters in terms of expected earthquake intensity in the considered zone and the supporting base soil behaviour is considered. One of the other parameters related to the computation process is the modal analysis in which the RS analysis computes the structure's response through considering the significant modes. Mode contribution to the structure's response and flexural deformation is mainly dependent on the structure's height. For low to mid-rise structures, the first three modes are sufficient to capture accurate results where the higher modes contributions diminish very quickly. However, more than three modes have to be considered for high-rise structures. These numbers of requested modes can be selected such that their combined participating mass is at least of 90% of the total effective mass in the structure. Once the number of significant modes is established, several methods are used for the purpose of estimating the peak response values. The Square Root of Some of Squares (SRSS) of the maximum modal values is one of the popular methods. The two types of building were analysed using UBC 1994 response spectrum.

RESULTS & DISCUSSION

The existing Chittagong Medical Hospital Building (Model 1) and seismically isolated Chittagong Medical Hospital building (Model 2) are analyzed for moderate and high seismic zone (BNBC 1993) effects. Static and Dynamic analysis is done for the two model considering the seismic zone effect. From the equivalent static force method it is estimated that due to presence of isolation devices in the model 2 the base shear is reduced by 17% which indicates that large amount of column shear stress will be reduced. The base shear comparison of two models from dynamic analysis in moderate seismic zone (Z =0.15g) and high sesimic zone (Z=0.25g) are presented in Fig. 5 & Fig. 6.

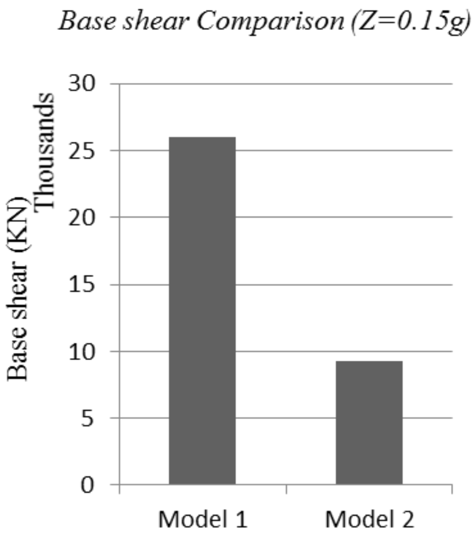

Fig. 5

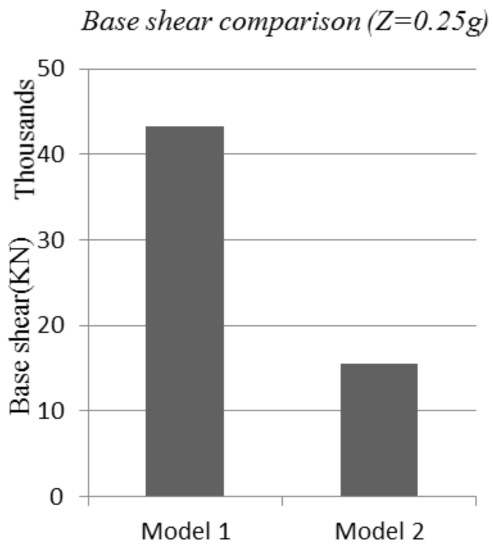

Fig. 6

The base shear from dynamic analysis is reduced by 64 % in case of model 2 for both seismic zone. It is also calculated that base shear is increased by 40 % in high seismic zone with compare to medium seismic zone.

Story displacements as a structural responses from dynamic analysis for moderate seismic zone and high seismic zone are shown in Fig. 7 and Fig. 8. From Fig. 7 & Fig. 8 it is seen that in case of model 1 the story displacement curve starts with steep gradient due to present of shear wall. In the base isolation devices level the story displacement is high due to presence of flexible isolation devices.

Story displacement comparison (Z=0.15g)

Story displacement comarison (Z=0.25g)

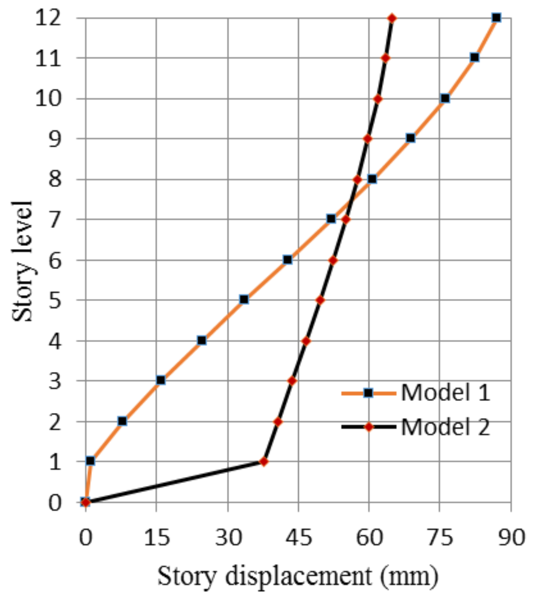

Fig. 7

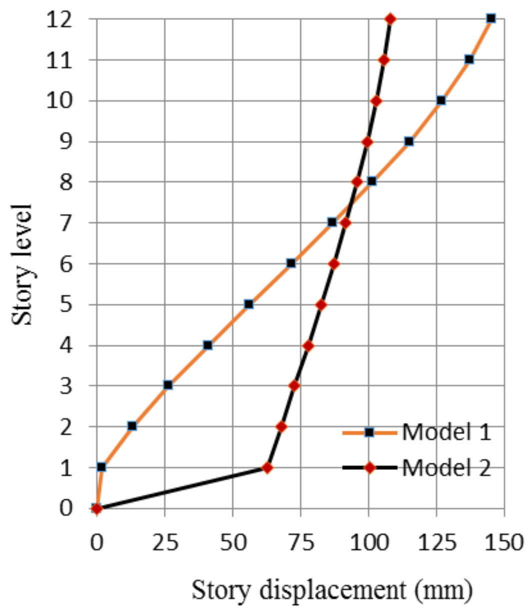

Fig. 8

Fig.7 & Fig. 8 also represents that the story displacement curve for model 2 is of high gradient after isolation level.

Fig. 9 & Fig. 10 represent the story drifts profile for considered seismic zone. It is found that the story drifts from dynamic analysis is decreased by 68 % for model 2. The rate of change of story drifts is nearly zero for model 2.

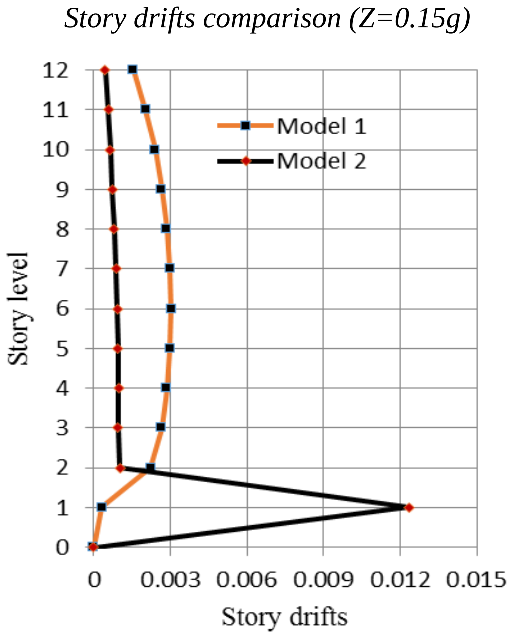

Fig. 9

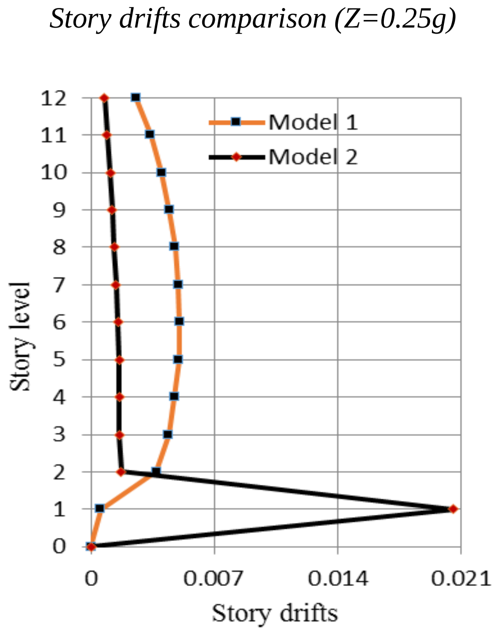

Fig. 1

Presence of isolation devices in the model 2, a portion of the seismic force is absorbed by the devices induces during earthquake.

It is also calculated the reinforcement's requirement for columns for response loading in high seismic zone and the result is given in the Table 4.

Table 4. COMPARISON OF QUANTITY OF REINFORCEMENTS REQUIRED IN COLUMN

Story	fixed base (m^3)	isolated base (m^3)	% reduction
GF	0.9083	1.023322	-12.66
1st	1.4798	0.79248	46.44
2nd	1.356	0.714632	47.30
3rd	1.11	0.601856	45.77
4th	1.003	0.531752	45.75
5th	0.9296	0.522608	43.78
6th	0.8768	0.516512	41.09
7th	0.825	0.513464	37.75
8th	0.789	0.513464	34.92
9th	0.754	0.513464	31.90
10th	0.742	0.513464	30.80
Top	0.784736	0.513464	34.55
Total	11.56063	7.270484	37.11 %

From the Tab. 4 it is found that the requirement of reinforcements in model 2 is 37.11 % less than model 1.

CONCLUSION

This study was conducted just to understand the seismic responses of the existing medical building if base isolation could be provided. From the structural analyses, it was found that structural model with base isolation reduced about 64 % base shear. Story drifts also reduced by 68 % for the building having base isolation. Due to reduction of structural responses the reinforcement's requirement in the column of the structure reduced approximately to 37.11 %. It can be concluded that the use of isolation devices reduces the reinforcement requirement in structural members. The analyses results recommend that base isolation can be a good solution to strengthen this existing structure.

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FINITE ELEMENT MODELING, ANALYSIS AND VALIDATION OF THE SHEAR CAPACITY OF RC BEAMS MADE OF STEEL FIBER REINFORCED CONCRETE (SFRC)

M. M. Islam[1]*, M. S. Hossain[1], S. Amin[1], Z. Islam[1], M. Bala[1], M. Chowdhury[1] & A. Siddique[1]

[1]Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka 1208, Bangladesh.
**Corresponding Author, <mashfiq7777@gmail.com>*

ABSTRACT

Steel fiber reinforced concrete (SFRC) is used for its improved shear resistance, ductility and crack control properties. Steel fibers distributed randomly to a concrete mix act as crack arrestors, changing concrete from a brittle material to a ductile one, in addition to improving toughness and rigidity. To evaluate the shear capacity enhancement of SFRC, reinforced concrete (RC) beams are cast and tested in a 1000 kN capacity digital universal testing (UTM) machine under two point loading. The specimens are shear critical (SC) beams with only longitudinal reinforcement to observe the actual shear capacity enhancements due to steel fibers. In this work, the laboratory prepared end enlarged type of steel fibers of three aspect ratio are used, i.e. 40, 60 and 80 to make the SFRC and control beam (without steel fiber) for comparing the results. The shear capacity is found to be enhanced 25%, 29% and 18% for SFRC made of steel fiber aspect ratio (SFAR) 40, 60 and 80, respectively. These beams are also modelled and analyzed in the finite element (FE) platform of ANSYS 11.0 The FE outcomes upholds a good agreement with the experimental investigations. The failures patterns of the beams also showed wonderful similarities. The use of SFRC is not yet started in the construction industry of Bangladesh for the lack of its reliable experimental data and FE modeling and analysis. This study forwards with original experimental results which may help the construction industry of Bangladesh to introduce this composite material.

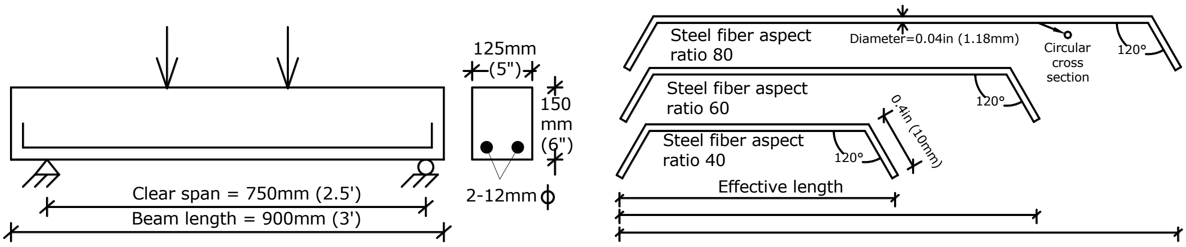
Keywords: Steel Fiber Reinforced Concrete (SFRC), Shear capacity, Steel Fiber Aspect Ratio (SFAR), Finite Element (FE) Modelling, ANSYS 11.0.

INTRODUCTION

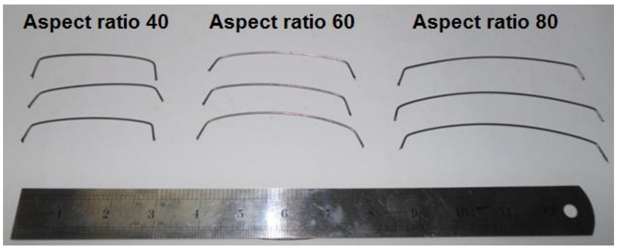
Steel fiber reinforced concrete (SFRC) is being increasingly used in shortcrete as tunnel linings, pre-cast structures, off-shore platforms, water-retaining structures, structures in high seismic risk areas, bridges, industrial or factory pavements, highways, roads, parking areas, bridge decks, airport runways and also in versatile engineering structures. Steel fiber reinforced concrete (SFRC) is a composite material whose components include the traditional constituents of port-land cement concrete (hydraulic cement, fine and coarse aggregates, and admixtures) and a dispersion of randomly oriented short discrete steel fibers (Aoude et al. 2009). The use of steel fiber-reinforced concrete (SFRC) is increasing due to its improved material and structural behavior relative to plain concrete and even to conventionally reinforced concrete with the same steel volume fraction (Kang et al. 2011). The objectives for the addition of fibers are to improve the tensile strength, flexural strength, shear strength impact toughness and also enhance ductility to control cracking (Yakoub, 2011). The use of steel fiber-reinforced concrete (SFRC) is increasing in many countries due to its improved material and structural behavior relative to plain concrete and even to conventionally reinforced concrete with the same steel volume fraction. In Bangladesh the use of steel fiber reinforced concrete has not yet been started. In this research, shear critical (SC) RC beams made of SFRC and plain concrete made of stone aggregate are tested experimentally and also modeled in the Finite Element (FE) framework of ANSYS 11.0 to analyze the shear capacities. Results obtained from the FE analysis are compared with experimental test results and good correlation is obtained. This experimental investigation is intended to validate the shear capacity of FE models with the experimental results and to provide reliable experimental data and FE models for the construction industry of Bangladesh.

MATERIALS, METHODOLOGY AND RESULTS

The SFRC and plain concrete SC beams are designed with 2-12mmφ longitudinal reinforcements at bottom (Fig. 1a) without any web reinforcements, so that, the enhancement of shear capacity due to SFRC can be evaluated. Three different steel fibers aspect ratio (SFAR), i.e., 40, 60 and 80 are selected to cast the shear critical (SC) beam specimens with a volume ratio of 1.5%. Mild steel cold drawn wires (Type V as per ASTM A 820/A 820M-06) are selected to make the steel fibers and are prepared manually in the laboratory. The strategy is to estimate the shear capacity increment due to steel fiber in the concrete mix and also to evaluate the performance of fibers with respect to steel fiber aspect ratio (SFAR). In this case the diameters of the steel fiber is 1.18 mm, effective lengths for the SFAR 40, 60 and 80 are 47.2 mm, 70.8 mm and 94.4 mm respectively and the original lengths are 67.2 mm, 90.8 mm and 114.4 mm respectively. According to ACI-544.4R-88, enlarged-end fibers show the maximum energy absorption as well as tensile strength enhancement, so the fibers are bent 120° at the ends to make the enlarged ends (Fig. 1b & c). Crushed stone (CS) as coarse aggregate is used to make the plain concrete and SFRC SC beams. A total of 4 compression cylinder (4x8in), 4 splitting cylinder (6x12in) and 4 SC beams (6x5x36in i.e. 150x125x900 mm) specimens are tested at constant rate of 0.5mm/min by a displacement control 1000kN capacity digital universal testing machine (UTM) until the failure occurred. Load and displacement values are measured from load cell of UTM. The tensile strength of steel fiber is 1100 MPa (160 ksi) which satisfies the minimum requirement of ASTM A 820/A 820M-06 (i.e. 345 MPa or 50 ksi). All the beams are made with a Cement:FA:CA of 1:1.5:3 and w/c of 0.5. Mild steel rebar of 60 grade (yeild strength 60 ksi or 420 Mpa) is used as longitudinal reinforcement. Lateral strain of cylinder at the failure location are measured by analyzing the image histories obtained from high speed video clips and high definition (HD) images employing an image analysis technique which is called digital image correlation technique i.e. DICT (Islam et al. 2011, Islam et al. 2014). Then the load and displacement values from UTM are synthesized with the strain measurement results gathered from DICT. The compressive strength of SFRC made of steel fibers having aspect ratio 40 is found 17.6% increased with respect to control specimen (normal concrete without fiber). But in case of steel fiber aspect ratio 60 and 80 reduced compressive strength is observed due to uneven distribution of concrete in cylinders for larger length of steel fibers. But in case of ductility is increased about 5, 3.6 and 3 times for SFAR 40, 60 and 80 respectively which one of the major requirement of this investigation. The tensile capacity of concrete with steel fiber aspect ratio 40, 60 and 80 is increased 58%, 117.5% and 64.1% respectively and the ductility also increased about 15, 9.2 and 13 times respectively compared to the plain concrete. The ultimate shear capacity is enhanced 25%, 27% and 18% for SFAR 40, 60 and 80 respectively and ductility is enhanced 1.33, 1.58 and 1.17 times respectively. Figure 4 shows the tensile stress-strain behavior of concrete made of stone aggregate without fiber and with steel fiber. These results are summarised in Table 1 and 2 and Fig. 2 represents the experimental test results graphically.



(a) (b)



(c)

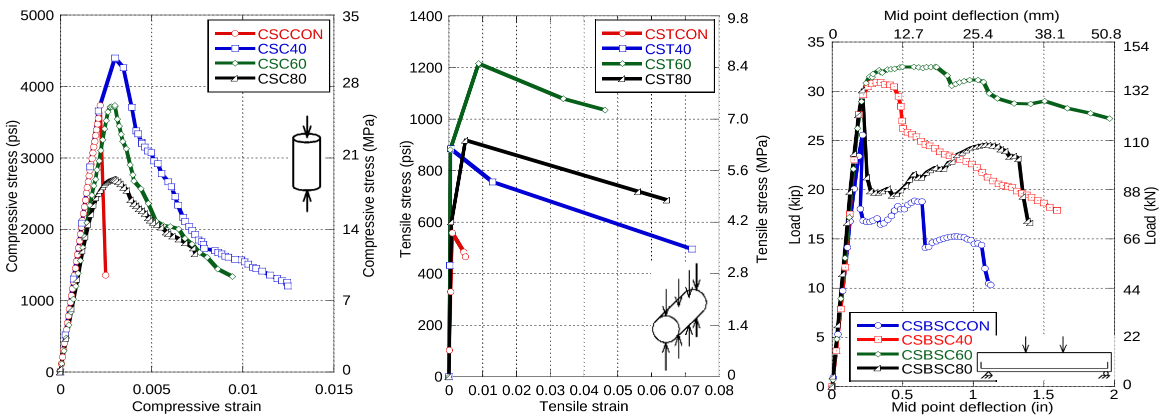
Fig. 1: (a) Reinforcement layout and beam dimension as per strategy (b) size and geometry of steel fibers (c) image of steel fibers of different aspect ratio.

Table 1: Summary of the concrete cylinder specimen

Specimen ID	Maximum compressive strength, psi (MPa)	Specimen ID	Maximum tensile Strength, psi (MPa)	Steel fiber aspect ratio
CSCCON	3741 (26)	CSTCON	560 (4)	-
CSC40	4400 (30)	CST40	885 (6)	40
CSC60	3733 (25.7)	CST60	1218 (8.5)	60
CSC80	2691 (19)	CST80	919 (6.5)	80

Table 2: Summary of SC beam specimens

Specimen designation	1st crack load kip (kN)	% incre-ment	1st shear crack load kip (kN)	% incre-ment	Ultimate load kip (kN)	% incre-ment	Ductility increment (times)
CSBSCCON	13 (58)	-	21 (95)	-	25.5 (112)	-	-
CSBSC40	21 (91)	54	27 (120)	29	32 (136)	25	1.33
CSBSC60	21 (93)	54	29 (129)	38	33 (143)	29	1.58
CSBSC80	23 (102)	77	27 (120)	29	30.2 (133)	18	1.17



(a) (b) (c)

Fig. 2: Experimental results of plain concrete and SFRC (a) compression (b) splitting tension (c) load deflection behaviour of beams.

FE MODELLING, ANALYSES AND VALIDATION

SOLID65 is used in ANSYS 11.0 to model the concrete and also SFRC, which is a three dimensional (3D) solid element having eight nodes with three degrees of freedom at each node, i.e., translational in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in tension, crushing in compression and is also applicable for reinforced composites (ANSYS 2005), such as, fibreglass, SFRC etc. The flexural reinforcement is modelled using LINK8 element, which is a 3D spar element as well as a uniaxial tension-compression element with three degrees of freedom at each node same as SOLID65. The geometry and node locations for SOLID65 and LINK8 elements are shown in Fig. 3. For the perfect modelling of plain concrete and SFRC, ANSYS requires data for material properties such as (i) elastic modulus, (ii) density (iii) Poisson's ratio (iv) stress-strain behaviour (Fig. 2a) (v) tensile strength (vi) closed shear transfer co-efficient (vii) open shear transfer co-efficient etc. Table 3 shows the properties applied in FE modelling. Loading is applied as displacement boundary condition (Fig. 3c) and deformed shape is shown in Fig. 3d. The displacement boundary condition is applied in 500 steps followed by 2 sub-steps for each step. Fig. 4 shows the validation of results of load deflection behaviour gathered from the experimental investigation with the FE analysis by ANSYS 11.0 and also satisfactorily demonstrate the accuracy of the FE model plain concrete as well as SFRC. FE analyses have shown conservative results compared to experimental result which indicate sufficient factor of safety and also ensures a reliable model. Typical stress and strain contours of SC beam specimens are given in Fig. 5. The failure pattern of experimental and FE RC beams are given in Fig. 6. In the both cases, the failure patterns are almost similar pattern and at same locations and shear and flexural cracks are found and also. This also validates the FE models. Development of the cracks which are diagonal in pattern signifies that the beam models are critical in shear as no web reinforcements are provided. Also flexural cracks are developed at the middle span of the beam.

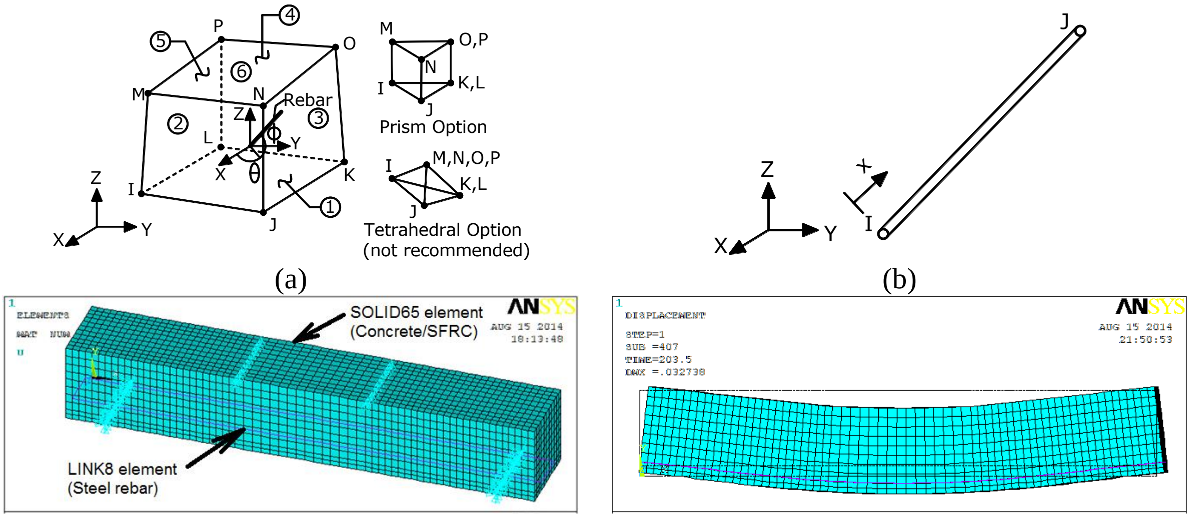


Figure 3: (a), (b) Geometry of SOLID 65 and LINK8 element in ANSYS 11.0 platform respectively (c) beam with boundary condition (d) deformed beam.

Table 3: FE input data for SOLID65 and LINK8 element

Properties for FE model	Beam specimen (SOLID65)				Rebar (LINK8)
	CSBSCCON	CSBSC40	CSBSC60	CSBSC80	
Density	2.69g/cm^3	2.77g/cm^3	2.72g/cm^3	2.74g/cm^3	7.8g/cm^3
Tensile strength	4 MPa	6 MPa	8 MPa	6.3 MPa	-

Poisson's ratio	0.325	0.325	0.325	0.325	0.3
Shear transfer co-efficient: closed crack	0.25	0.5	0.5	0.5	-
Open crack	0.3	0.3	0.3	0.3	-
Yield stress	-	-	-	-	420 MPa

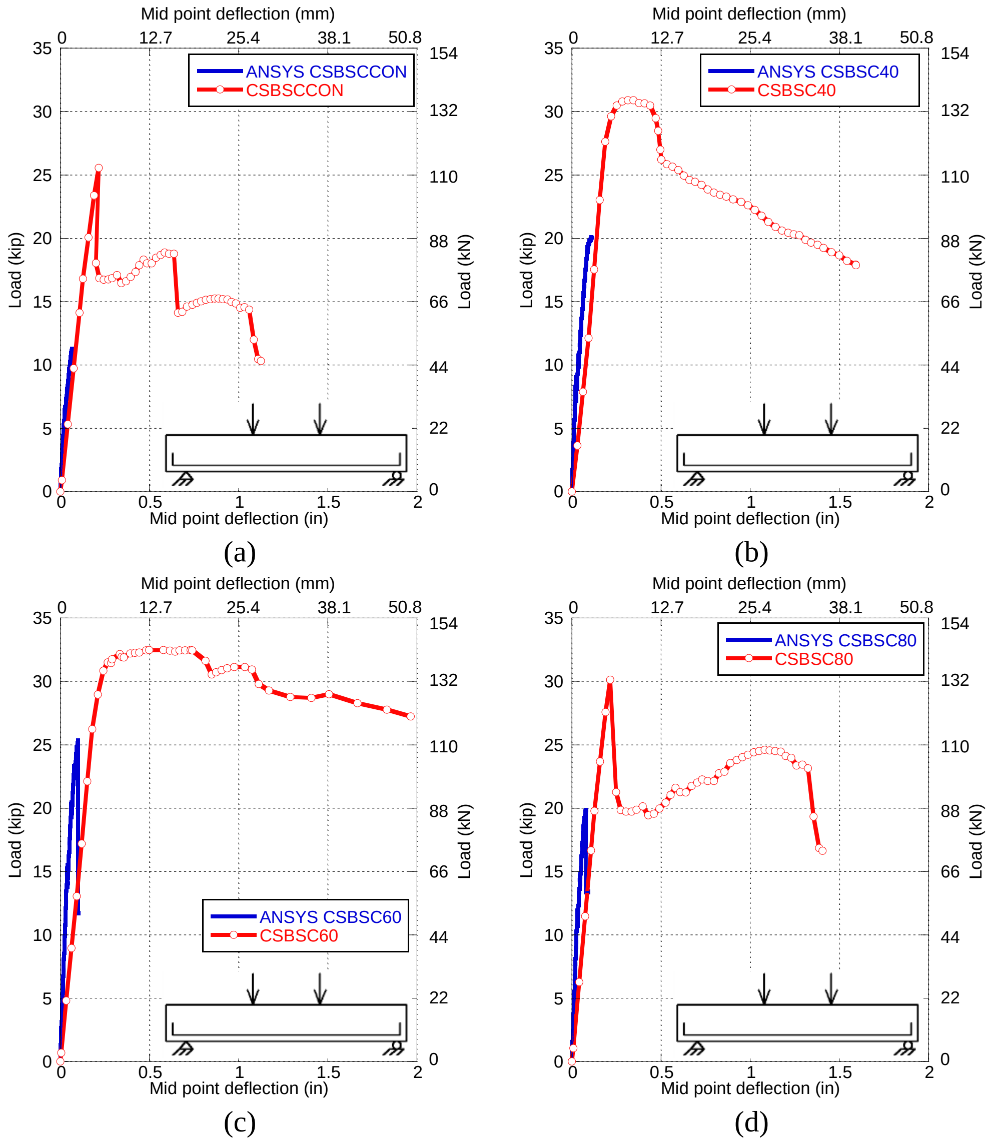
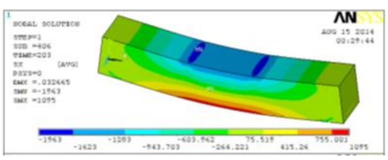
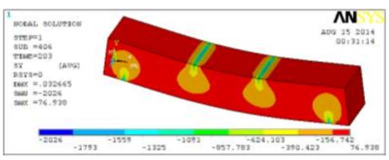


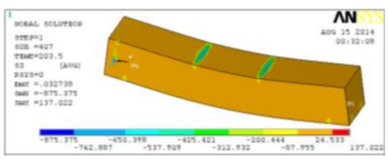
Fig. 4: Evaluation of load deflection behaviour FE and experimental SC beams a) CSBSCCON i.e. control beam b) CSBSC40 c) CSBSC60 d) CSBSC80.



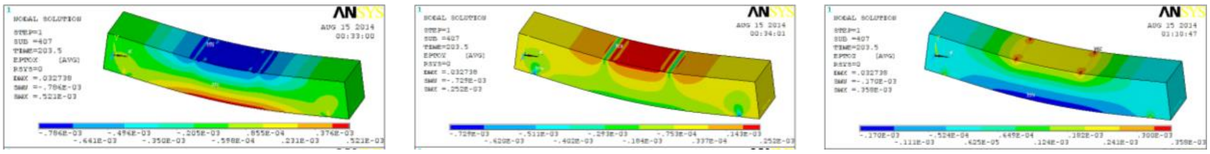
X component stress



Y component stress

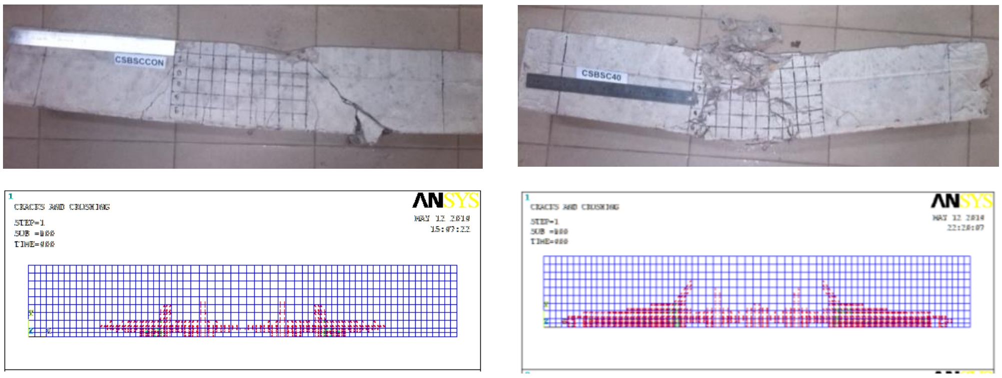


Z component stress

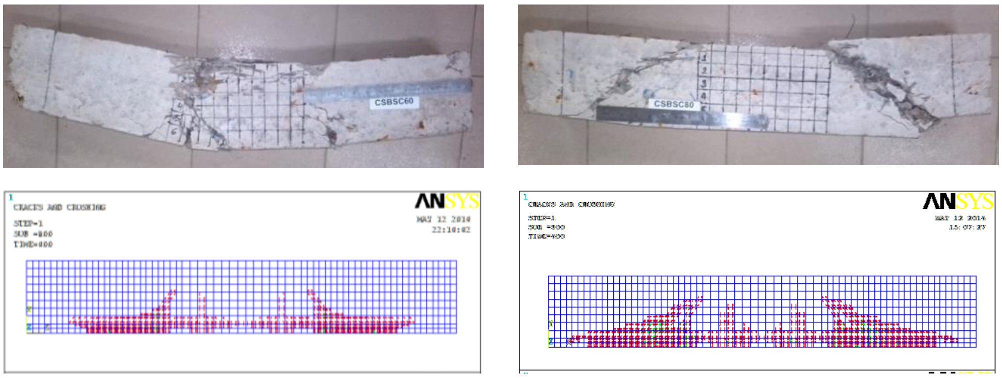


X component strain Y component strain Z component strain

Fig. 5: Typical stress and strain contours at X, Y, Z direction.



(a) (b)



(c) (d)

Fig. 6: Experimental and FE failure pattern (a) CSBFCCON (b) CSBFC40 (c) CSBFC60 (d) CSBFC80.

CONCLUSION

The load-deflection behaviour from experimental investigation shows that the shear strength of SC beams increased about 25%, 29% and 18% for the SFAR 40, 60 and 80 respectively with compared to control specimen and the ductility is enhanced 1.33, 1.58 and 1.17 times respectively. The FE outcomes showed a good aggrement with the experimental results, similar failure patterns and failure location. FE models showed conservative results which ensure adequate factor of safety as well as reliability of FE modeling and analyses that may help the construction industry of Bangladesh.

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ID: SEE 076

SEISMIC REHABILITATION OF FLAT SLAB BUILDING BY USING CFRP BARS AND STEEL BRACKET

M. Ashiquzzaman[1*] & M. T. Islam[2]

[1] *Dept. of Civil Engineering, Saint Louis University, St. Louis, MO 63103, USA.*
[2] *Dept. of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka 1000, Bangladesh.*
**Corresponding Author, <ashiquzzamanm@slu.edu>*

ABSTRACT

Old flat slab buildings are usually seismically vulnerable. Most of the cases the old flat slab buildings were not designed or constructed as per seismic code provisions. These old flat slab buildings are now-a-days become a great headache for engineers for its seismic rehabilitation. In this study, the flat slab is strengthened along the direction of column by using CFRP bars at the bottom of the slab. These CFEP bars are capable enough to increase the effective depth of the slab which is good enough to increase flexural capacity. In this situation, the slab stripe with CFRP bars acts like a beam. In addition, to increase the strength of the beam (slab stripe with CFRP bars)-column joints, steel brackets are used at the joining corner of the beam (slab stripe with CFRP bars) and column. The application of this rehabilitation technique was conducted based on FEMA 356, FEMA 440, ACI 318-11, and ACI 440.2R-08. An ABAQUS model is developed to find out the stress distribution and other responses of the structure after applying this rehabilitation technique.

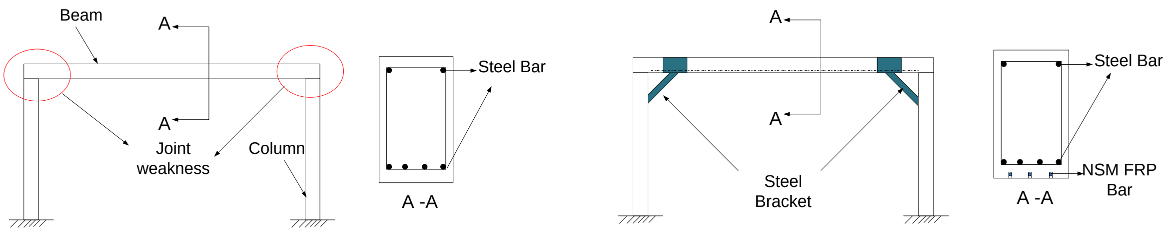
Keywords: Seismic rehabilitation, Flat slab, FRP bar, Beam-column joint, Steel bracket.

INTRODUCTION

Improvement of knowledge of the dynamic response and seismic performance of structures has led to new era of advances in earthquake engineering in recent years. Many existing structures located in near seismic regions or far seismic regions are not adequate in design based on current seismic design codes. In general, buildings that were constructed before the 1970s have significant deficiencies in their overall structural design, such as discontinuity in reinforcement in beams and slabs, or wide spacing of transverse reinforcement. Flat-slab buildings designed and detailed for gravity loads only typically do not have the ability to resist moderate earthquakes without experiencing severe damage. The damage potential of such seismically deficient buildings therefore needs to be assessed and strategies developed to improve their seismic resistance. Punching failure at slab-column connections in non-ductile flat-slab buildings during earthquakes can trigger progressive collapse of floor slabs.Here in this research, flat slab buildings are strengthen by using CFRP bars at the bottom of the slab in the column to column direction. The slab stripe with CFRP bars works together as a beam in an increased effective depth of the slab which is good enough to increase flexural capacity. To construct the building more earthquake resistant it is important to make the beam-column joint stronger. By keeping this point in mind steel brackets are used at the joining corner of the beam (slab stripe with CFRP bars) and column of existing flat slab building to increase more strength of the beam (slab stripe with CFRP bars)-column joint.

REHABILITATION TECHNIQUE

The joint weakness of beam (slab stripe)-column joint is clearly shown in Fig. 1(a). To improve the joint weakness, steel bracket is used at the corner of the beam and column joint together shown in Fig. 1(b).



(a) Before rehabilitation (b) After rehabilitation

Fig. 1. Retrofiting by NSM FRP bar and steel bracket

CALCULATION FOR USING CFRP BARS

Selected seismic zone of project building

The seismic zone and its related information are found out with the help of U.S. Geological Survey shown in Fig. 2.

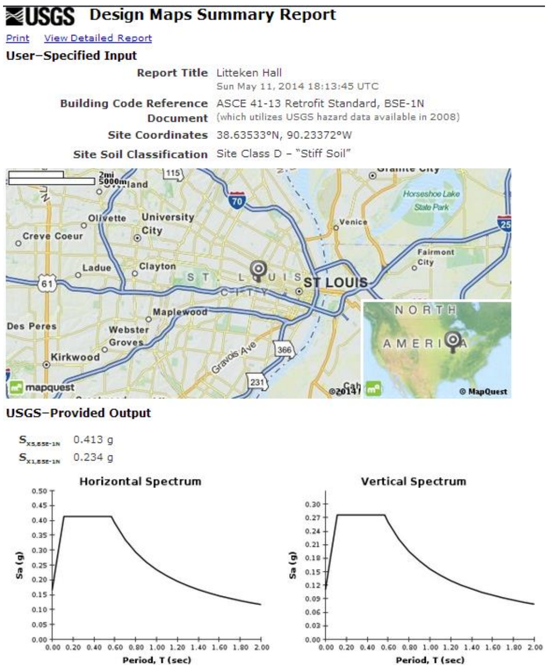


Fig. 2. Seismic zone of the project building

Building Plan and its parts need to be retrofitted

Fig. 3 shows the project building plan and portion need to retrofitted (marked by red line).

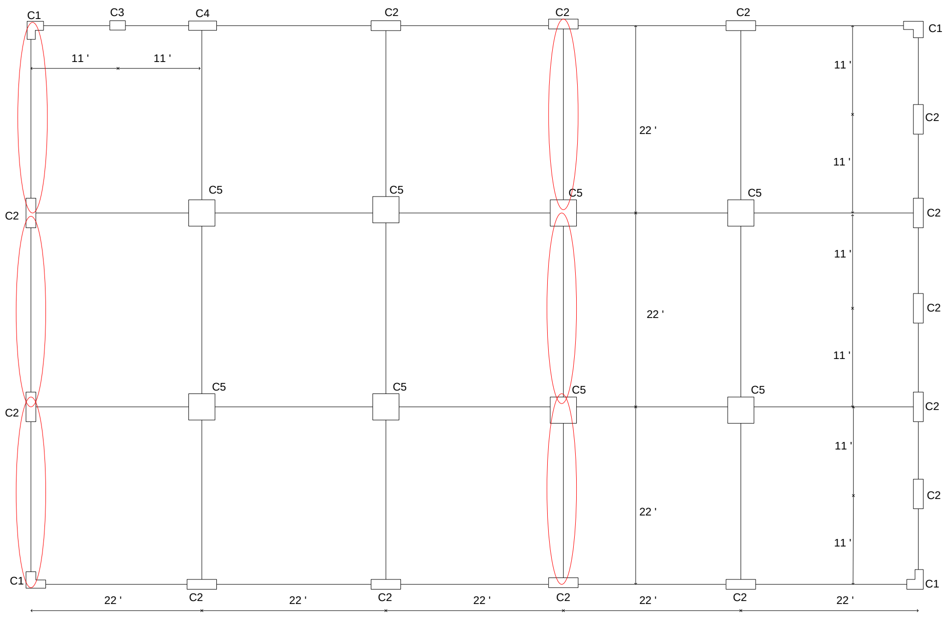


Fig. 3. Plan of the project building

Vertical distribution of seismic forces

Vertical distribution of seismic forces							
	Height, h (ft)	**Weight, w kips)**	**Wh^k/10^3**	**C_{vx}**	**(kips)**	**F_x (kips)**	**F_x when R=3.5**
Roof	20 ft.	822	328.8	0.74	1048	**775.52**	**222**
2nd FL	10 ft.	1155	115.5	0.26		**272.48**	**78**

Forces due to applied seismic and gravity load

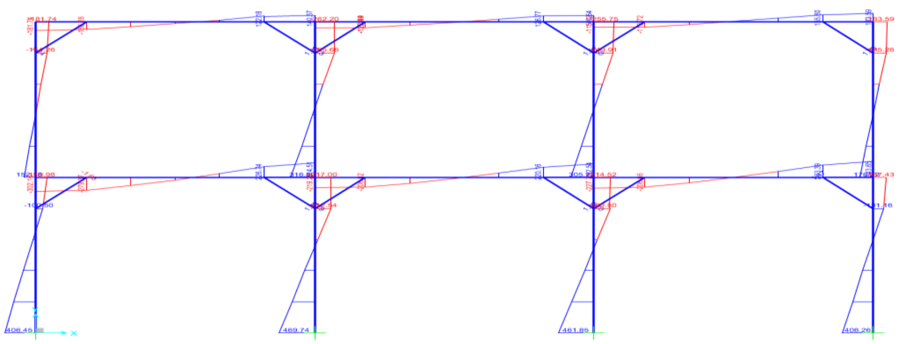


Fig. 4. Moment Diagram (Seismic load + Gravity load)

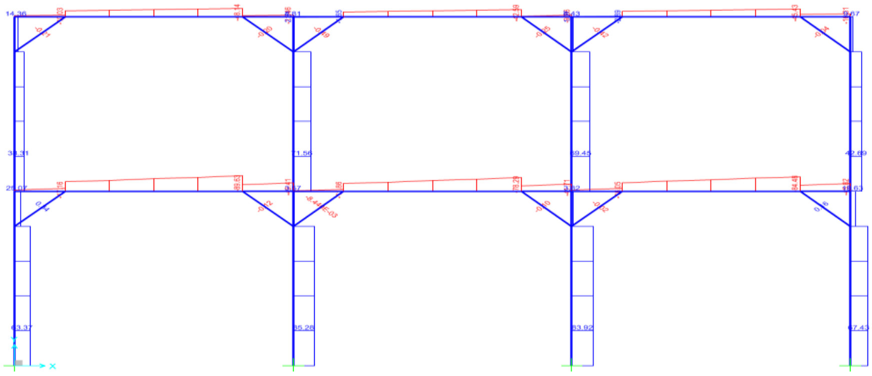


Fig 5. Shear force diagram (Seismic load + Gravity load)

Sample calculation steps

The following calculation for FRP bars is based of FEMA 356, FEMA 440, ACI 318 and ACI 440. Manufacturer's reported NSM FRP system properties

Area per No. 3 bar	0.1 in^2
Number of bars	8
Ultimate tensile strength f^{*}_{fu}	250 ksi
Rupture strain ε^{*}_{fu}	0.013 in/in
Modulus of elasticity of FRP laminates E_f	19230 ksi

Calculate design flexural strength of the section

$$\phi M_n = \phi\left[M_{ns} + \Psi_f M_{nf}\right] = 313.5\ kips < 302.1\ kips$$ (From FE Analysis) **OK**

Check service stresses in the reinforcing steel and the FRP

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - \frac{kd}{3}\right)\right](d-kd)E_s}{A_s E_s\left(d - \frac{kd}{3}\right)(d-kd) + A_f E_f\left(d_f - \frac{kd}{3}\right)(d_f - kd)} = 43.6\ ksi \le 0.08 f_y = 48\ ksi$$ **OK**

Check creep rupture limit at service of the FRP

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\left(\frac{d_f - kd}{d - kd}\right) - \varepsilon_{bi} E_f = 8.48\ ksi \le 0.55 f_y = 50\ ksi$$ **OK**

Check for shear capacity

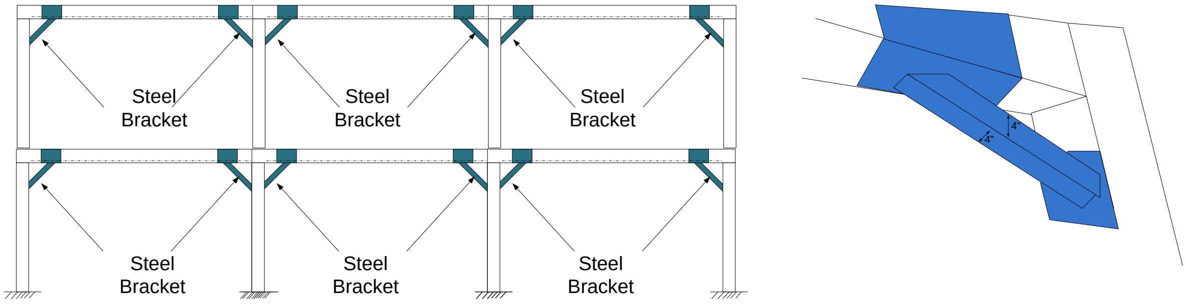
$$\phi V_n = \phi\left(V_c + V_s + \Psi_f V_f\right) = 62\ kips > V_u = 43.75\ kips$$ **OK**

Check for acceptance criteria of column for linear procedures (for "m" factor)

$$mkQ_{CE} = 279\,kips > Q_{UD} = 110.25\,kips \quad \textbf{OK}$$

FE MODEL FOR STEEL BRACKET

Detailed presentation of rehabilitation technique

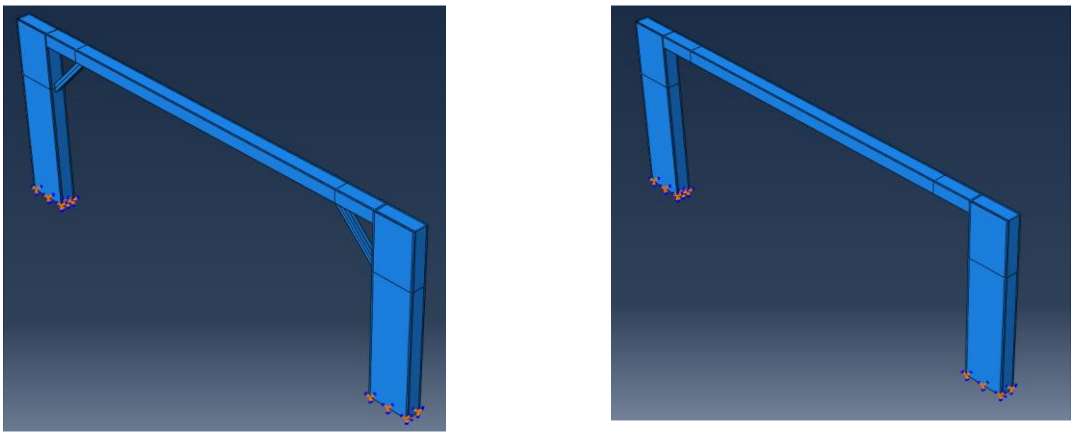


(a) Location of steel bracket (b) Method of installation

Fig. 6. Location and method of placing the steel bracket in the beam-column joint

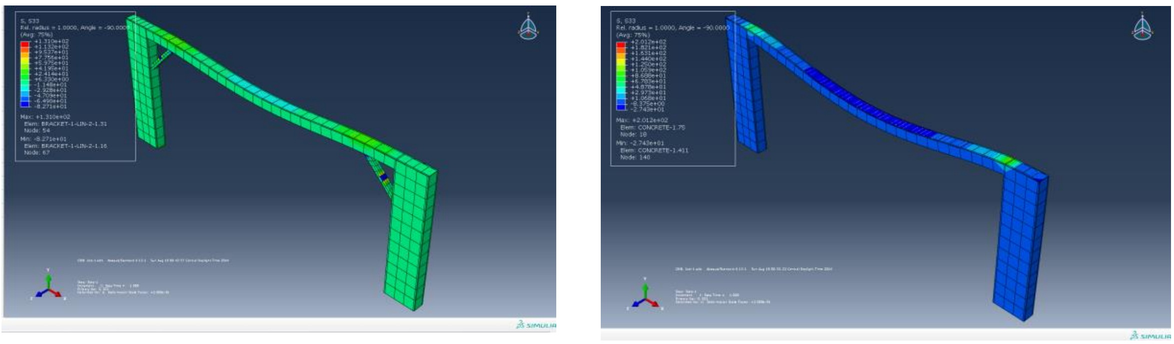
FE Model

Finite element model of retrofitted structure is made to compare unretrofitted structure shown in Fig. 7. It is clearly shown in the result that retrofitting with the steel bracket in the beam column joint offers less stress and deformation on the element which is shown in Fig. 8 and Fig. 9.



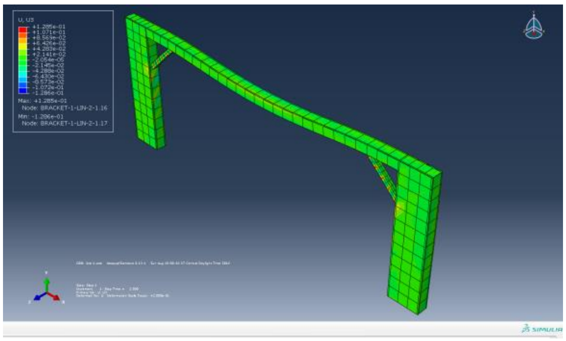
(a) Retrofitted model (b) Unretrofitted model

Fig. 7. Comparison in between retrofitted and unretrofitted model

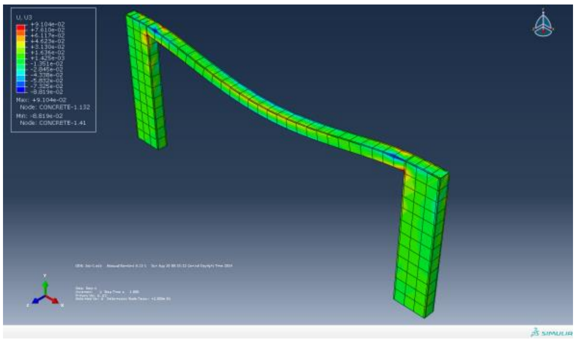


(c) Retrofitted model (d) Unretrofitted model

Fig. 8. Stresses comparison in between retrofitted and unretrofitted model



(e) Retrofitted model



(f) Unretrofitted model

Fig. 9. Deformation comparison in between retrofitted and unretrofitted model

RESULTS AND DISCUSSIONS

From the above calculation of using FRP bar in the slab stripe and combined FRP bar and steel bracket ABAQUS model of structure, it can easily be deduced that;

- FRP bars can increase the effective depth of the slab stipe so that it can act like a stiff beam.
- Steel brackets increase the strength the beam (slab stripe)-column joint.
- Steel brackets reduce the stress and deformation of the structure which make the structure more stable.
- Considering the overall aspect of this retrofitting technique it can be said that this technique can perform really good to the flat slab building during seismic events.

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SSI MODEL FOR BASE-ISOLATED CONTAINMENT BUILDING IN NUCLEAR POWER PLANT

M. Ashiquzzaman[1*], L. Hui[1], M. T. Islam[2] & K. J. Hong[3]

[1] Dept. of Civil Engineering, Saint Louis University, St. Louis, MO 63103, USA.
[2] Dept. of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka 1000, Bangladesh.
[3] Dept. of Civil Engineering, Kookmin University, Seoul, South Korea.
*Corresponding Author, <ashiquzzamanm@slu.edu>

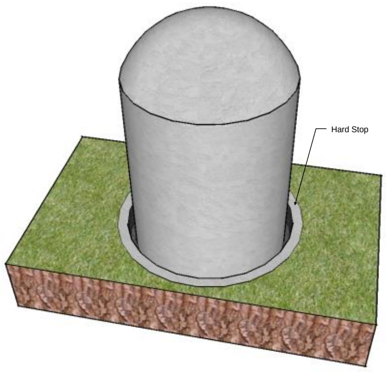
ABSTRACT

Soil-structure-interaction (SSI) has a significant effect on base isolated containment building in nuclear power plant (NPP) during very strong ground motion. Here, a simplified model is constructed to present the containment building of nuclear power plant. The model consists of basically a vertical rigid bar with a lumped structural mass at the top and a rigid linear isolation mass at the bottom of the plant. The equations of motion are derived in frequency domain by assuming soil stiffness and damping constants. Parametric studies are performed based on the modal analysis. The focus of the research is to find out the contributing mode of the model due to SSI effect on containment building in nuclear power plant.

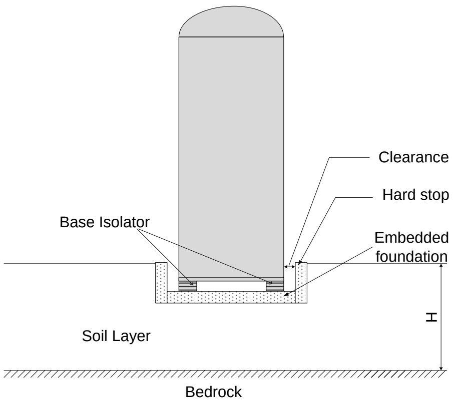
Keywords: Nuclear Power Plants, Soil-Structure-Interaction, Seismic response, Base-isolation.

INTRODUCTION

Base isolated containment building system of nuclear power plant intends to isolate the building structure from seismic ground motion. Nowadays displacement demands of isolators are a key factor for the structures in the high seismic zones. The impact of long-period pulses on the displacement demands of isolators for structures built at high seismic zone has been studied. In recent study, a comparative research has been done by Jangid and Kelly (Jangid et al., 2001) on the response of isolation systems in near-source regions, while other researchers have suggested use of additional energy dissipative mechanisms (Hall et al., 1999, Hall et al., 2000, Zhang et al., Sahasrabudhe et al. 2005). Usually soil and structure interaction results in a decrease of the fundamental frequency of the response and a modification in the energy dissipation, which is attributed to radiation and material damping in the soil (chen et al., 2003). The containment building of nuclear power plant shown in Fig.1 is greatly affected by soil structure interaction during earthquake. In this research, a simple model of containment building shown in Fig. 2 has been created to perform soil structure interaction analysis. In the simplified model, a rotational inertia of isolation mass is considered. The equations of motion are derived in frequency domain by assuming soil stiffness and damping constants. A detailed parametric study is performed based on the modal analysis to find out the contributing mode of the simple model. The objective of this paper is to extend the existing knowledge in the field of base isolated containment building structure in nuclear power plant, taking into account the role of soil structure interaction. The coupled effect of base isolation and SSI is studied with the aid of a simple analytical model. The methodology could be applied for preliminary analysis and design of base-isolated building structures accounting for the effects of SSI.



(a) 3D view of containment building



(b) Detail pictorial representation of containment building

Fig. 1. Containment building of nuclear power plant

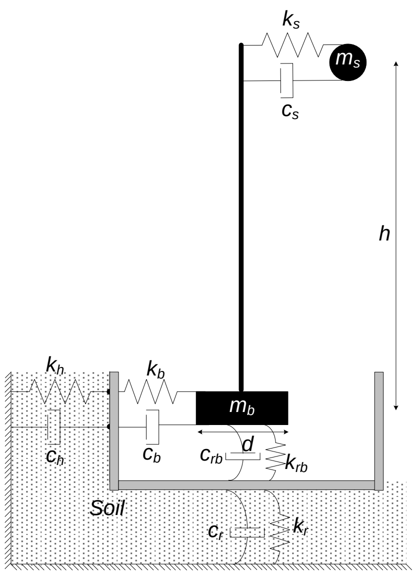


Fig. 2. Proposed Simplified lumped-mass system of containment building

THEORETICAL APPROCH

Response of the Simplified lumped-mass system of containment building

The total response due earthquake excitation of the structural mass is shown in Fig. 3. Response due to soil-structure interaction is also considered in the total structural response. The total displacement amplitudes of the structure, u_s^t, and the total displacement amplitude for isolator, u_b^t, can be expressed as, $u_s^t = u_g + h\phi_g + u_h + u_s + h\phi_r + h\phi_{rb}$ (1)

$$u_b^t = u_g + u_h + u_b \tag{2}$$

where, u_s, u_b, u_h, ϕ_{rb} & ϕ_r stand for structural, base isolation, translational, base isolation rotational and foundation rotational degree of freedom respectively.

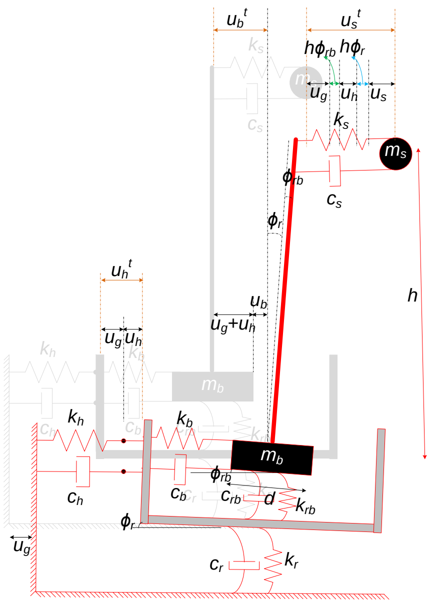


Fig. 3. Total response of the lumped masses

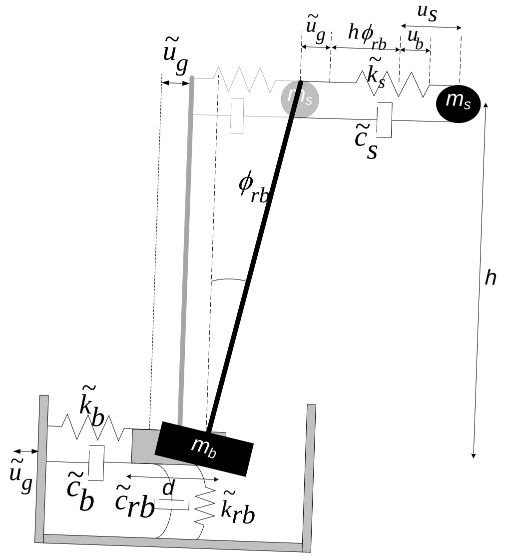


Fig. 4. Response of equivalent 3-DOF system

where, m_s, m_b stand for structural and isolation mass respectively. k_s, k_b, k_h, k_{rb} and k_r stand for structural, base isolation, translational, base isolation rotational and foundation rotational stiffness coefficient respectively. c_s, c_b, c_h, c_{rb} and c_r stand for structural, base isolation, translational, base isolation rotational and foundation rotational damping coefficient respectively. Horizontal and rocking component of input ground motion are denoted by ϕ_g & u_g respectively. h is height of structure and d is the diameter of the rigid isolation mass.

Equation of motion

Finding large displacement of the containment building due to very strong ground motion is the dominating part for this research. In the past, total structural displacement for the lumped mass system was determined for SSI analysis to get the equation of motion [7]. Those researches did not consider the rotational inertia of the isolation mass. Here in this research rotational inertia of the isolation mass is considered. The equations of motion are,

$$m_s\left(\ddot u_g+\ddot u_h+\ddot u_s+h\ddot\phi_r+h\ddot\phi_{rb}+h\ddot\phi_g\right)+m_b\left(\ddot u_g+\ddot u_h+\ddot u_b\right)+k_hu_h+c_h\dot u_h=0 \quad (3)$$

$$m_sh\left(\ddot u_g+h\ddot\phi_g+\ddot u_h+\ddot u_s+h\ddot\phi_r+h\ddot\phi_{rb}\right)+k_r\phi_r+c_r\dot\phi_r=0 \quad (4)$$

$$m_s\left(\ddot u_g+\ddot u_h+\ddot u_s+h\ddot\phi_r+h\ddot\phi_{rb}+h\ddot\phi_g\right)+k_s\left(u_s-u_b\right)+c_s\left(\dot u_s-\dot u_b\right)=0 \quad (5)$$

$$m_b\left(\ddot u_g+\ddot u_h+\ddot u_b\right)+k_bu_b+c_b\dot u_b-k_s\left(u_s-u_b\right)-c_s\left(\dot u_s-\dot u_b\right)=0 \quad (6)$$

$$m_sh\left(\ddot u_g+h\ddot\phi_g+\ddot u_h+\ddot u_s+h\ddot\phi_r+h\ddot\phi_{rb}\right)+\left(m_bd^2/12\right)\ddot\phi_{rb}+k_{rb}\phi_{rb}+c_{rb}\dot\phi_{rb}=0 \quad (7)$$

Equation of motion in Matrix form

$$\begin{bmatrix} m_s & 0 & m_s & hm_s & hm_s \\ 0 & m_b & m_b & 0 & 0 \\ m_s & m_b & m_s+m_b & hm_s & hm_s \\ hm_s & 0 & hm_s & h^2 m_s & h^2 m_s \\ hm_s & 0 & hm_s & h^2 m_s & h^2 m_s + m_b . d^2/12 \end{bmatrix} \begin{Bmatrix} \ddot{u}_s \\ \ddot{u}_b \\ \ddot{u}_h \\ \ddot{\varphi}_r \\ \ddot{\varphi}_{rb} \end{Bmatrix} + \begin{bmatrix} c_s & -c_s & 0 & 0 & 0 \\ -c_s & c_s+c_b & 0 & 0 & 0 \\ 0 & 0 & c_h & 0 & 0 \\ 0 & 0 & 0 & c_r & 0 \\ 0 & 0 & 0 & 0 & c_{rb} \end{bmatrix} \begin{Bmatrix} \dot{u}_s \\ \dot{u}_b \\ \dot{u}_h \\ \dot{\varphi}_r \\ \dot{\varphi}_{rb} \end{Bmatrix} +$$

$$\begin{bmatrix} k_s & -k_s & 0 & 0 & 0 \\ -k_s & k_s+k_b & 0 & 0 & 0 \\ 0 & 0 & k_h & 0 & 0 \\ 0 & 0 & 0 & k_r & 0 \\ 0 & 0 & 0 & 0 & k_{rb} \end{bmatrix} \begin{Bmatrix} u_s \\ u_b \\ u_h \\ \varphi_r \\ \varphi_{rb} \end{Bmatrix} = - \begin{bmatrix} m_s \\ m_b \\ m_s+m_b \\ m_s h \\ m_s h \end{bmatrix} \{ \ddot{u}_g \} - \begin{bmatrix} m_s h \\ 0 \\ m_s h \\ m_s h^2 \\ m_s h^2 \end{bmatrix} \{ \ddot{\varphi}_g \} \tag{8}$$

Equivalent 3-DOF system

Although the simplified model with soil-structure interaction system has five degree-of-freedom, three of them are dynamic, since the foundation is assumed to be founded on a semi-infinite soil medium through a massless foundation and therefore no inertia forces are developed at the base of the structure. Therefore, an equivalent fixed-base three-degree-of-freedom (3-DOF) system can be developed from five-degree-of-freedom (5-DOF) via dynamic equilibrium. The procedure requires that the response of the equivalent system fixed-base 3- DOF system, when excited by a harmonic base excitation $\tilde{u}_g e^{i\omega t}$ should coincide with the dynamic response of the initial 5-DOF system.

For the equivalent fixed-base 3-DOF system shown in Fig. 4, the equations of motion in matrix formulation,

$$\begin{bmatrix} m_s & 0 & m_s h \\ 0 & m_b & 0 \\ m_s h & 0 & m_s h^2 + m_b \dfrac{d^2}{12} \end{bmatrix} \begin{Bmatrix} \ddot{u}_s \\ \ddot{u}_b \\ \ddot{\varphi}_{rb} \end{Bmatrix} + \begin{bmatrix} \tilde{k}_s & -\tilde{k}_s & 0 \\ -\tilde{k}_s & \tilde{k}_s+\tilde{k}_b & 0 \\ 0 & 0 & \tilde{k}_{rb} \end{bmatrix} \begin{Bmatrix} u_s \\ u_b \\ \varphi_{rb} \end{Bmatrix}$$

$$+ \begin{bmatrix} \tilde{c}_s & -\tilde{c}_s & 0 \\ -\tilde{c}_s & \tilde{c}_s+\tilde{c}_b & 0 \\ 0 & 0 & \tilde{c}_{rb} \end{bmatrix} \begin{Bmatrix} \dot{u}_s \\ \dot{u}_b \\ \dot{\varphi}_{rb} \end{Bmatrix} = - \begin{bmatrix} m_s \\ m_b \\ m_s h \end{bmatrix} \{ \ddot{u}_g \} \tag{9}$$

Equation of motion in frequency domain,

$$-m_s\omega^2 u_s - m_s h\omega^2 \varphi_{rb} + i\omega\tilde{c}_s (u_s - u_b) + \tilde{k}_s (u_s - u_b) = m_s\omega^2 \tilde{u}_g \tag{10}$$

$$-m_b\omega^2 u_b + i\omega\tilde{c}_b u_b - i\omega\tilde{c}_s (u_s - u_b) + \tilde{k}_b u_b - \tilde{k}_s (u_s - u_b) = m_b\omega^2 \tilde{u}_g \tag{11}$$

$$-m_s h\omega^2 u_s - m_s h^2\omega^2 \varphi_{rb} - m_b\omega^2 \frac{d^2}{12}\varphi_{rb} + i\omega\tilde{c}_{rb}\varphi_{rb} + \tilde{k}_{rb}\varphi_{rb} = m_s h\omega^2 \tilde{u}_g \tag{12}$$

The title symbol (~) denotes the parameters of the equivalent system: u_s, u_b and ϕ_{rb} are the structural, isolation horizontal and isolation rotational degrees-of-freedom respectively considered identical with the deformations of the initial system; $\tilde{k}_i$ and $\tilde{c}_s$ are the stiffness and damping coefficients of the i th degree-of-freedom and $\tilde{u}_g$ is the amplitude of the equivalent input ground motion.

Now, natural frequencies ($\frac{\tilde{\omega}_m^2}{\tilde{\omega}_s^2}$ where, m=1, 2, 3 are the natural horizontal frequencies of the isolation mode, natural horizontal frequencies of the structural mode and natural rotational frequencies of the isolation mode respectively.) of each mode and their corresponding eigen modes can be obtained by solving a series of mathematical derivations. The modal participation factors ($\tilde{\psi}$) also determined to find out the contributing mode.

RESULTS AND DISCUSSIONS

In the Fig. 5 to Fig. 7, natural frequencies of modes are shown with respect to equivalent horizontal stiffness ratio ($\tilde{R}_s$) and equivalent rotational stiffness ratio ($\tilde{R}_{rb}$) in 3D graph. The variation of the participation factors drawn in Fig. 8 to Fig. 10. It is clearly shows that Y_1 is greater than Y_2 and Y_3 for the whole range of the range of $\tilde{R}_s$ and $\tilde{R}_{rb}$.

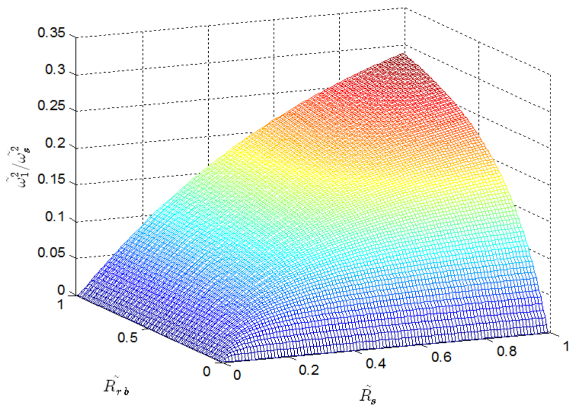


Fig. 5. Natural frequency of the isolation mass (h = 77.27 m, d = 46.94 m, γ = 0.75)

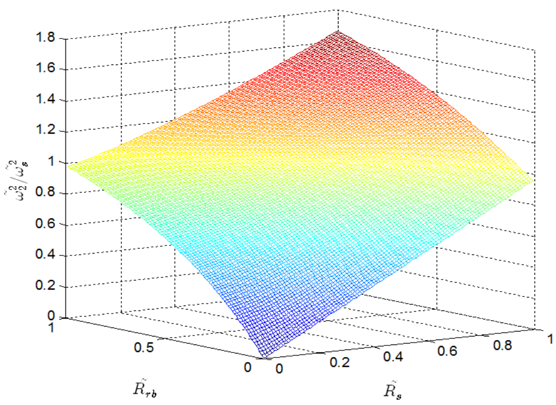


Fig. 6. Natural frequency of the structural mass (h = 77.27 m, d = 46.94 m, γ = 0.75)

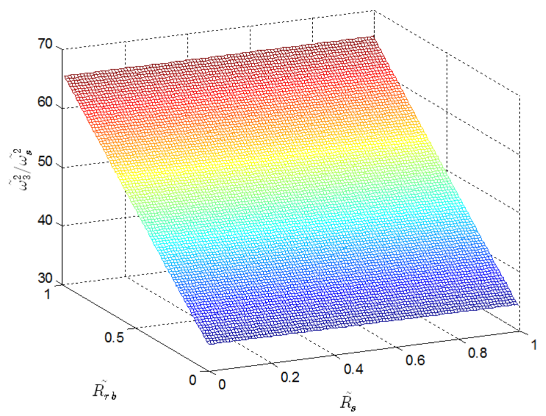


Fig. 7. Natural frequency of the rotational isolation mass (h = 77.27 m, d = 46.94 m, γ = 0.75)

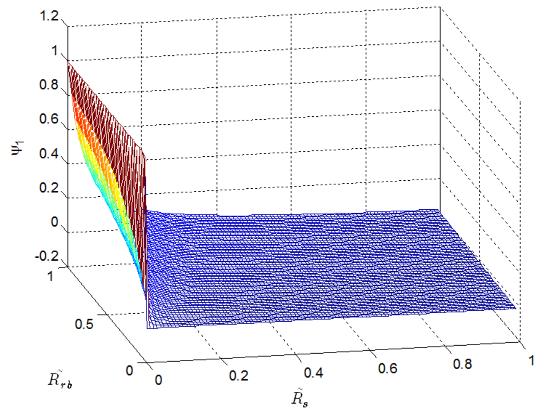


Fig. 8. Participation factor of isolation mode (h = 77.27 m, d = 46.94 m, γ = 0.75)

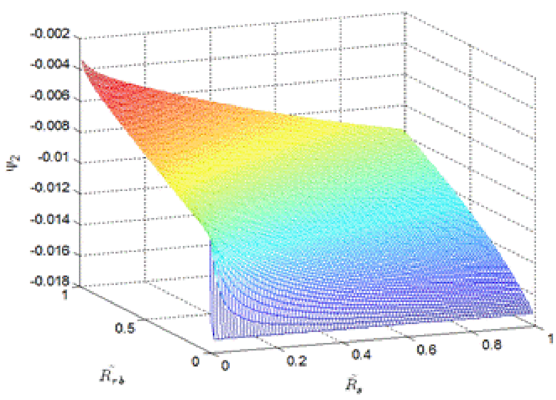


Fig. 9. Participation factor of structural mode (h = 77.27 m, d = 46.94 m, γ = 0.75)

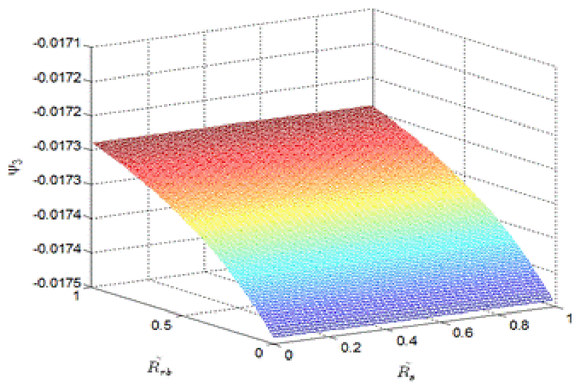


Fig. 10. Participation factor of rotational isolation mode (h = 77.27 m, d = 46.94 m, γ = 0.75)

CONCLUSION

From the above result shown in the graphs it can be easily deduced that isolation mode contributes more than other modes. It can be used in the preliminary design of containment of nuclear power plant in SSI analysis.

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SEISMIC PERFORMANCE OF A MOMENT RESISTING REINFORCED CONCRETE FRAME INCLUDING UNCERTAINTY

P. K. M. Moniruzzaman[1*], F. H. Oyshi[2], A. F. Farah[3] & F. M. Omar[4]

[1] *Department of Wood Science, The University of British Columbia, Vancouver, Canada.*
[2] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
[3] *Department of Structural Engineering, Sakarya University, Sakarya, Turkey.*
[4] *Department of Civil Engineering, EELO University, Borama, Somaliland.*
**Corresponding Author, <pkm.moni@alumni.ubc.ca>*

ABSTRACT

This study examines the seismic behaviour of a seven-storey moment resisting reinforced concrete frames under nine different ground motion (GM) records through incremental dynamic analysis (IDA). The IDA results allowed a thorough understanding of changes in the structural response as the intensity of the GM increases. The selected earthquake hazard is based on maximum considered earthquake ground motion. The seismic performance is quantified through nonlinear collapse simulation on a set of archetype models developed in the SeismoStruct platform. The drift behaviour, record-to-record variability of the response and height-wise distribution of the drift demand were reported. On the other hand, for collapse evaluation, ground motions are systematically scaled to increasing earthquake intensities until median collapse is established and analysed as a form of IDA. Using collapse data obtained from the IDA results, the collapse fragility curve are defined through a cumulative distribution function, which is related the ground motion intensity to the probability of collapse. In addition, the uncertainty is explicitly considered in the collapse performance evaluation. The calculated collapse margin ratio was unchanged for the cases of uncertainties.

Keywords: Incremental dynamic analysis, seismic capacity, inter-storey drift, reinforced concrete structure.

INTRODUCTION

Reinforced concrete (RC) frames consist of horizontal elements (beams) and vertical elements (columns) connected by rigid joints. These structures are cast monolithically- that is, beams and columns are cast in a single operation in order to act in unison. RC frames provide resistance to both gravity and lateral loads through bending in beams and columns. On the other hand, moment-resisting frames are rectilinear assemblages of beams and columns that resist forces by bending. In moment resisting frames, the joints, or connections, between columns and beams are designed to be rigid (ACI, 2005). The bending rigidity and strength of the frame members are the primary source of lateral stiffness and strength for the entire frame. Resistance to lateral forces is provided primarily by rigid frame action-that is, by the development of bending moment and shear force in the frame members and joints. At a rigid joint, the ends of the columns and beams cannot rotate. This means that the angle between the ends of the columns and beams always remain the same. This causes the columns and beams to bend during earthquakes depending on the geometry of the connection. Therefore, these structural members are designed to be strong in bending. Frequently, reinforced concrete construction is used in regions of high seismic risk. By virtue of moment resistance frames, rigid joints should be designed carefully to make sure they do not distort. However, the 1994 Northridge earthquake revealed a common flaw in the construction, and building codes (NBCC, 2005).

There is a lack of information about the dynamic performance of RC moment resistance frames. Thus, the progress and adoption of moment resistance frames, particularly in practice, has been hindered by the lack of performance-based criteria and design methodology for this type of structural system. To address this issue and examine the seismic response of this system under different earthquake records, incremental dynamic analyses (IDAs) were performed on seven-storey RC frame. For collapse evaluation, ground motions are systematically scaled to increasing earthquake intensities until median collapse is established and analysed the model as a form of IDA. Using collapse data obtained from IDA results, the collapse fragility curve defined through a cumulative distribution function, which related the ground motion intensity to the probability of collapse. In addition, the uncertainty is explicitly considered in the collapse performance evaluation.

METHODOLOGY

Description of the structure

In order to investigate the seismic performance of a RC moment resisting frame, a case study building was adopted which has 7-story, 166 m2 residential building located in Dhaka, Bangladesh (Figure 1). This structure is made of reinforced concrete frame building, is located on stiff soil and in an area in which near-fault ground motions are not prevalent (Zone 2 in BNBC 1993). In layout plan, the building has 19 m x 8.5 m and 4 bays x 2 bays (Figure 1). The long direction is oriented East-West. The building is approximately 21 m tall. The slabs are 115 mm deep. Columns in the south frame are 305 mm wide by 508 mm deep, i.e., oriented to bend in their weak direction when resisting lateral forces in the plane of the frame. Beams are generally 254 mm wide by 508 mm. The concrete has a nominal strength of 25 MPa and the reinforcement steel is scheduled as Grade 60 (400 MPa).

Finite element modelling and model validation

The building was modelled in a simulation environment, SeismoStruct 5.2.2 for analysis considering a 2D interior frame in the East-West direction. The concrete and steel materials were modelled using the built-in models in SeismoStruct. For instance, Menegotto-Pinto steel model and Mander et al. nonlinear concrete model were implemented. On the other hand, RC rectangular sections were used to model the beam and column sections. The beams were divided longitudinally into 5 elements and each beam and column element was divided transversely into 300 by 300 fibre elements. The model was validated against the time period of the structure as calculated according to BNBC code. In the current study, the time period was 0.47 second which is about 2% lower than the code value.

a) b) c)

Figure 1. The studied structure: a) Full scale building, b) building layout, c) Finite element model

INCREMENTAL DYNAMIC ANALYSIS

The IDA method involves performing a series of nonlinear time history analyses on the modelled structure subjected to one or more GM records (FEMA P695, 2009). Each record is scaled to several intensity levels so as to cover the entire range of structural response, from elastic behaviour through yielding to collapse (or until a defined 'failure' limit state occurs). In this study, the 5% damped spectral acceleration at the fundamental mode period of the structure, Sa(T1, 5%), was used as an intensity measure. The analysis was continued until Sa =2.4g or until numerical non-convergence occurred which indicated the global dynamic instability. The drift behavior, record-to-record variability of the response and height-wise distribution of drift demand were reported.

Selected ground motion records

A set of 9 GMs records are considered to be representative of events that have the potential to cause severe GMs at the considered location. The GM records were selected in a bin of relatively large magnitudes of 5.5– 7.6. Soil type C was considered for all the records. The selected ground motions were scaled with the Dhaka response spectrum by using SeismoMatch software. A typical time history records for the scalded records and the spectrums of the scaled records are shown in Figure 2.

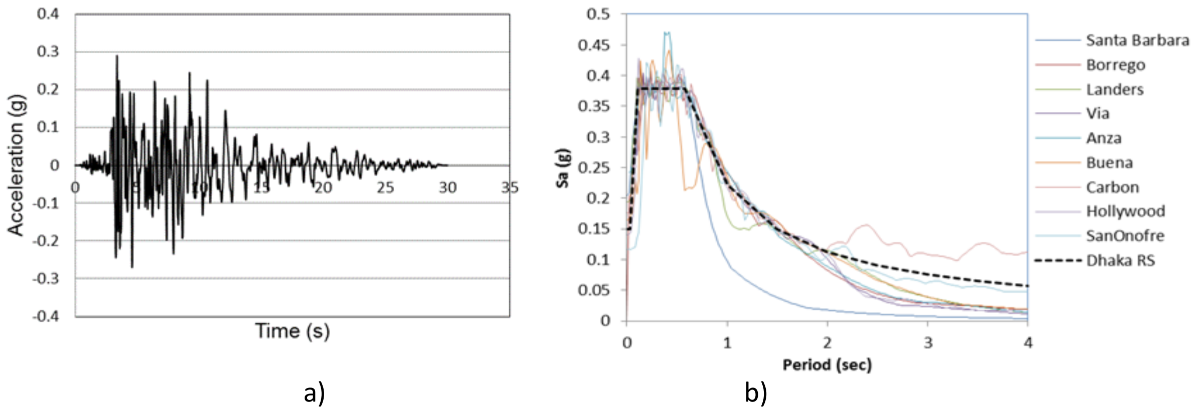
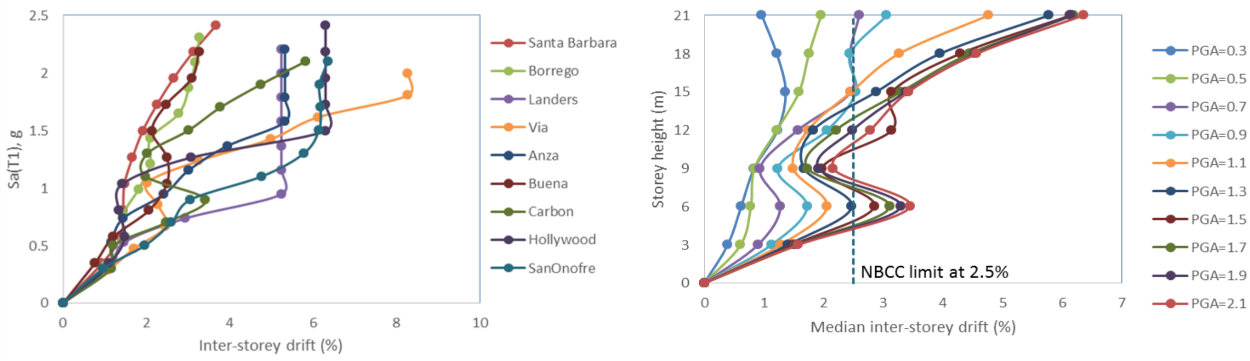


Figure 2. a) A typical scaled earthquake record, b) scaled GM with Dhaka response spectrum

RESULTS AND DISCUSSIONS

Figure 3 shows the IDA curves start as a straight line signalling the linear elastic range which stays straight up to 1.5% inter-storey drift ratio. Then the tangent slope changes as a result of nonlinearity. From the IDA curves, it is observed that the inter-storey drift demand varied in a wide range. For instance, at Sa=2.0 the inter-storey drift ratio for the 9 GMs varied from 2.2%-8.0% (Figure 3). On the other hand, from the median inter-storey drift ratio distribution along the height of the frame it can be concluded that the RC frame can be withstood up to Sa of 0.7g according to NBCC specified limit.

Seismic performance



a) b)

Figure 3. a) IDA curves for 9 GMs b) Median inter-storey drift ratios at different Sa (T1)

COLLAPSE MARGIN RATIO (CMR)

In order to quantify the safety, the collapse level ground motions are considered as the intensity that would result in median collapse of the seismic-force-resisting system, whereas, median collapse occurs when one-half of the structures exposed to this intensity of ground motion would have some form of life-threatening collapse. Thereafter, the collapse margin ratio, CMR, is the ratio of the median 5%-damped spectral acceleration of the collapse level ground motions to the 5%-damped spectral acceleration of the MCE ground motions at the fundamental period of the seismic-force-resisting system.

Using collapse data obtained from IDA results, the collapse fragility curve defined through a cumulative distribution function (CDF), which related the ground motion intensity to the probability of collapse. Figure 4 shows the probability of collapse of the given structure under a given ground motion. Besides, the CMR value for the given RC moment resisting frame building is obtained as of 4.72.

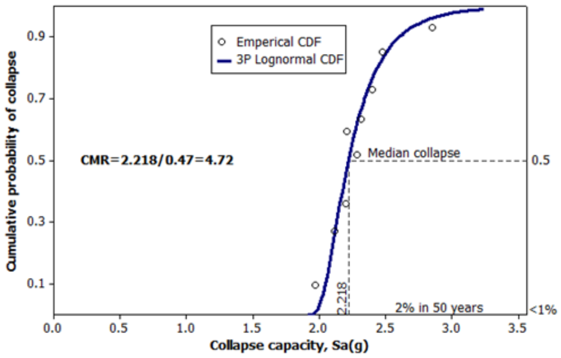


Figure 4. The collapse fragility curve

CMR INCLUDING UNCERTAINTY

In general, life safety risk (i.e., probability of death or life-threatening injury) is difficult to calculate accurately due to uncertainty in casualty rates given collapse. To account for unique characteristics of extreme ground motions that lead to building collapse, the CMR is converted to an adjusted collapse margin ratio. CMR is influenced by many factors, including ground motion variability and uncertainty in design, analysis, and construction of the structure. The considered weightage of these uncertainties are described in the following section

Record-to-record uncertainty

The uncertainty for record-to-record variability is taken in this study based on the following consideration: the frequency content and dynamic characteristics of the various records, and the hazard characterization. FEMA P695 document defined as a constant value for the Far-Field ground motion record set. Thus the study was considered the uncertainty for record-to-record variability (β_{RTR}) as of 0.40.

Design requirements-related uncertainty

The uncertainty for design requirements is taken in this study based on the following consideration: the structure was modelled with complete lateral- and vertical-force-resisting systems. Thus it was capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. According to Table 3-1 in FEMA P695 document, quality rating for design requirement (β_{DR}) was considered as of 0.35.

Test data related uncertainty

Analytical modelling alone is not adequate for predicting nonlinear seismic response with confidence, particularly for structural systems that have not been subjected to past earthquakes. A comprehensive experimental investigation program is necessary. The uncertainty for test data is taken in this study based on the following consideration: material test data, test set-up, loading, boundary and environmental conditions. Then according to Table 3-2 in FEMA P695 document, quality rating for test data (β_{TD}) was considered as of 0.20.

Modelling uncertainty

The uncertainty for modelling is taken in this study based on the following consideration: seismic behavioural effect for model idealization, and simulated and non-simulated collapse modes. Then according to Table 5-3 in FEMA P695 document, quality rating for modelling (β_{MDL}) was considered as of 0.35.

Total uncertainty

To calculate acceptable values of the adjusted collapse margin ratio, the total system uncertainty (β_{TOT}) is needed. The total uncertainty was accounted in the collapse fragility curve as the standard deviation of log-normal distribution. Following the below equation the value was calculated as of 0.67.

$$\beta_{TOT} = \sqrt{\beta^2{}_{RTR} + \beta^2{}_{DR} + \beta^2{}_{TD} + \beta^2{}_{MDL}} \qquad (1)$$

Adjusted CMR

The collapse fragility curves with and without considering uncertainties are shown in Figure 5. It shows that the probability of collapse under an earthquake of having Sa of 2g was 2% in case of without any consideration of uncertainty, whereas, this value increases to 16% while accounting all uncertainties. Moreover, the calculated CMR was unchanged for the cases whether including all uncertainties. This is because the median collapse and 2% in 50 years collapse capacity was unchanged for both cases. However, the probability of collapse found for the particular seismic-force-resisting system in this analysis when subjected to maximum considered earthquake can be considered as very low (0.03%). This low probability meets the primary life safety performance objective, results acceptable safety margin, i.e. CMR indicates the safe condition of the RC moment resisting frame building. Thus, result of this study provides insight into the collapse performance of the given structure.

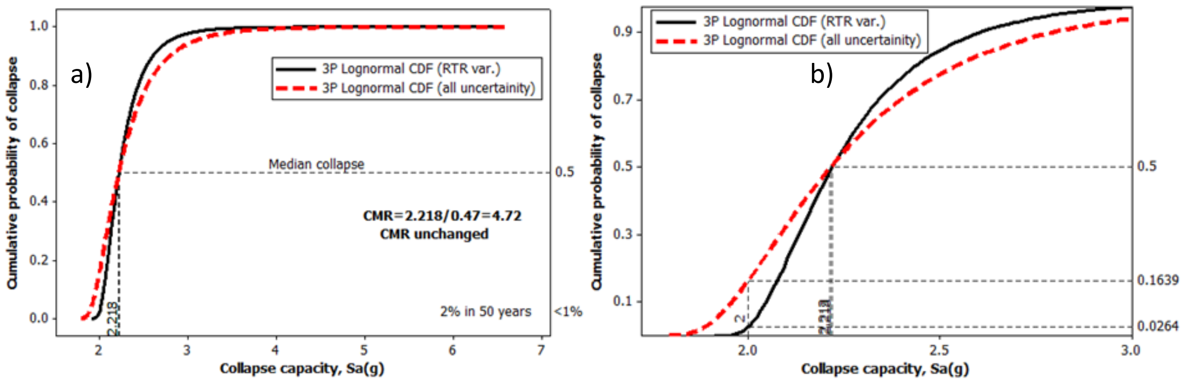


Figure 5. Collapse fragility curves: a) with and without uncertainty b) zoom-in up to Sa of 3g

CONCLUSION

The present study provided a better understanding for the seismic performance of a RC moment resisting frame. A nonlinear incremental dynamic analysis procedure was developed in a finite element program, SeimoStruct. The results showed that RC frame can be withstood up to Sa of 0.7g with compared to the NBCC inter-storey drift limit of 2.5%. Uncertainty is explicitly considered in the collapse performance evaluation. The calculated CMR was unchanged for the cases whether including all uncertainties.

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SEISMIC PERFORMANCE OF DIFFERENT HEIGHT REINFORCED CONCRETE STRUCTURES INCLUDING UNCERTAINTY

P. K. M. Moniruzzaman[1*], T. Biswas[2], & A. Shirwa[3]

[1] *Department of Wood Science, The University of British Columbia, Vancouver, Canada.*
[2]*Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
[3] *Department of Structural Engineering, Suleyman Demirel University, Isparta, Turkey.*
**Corresponding Author, <pkm.moni@alumni.ubc.ca>*

ABSTRACT

This study examines the seismic behaviour of a reinforced concrete structure having different heights under nine different ground motion (GM) records through incremental dynamic analysis (IDA). Three different heights of a structure, namely, 4, 7 and 10 storey were considered in this study. The selected earthquake hazard is based on maximum considered earthquake ground motions. The seismic performance is quantified through nonlinear collapse simulation on a set of archetype models developed in the SeismoStruct platform. The drift behaviour, record-to-record variability of the response and height-wise distribution of drift demand were reported. On the other hand, for collapse evaluation, ground motions are systematically scaled to increasing earthquake intensities until median collapse is established and analysed as a form of IDA. Using collapse data obtained from IDA results, the collapse fragility curve defined through a cumulative distribution function, which related the ground motion intensity to the probability of collapse. In addition, the uncertainty is explicitly considered in the collapse performance evaluation. The calculated CMRs were unchanged for the cases including all uncertainties.

Keywords: Incremental dynamic analysis, seismic capacity, inter-storey drift, reinforced concrete structure.

INTRODUCTION

Reinforced concrete (RC) frames consist of horizontal elements (beams) and vertical elements (columns) connected by rigid joints. These structures are cast monolithically- that is, beams and columns are cast in a single operation in order to act in unison. RC frames provide resistance to both gravity and lateral loads through bending in beams and columns. Frequently, reinforced concrete construction is used in regions of high seismic risk to provide resistance against moment force (ACI, 2005). By virtue of moment resistance frames, rigid joints should be designed carefully to make sure they do not distort. However, the 1994 Northridge earthquake revealed a common flaw in the construction, and building codes (NBCC, 2005). There is a lack of information about the dynamic performance of RC structures. Thus, the progress and adoption of this type of structural system, particularly in practice, has been hindered by the lack of performance-based criteria and design methodology. Thus the aim of this paper is to address this issue and examine the seismic response of this system having different heights through incremental dynamic analyses (IDAs). For collapse evaluation, ground motions are systematically scaled to increasing earthquake intensities until median collapse is established and analysed the model as a form of IDA. Using collapse data obtained from IDA results, the collapse fragility curve defined through a cumulative distribution function, which related the ground motion intensity to the probability of collapse. In addition, the uncertainty is explicitly considered in the collapse performance evaluation.

METHODOLOGY

Description of the structures

In order to investigate the seismic performance of RC structures, a case study building having different heights was adopted which has 166 m^2 area, located in Dhaka, Bangladesh. In the parametric study, three different heights of a structure, namely, 4, 7 and 10 storey were considered to predict the seismic response depends on the height of the structure. These structures are made of reinforced concrete frames, are located on stiff soil and in an area in which near-fault ground motions are not prevalent (Zone 2 in BNBC 1993). In layout plan, the buildings have 19 m x 8.5 m and 4 bays x 2 bays (Figure 1). The long direction is oriented East-West. The buildings are approximately 12, 21, and 30 m tall in the name of 4th, 7th and 10th storey, respectively. The slabs are 115 mm deep. Columns in the south frame are 305 mm wide by 508 mm deep, i.e., oriented to bend in their weak direction when resisting lateral forces in the plane of the frame. Beams are generally 254 mm wide by 508 mm. The concrete has a nominal strength of 25 MPa and the reinforcement steel is scheduled as Grade 60 (400 MPa.

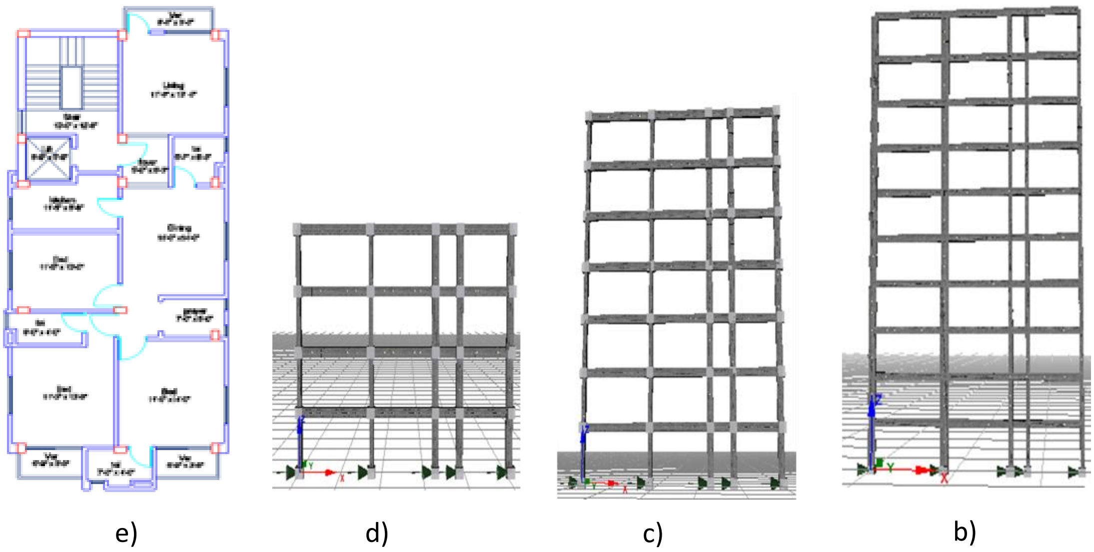


Figure 1. The studied structures: a) building layout, b) 4th storey, c) 7th storey, d) 10th storey

Finite element modelling and models validation

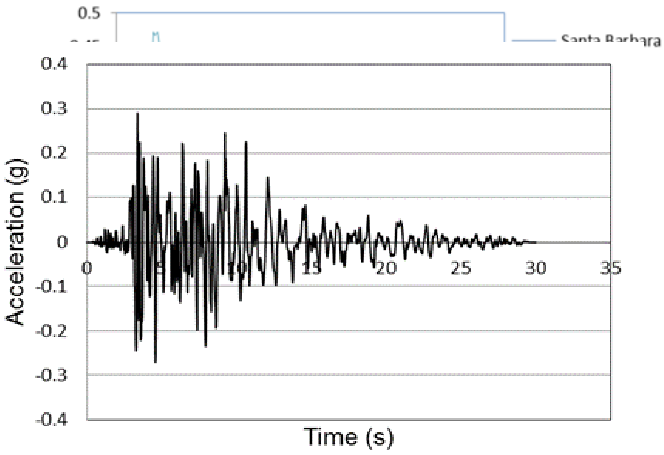
The buildings were modeled in a simulation environment, SeismoStruct 5.2.2 for analysis considering a 2D interior frame in the East-West direction. The modeled structures are shown in Figure 1. The concrete and steel materials were modeled using the built-in models in SeismoStruct. For instance, Menegotto-Pinto steel model and Mander et al. nonlinear concrete model were implemented. On the other hand, reinforced concrete rectangular sections were used to model the beam and column sections. The beams were divided longitudinally into 5 elements and each beam and column element was divided transversely into 300 by 300 fiber elements. The models were validated against the time period of he structure as calculated according to BNBC code. In the current study, the time period was 0.34, 0.47 and 0.73 second for the 4th, 7th, and 10th storey buildings, respectively, which are about 1-2% lower than the code calculated values.

INCREMENTAL DYNAMIC ANALYSIS

The IDA method involves performing a series of nonlinear time history analyses on the modelled structure subjected to one or more GM records (FEMA P695, 2009). Each record is scaled to several intensity levels so as to cover the entire range of structural response, from elastic behaviour through yielding to collapse (or until a defined 'failure' limit state occurs). In this study, the 5% damped spectral acceleration at the fundamental mode period of the structure, Sa(T1, 5%), was used as an intensity measure. The analysis was continued until Sa =2.4g or until numerical non-convergence occurred which indicated the global dynamic instability. The drift behavior, record-to-record variability of the response and height-wise distribution of drift demand were reported.

Selected ground motion records

A set of 9 GMs records are presumed to be representative of events that have the potential to cause severe GMs at the considered location. The GM records were selected in a bin of relatively large magnitudes of 5.5– 7.6. Soil type C was considered for all the records. The selected ground motions were scaled with the Dhaka response spectrum by using SeismoMatch software. A typical time history records for the scalded records and the spectrums of the scaled records are shown in Figure 2.



b)

Figure 2. a) A typical scaled earthquake record, b) scaled GM with Dhaka response spectrum

RESULTS AND DISCUSSIONS

Figure 3 shows the IDA curves start as a straight line signalling the linear elastic range which stays straight up to 4, 1.5 and 2.3% inter-storey drift ratio for the 4th, 7th and 10th storey buildings, respectively. Then the tangent slope changes as a result of nonlinearity. From the IDA curves, it is observed that the inter-storey drift demand varied in a wide range. For instance, at Sa=2.0 the inter-storey drift ratio for the 9 GMs varied from 2.2-3.7%, 2.2-8.0%, and 1.5-3.7% for the 4th, 7th and 10th storey buildings, respectively (Figure 3). On the other hand, from the median inter-storey drift ratio distribution along the height of the frame it can be concluded that the RC structures can be withstood up to Sa of 1.5g, 0.7g and 0.9g for the 4th, 7th and 10th storey structures, respectively,s according to NBCC specified limit.

4th Storey 7th Storey 10th Storey

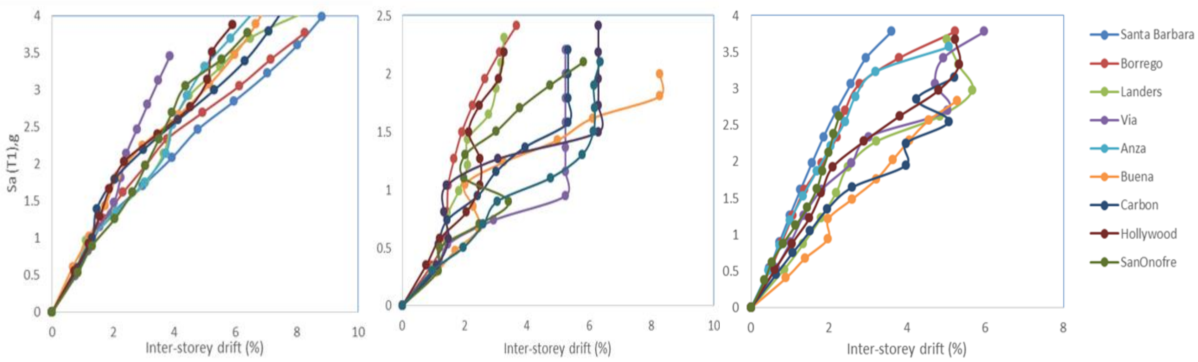


Figure 3. IDA curves of the studied structures for 9 GMs

SEISMIC PERFORMANCE

Collapse margin ratio (CMR)

In order to quantify the safety, the collapse level ground motions are considered as the intensity that would result in median collapse of the seismic-force-resisting system, whereas, median collapse occurs when one-half of the structures exposed to this intensity of ground motion would have some form of life-threatening collapse. Thereafter, the collapse margin ratio, CMR, is the ratio of the median 5%-damped spectral acceleration of the collapse level ground motions to the 5%-damped spectral acceleration of the MCE ground motions at the fundamental period of the seismic-force-resisting system.

Using collapse data obtained from IDA results, the collapse fragility curve defined through a cumulative distribution function (CDF), which related the ground motion intensity to the probability of collapse. Figure 4 and 5 show the probability of collapse of the given structures under a given ground motion. Besides, the CMR for the given RC structures are obtained as of 11.57, 4.72, and 5.35 for the 4th, 7th, 10th storey building, respectively.

CMR INCLUDING UNCERTAINTY

In general, life safety risk (i.e., probability of death or life-threatening injury) is difficult to calculate accurately due to uncertainty in casualty rates given collapse. To account for unique characteristics of extreme ground motions that lead to building collapse, the CMR is converted to an adjusted collapse margin ratio. CMR is influenced by many factors, including ground motion variability and uncertainty in design, analysis, and construction of the structure. The considered weightage of these uncertainties are described in the following section

Record-to-record uncertainty

The uncertainty for record-to-record variability is taken in this study based on the following consideration: the frequency content and dynamic characteristics of the various records, and the hazard characterization. FEMA P695 document defined as a constant value for the Far-Field ground motion record set. Thus the study was considered the uncertainty for record-to-record variability (β_{RTR}) as of 0.40.

Design requirements-related uncertainty

The uncertainty for design requirements is taken in this study based on the following consideration: the structure was modelled with complete lateral- and vertical-force-resisting systems. Thus it was capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. According to

Table 3-1 in FEMA P695 document, quality rating for design requirement (β_{DR}) was considered as of 0.35.

Test data-related uncertainty

Analytical modelling alone is not adequate for predicting nonlinear seismic response with confidence, particularly for structural systems that have not been subjected to past earthquakes. A comprehensive experimental investigation program is necessary. The uncertainty for test data is taken in this study based on the following consideration: material test data, test set-up, loading, boundary, and environmental conditions. Then according to Table 3-2 in FEMA P695 document, quality rating for test data (β_{TD}) was considered as of 0.20.

Modelling uncertainty

The uncertainty for modelling is taken in this study based on the following consideration: seismic behavioural effect for model idealization, and simulated and non-simulated collapse modes. Then according to Table 5-3 in FEMA P695 document, quality rating for modelling (β_{MDL}) was considered as of 0.35.

Total uncertainty

To calculate acceptable values of the adjusted collapse margin ratio, the total system uncertainty (β_{TOT}) is needed. The total uncertainty was accounted in the collapse fragility curve as the standard deviation of log-normal distribution. Following the below equation the value was calculated as of 0.67.

$$\beta_{TOT} = \sqrt{\beta^2{}_{RTR} + \beta^2{}_{DR} + \beta^2{}_{TD} + \beta^2{}_{MDL}} \qquad (1)$$

Adjusted CMR

The collapse fragility curves with and without considering uncertainties are shown in Figure 4 and 5. It shows that the probability of collapse under an earthquake of the same intensity of Sa increases when included all uncertainties. Moreover, the calculated CMRs were unchanged for the cases whether including all uncertainties. This is because the median collapse and 2% in 50 years collapse capacity was unchanged for both cases. However, the probability of collapse found for the particular seismic-force-resisting systems in this analysis when subjected to maximum considered earthquake can be considered as very low (0.01%, 0.03%, 0.02% for the 4th, 7th and 10th storey building, respectively). This low probability meets the primary life safety performance objective, results acceptable safety margin, i.e. CMR indicates the safe condition of the given RC structures. However, CMR of the 4th storey building indicates that it was very conservatively designed. Thus, results of this study provide insight into the collapse performance of the different height RC structures.

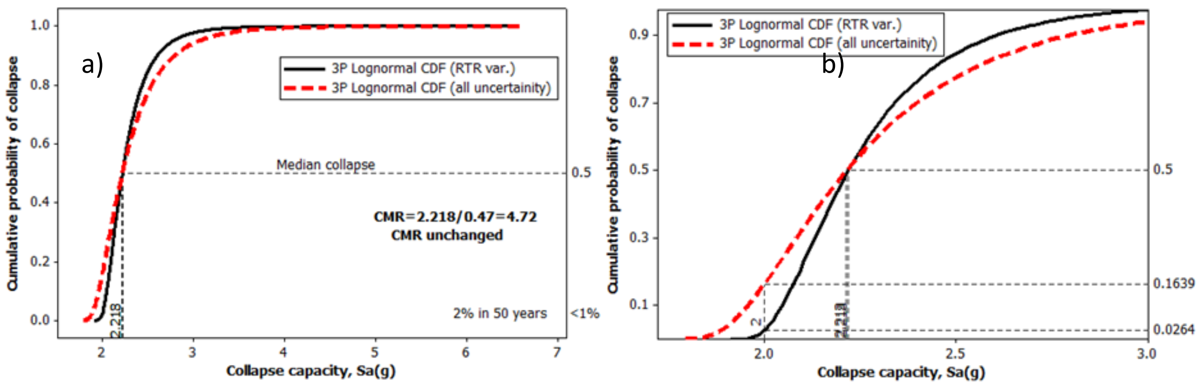


Figure 4. Collapse fragility curve for 7th storey: a) with & without uncertainty b) zoom-in up to Sa=3g

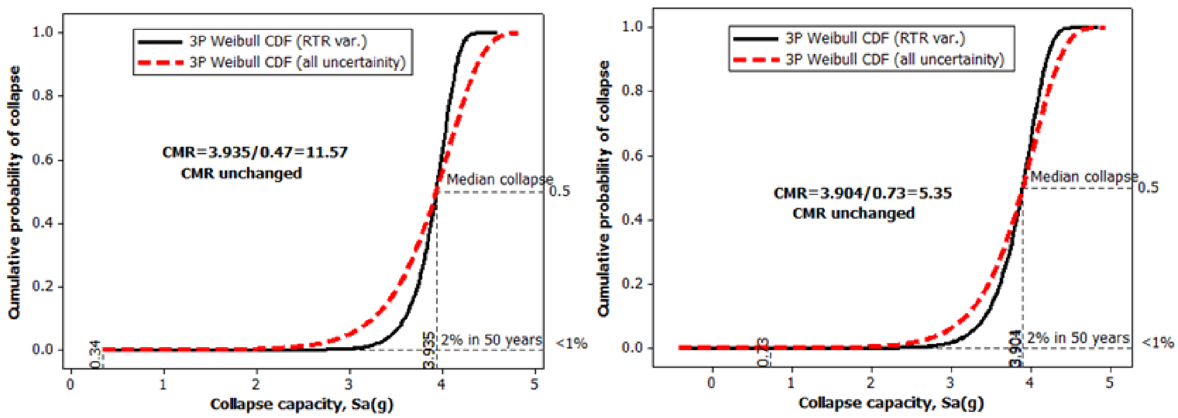


Figure 5. Collapse fragility curves: a) 4th storey, b) 10th storey

CONCLUSION

The present study provided a better understanding for the seismic performance of the RC structures having different heights. A nonlinear incremental dynamic analysis procedure was developed in a finite element program, SeimoStruct. The results showed that RC structures can be withstood up to Sa of 1.5g, 0.7g and 0.9g for the 4th, 7th and 10th storey structure, respectively, with compared to the NBCC inter-storey drift limit of 2.5%. Uncertainty is explicitly considered in the collapse performance evaluation. The calculated CMRs were unchanged for the cases whether including all uncertainties.

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FEMA P695. Quantification of Building Seismic Performance Factors. *Federal Emergency Management*

ID: SEE 081

NUMERICAL EVALUATION OF SEISMIC RESISTANCE OF MASONRY WALLS MADE WITH CLAY MASONRY UNITS

S. Biswas[1*] And A. R. Bhuiyan[2]

[1]*Department of Disaster and Environmental Engineering, Chittagong University of Engineering &Technology, Bangladesh,*
<shimul_bs@yahoo.com>
[2]*Department of Civil Engineering, Chittagong University of Engineering &Technology, Bangladesh,*
<arbhuiyance@cuet.ac.bd>

ABSTRACT

Unreinforced masonry (URM) walls in reinforced concrete frame building are used for functional or aesthetic reasons, rather than as structure element, their presence is generally ignored by the designers and no consideration is given to their own seismic safety or their effect on the performance of the structure. This study deals with numerical analysis of unreinforced masonry walls made with clay masonry units. The main purpose of this study is to observe the seismic resistance verification of unreinforced masonry walls made with clay masonry units. The numerical modeling of unreinforced masonry walls was developed on the basis of Finite Element Methods (FEM) using a professional nonlinear analysis computer program. As a result of the numerical analysis, it was observed that the load carrying capacity of unreinforced masonry walls depends on the types bond & presence of opening in the walls. The results obtained with the proposed model have a good agreement with those obtained in the laboratory tests.

Keywords: Unreinforced masonry (URM), Clay masonry units, Seismic resistance, Numerical modeling

1. INTRODUCTION

For many centuries and in different ways, masonry is one of the most commonly used and important construction materials around the world. Masonry construction is a traditional, widely used, extremely flexible and economical construction method with considerable potential for future developments. Despite this, nowadays there is a lack of information and research to characterize its structural performance. According to many studies, around 50% of the world's population lives in earthquake prone areas (Uzoegbo, 2011) and the effects of earthquakes worldwide have claimed approximately 8 million lives over the last 2,000 years (Jaiswal & Wald, 2008) and fatality rates are likely to continue to rise with increased population and urbanizations of global settlements especially in developing countries. Moreover, around 40% of urban the population lives in houses made of masonry and this number increases in under developed countries. In Chile, for instance, the percentage rises to about 50% (Barraza, 2012). Additionally, unreinforced masonry is responsible for approximately 60% of human casualties due to structural damage caused by earthquakes all over the world (Mayoeca & Meguro, 2013). The seismic performance of unreinforced masonry is very poor compared to other construction material, since it is a heavy, brittle material with low tensile strength and exhibits little ductility when subjected to seismic effects.

Taking these facts into account, it is clear that more research and development about the structural performance of masonry is needed, in order to improve its safety against seismic demands (Barraza, 2012). To assess vulnerability of such structures, a numerical tool is needed that can take into account non-linear behavior characteristics of masonry (Farshchi et al., 2009). Since the masonry constructions, are constructed by connecting bricks with the mortar, they do not usually form a medium. For this reason, it is quite difficult to introduce the behaviors of masonry walls with numerical methods (Kanit & Donduren, 2010). In this study, numerical modelling of unreinforced masonry walls was developed on the basis of Finite Element Methods (FEM).

2. TYPES OF BOND

The determination of the behavior of masonry walls, it is important factor to take into account the disposition of bricks or type of bond. Masonry is an organized disposition of bricks bonded with mortar and the way the bricks are organized may determine the structural response of the wall (Barraza, 2012). A general description of some of the most recognized types of bond (NCMA TEK, 2004) is those illustrated in Fig. 1.

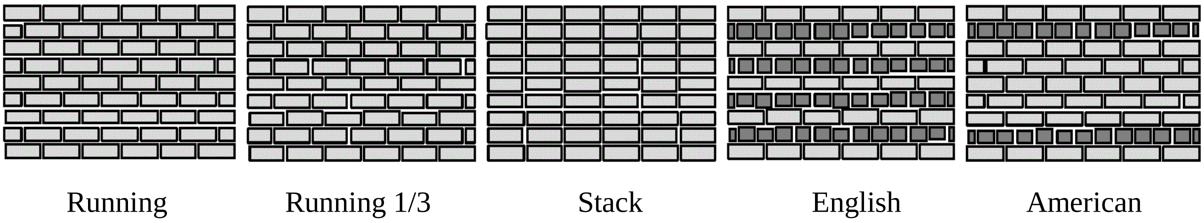


Fig. 1: Types of bond in masonry.

Running type of bond is the most common type that is frequently used in Bangladesh. For the model defined in this study, the running type of bond and stack type of bond are used for numerical analysis.

3. FAILURE MECHANISM OF MASONRY WALL

Unreinforced masonry wall is a composite structure that consists of brick and mortar and the brick is not strengthened with embedded steel bars. Masonry strength and stiffness depend on the properties of the bricks and mortar & also interaction between brick and mortar (Kelly, 1996). The failure of walls during earthquake is caused by combined effect of normal compressive and shear stress, which is represented by the principal tensile stress and when it exceeds, the diagonal tensile strength of the masonry failure will take place (Agarwal & Shrikhande, 2010). Masonry walls resisting in-plane loads usually exhibit the three modes of failure [Fig. 2]. Shear failure is exhibited when a wall is loaded with significant vertical as well as horizontal forces. This is the most common mode of failure (Maheri et al., 2011). Wall with poor shear strength, loaded predominantly with horizontal forces can exhibit the sliding failure mechanism. Rocking failure can occur where walls have improved shear resistance. This failure can occur due to small vertical loads, rather than high shear resistance. In this mode of failure the masonry panel can rock like a rigid body (Barraza, 2012).

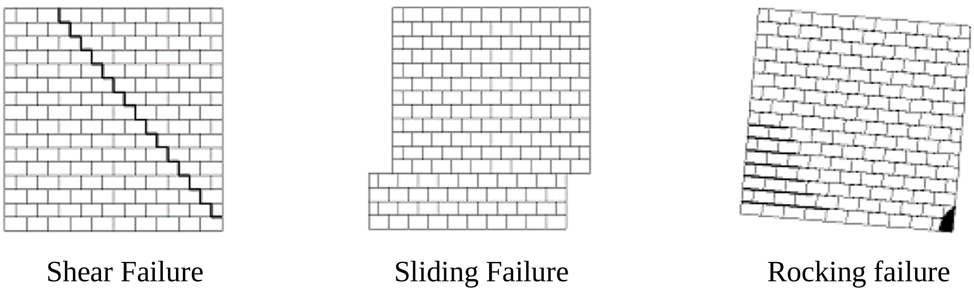


Fig. 2: Failure mechanisms of masonry walls.

3.1 EXPERIMENTAL INVESTIGATION

In Fig. 3, crack developed in the masonry wall due to shear failure which is tested in University of Kassel and T U Munich under ESECMaSE project. The first visible cracks in the masonry WALL 01 and WALL 02 occurred at a horizontal load of about respectively 70 KN & 63 KN. For WALL 03 first cracks were visible in the upper left corner of the wall at a horizontal force of about 90 KN.

(a)WALL 01 (UNIK) (b) WALL 02 (TUM) (c) WALL 03 (UNIK)

Fig. 3: Crack pattern of the masonry wall.

4. NUMERICAL MODELLING

ANSYS (V.11 SP1) finite element software is used to model the unreinforced masonry walls where modeling is done by two approaches (i.e. macro element and micro element) (Lourenco, 1996). In macro element approach (Fig. 4.a), considers the bricks and mortar as a unit element and the property of brick and mortar are considered together. This type of model should be able to reproduce the general structural behavior of a masonry panel but it is not able to reproduce all the types of failure mechanisms (Barraza, 2012).

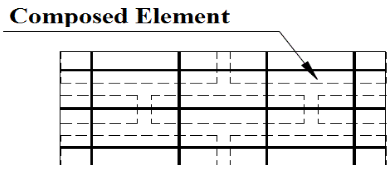


Fig. 4.a: Macro model approach.

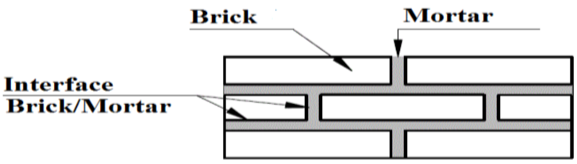


Fig. 4.b: Micro model approach.

In micro element approach (Fig. 4.b), brick and mortar are modeled separately and the property of each one will be assigned individually (Taghikhany et al., 2008). The interface between bricks and mortar is modeled by special elements that represent the discontinuities. In this case, the geometry of the wall is completely reproduced and it can represent most failure mechanisms in masonry.

4.1 ELEMENT TYPES

The SOLID65 & COMBIN7 element are used for numerical modeling of unreinforced masonry wall.

4.1.1 SOLID65

Solid65, an eight-node solid element, was used to model the clay brick unit and mortar (Induprabha, 2013). The solid65 [Fig. 5] element has eight nodes with three degrees of freedom at each node, translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in tension in three orthogonal directions and crushing in compression. The element behaves in a linear elastic manner until either the specific tensile or compressive strength is exceeded. The element has 8 integration points. The presence of cracking or crushing inside an element is verified at the integration points (ANSYS Release 12.1, 2009).

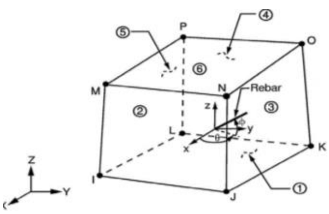


Fig. 5: Geometry of element SOLID65.

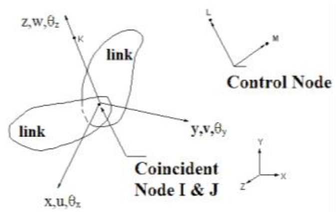


Fig. 6: Geometry of element COMBIN7.

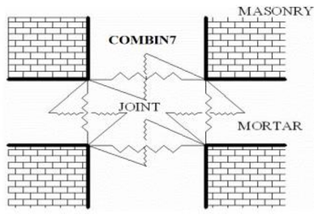


Fig. 7: Mortar-brick joint interface modeling.

4.1.2 COMBIN7

In this study, masonry units are linked to the mortar units by series of COMBIN7 element [Fig. 6]

(Taghikhany, 2008). COMBIN7 is a 3-D pin or revolute joint which is used to connect two or more parts of a model at a common point (Kamanli et al., 2011). Capabilities of this element include joint flexibility (or stiffness), friction, damping, and certain control features. An important feature of this element is a large deflection capability in which a local coordinate system is fixed to and moves with the joint (ANSYS Release 12.1, 2009).

4.2 MATERIAL PROPERTIES

The Solid65 element requires linear elastic and nonlinear inelastic material properties to properly model the brick and mortar unit. In order to define the failure of a material is essential to have a "failure criteria". In this study Willam and Warnke failure surface criterion are used to model the brick and mortar unit. Material property of the brick & mortar element is brought in [Table 1 & 2].

Table 1: Material property for macro & micro element in ANSYS (Taghikhany et al.,2008).

Material Property	Macro element	Micro element	
	Composed Unit	Brick Unit	Mortar Unit
Young's Modulus, E (MPa)	12930	12930	12500
Poisson's ratio, ν	0.2	0.19	0.17

Table 2: William and Warnke failure Surface criterion for macro & micro element (Willim & Warnke, 1974).

Material Property	Macro element	Micro element	
	Composed Unit	Brick Unit	Mortar Unit
Shear transfer coefficient for open crack, β_t	0.3	0.3	0.3
Shear transfer coefficient for closed crack, β_c	1.0	1.0	1.0
Uni-axial cracking stress, ft (MPa)	0.5	2.0	0.45
Uni-axial crushing stress, fc (MPa)	15.0	15.0	5.0

The shear transfer coefficient β represents conditions of the crack face. The value of β ranges from 0 to 1, with 0 representing a smooth crack (complete loss of shear transfer) and 1 representing a rough crack (ANSYS Release 12.1, 2009). In this study the shear transfer coefficient was taken as 0.3 for open crack and 1.0 for closed crack.

4.3 DESCRIPTION OF MODEL

The proposed model was implemented and applied to five unreinforced masonry walls [Fig. 8, 9, 10, 11 & 12] and same dimensions [Table 3] are used for both macro and micro model of all masonry walls.

Table 3: Dimension of macro and micro model

Length, mm	Height, mm	Width, mm	Opening , mm^2
2500	2500	152.5	660644

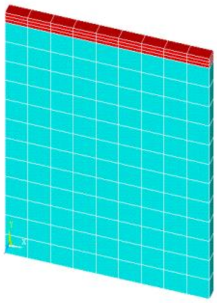


Fig. 8: Macro modeling.

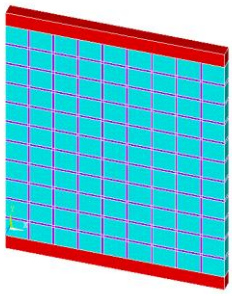


Fig. 9: Micro modeling (Stack Bond).

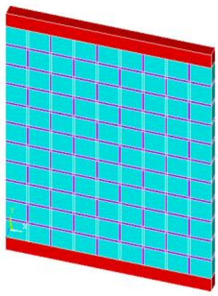


Fig. 10: Micro modeling (Running Bond).

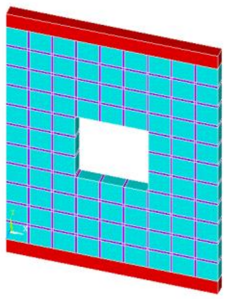


Fig. 11: Micro modeling (Stack bond with opening).

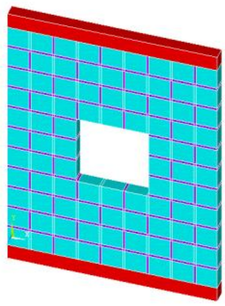
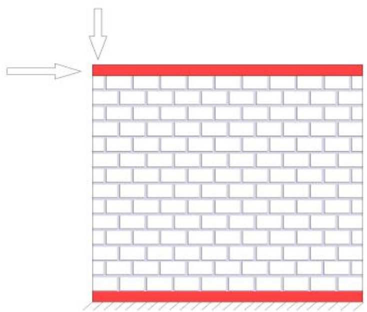


Fig. 12: Micro modeling (Running bond with opening).

4.4 LOADING SCHEME AND BOUNDARY CONDITION

Nonlinear static analyses are carried out using Newton Raphson approach, which provides convergence at the end of each load step within given tolerance limits (Biarnason, 2008). The simulation involves application of incremental horizontal and vertical load apply to the top of the walls (Dizhur and Ingham, 2013) and adopting a fixed support condition along the base of the unreinforced masonry wall. An automatic load incrimination scheme is included to the model & failure mechanism is obtained from different load steps and sub steps. Loading and boundary conditions used in the model are illustrated in Fig. 13.



Fixed support ($U_{X,Y,Z} = R_{X,Y,Z} = 0$)

Fig. 13: Loading scheme and boundary conditions (Maheri et al., 2011).

5. RESULT AND DISCUSSIONS

The crack pattern, deformed shape and force - displacement curve of the masonry walls are illustrated in the Fig. 14 to Fig. 28. In macro model, first visible cracks devolved at about 88.7 KN at load substeps 8. In micro model with stack and running bond, first visible cracks developed respectively at about 76.23 KN & 80.6 KN at load substeps 4. For opening in micro model with stack and running bond, first visible cracks developed respectively at about 29.06 KN & 34.5 KN at load substeps 4.

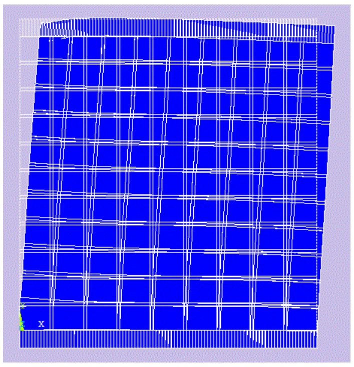
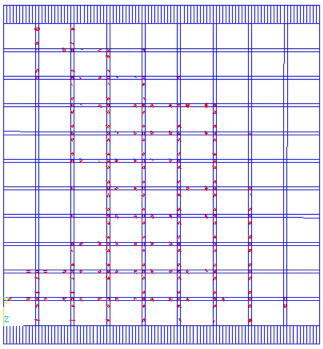
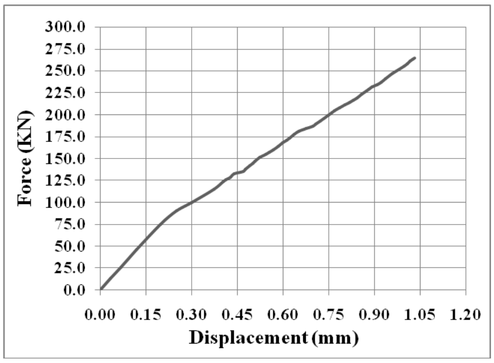




Fig. 14: Deformed shape (Micro model- Stack Bond). Fig. 15: Crack pattern (Micro model- Stack Bond). Fig. 16: Force- Displacement curve (Micro model- Stack Bond).

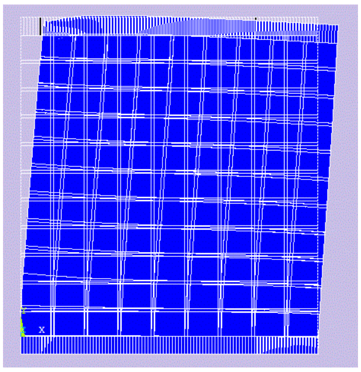
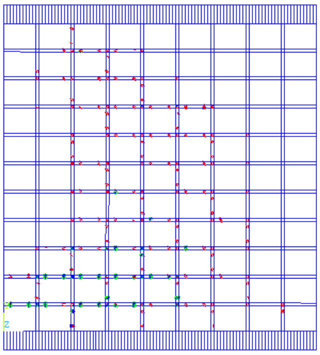
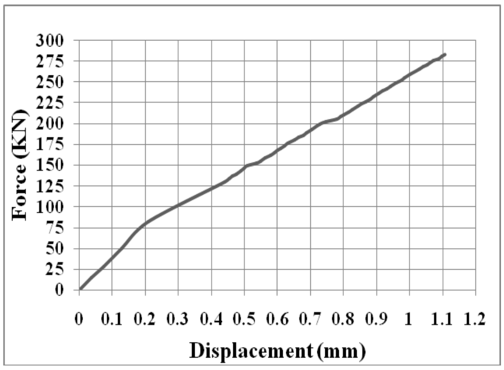




Fig. 17: Deformed shape (Micro model-Running Bond). Fig. 18: Crack pattern (Micro model-Running Bond). Fig. 19: Force- Displacement curve (Micro model -Running Bond).

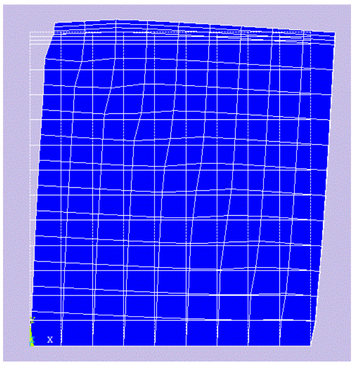
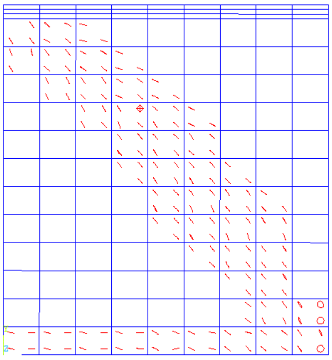
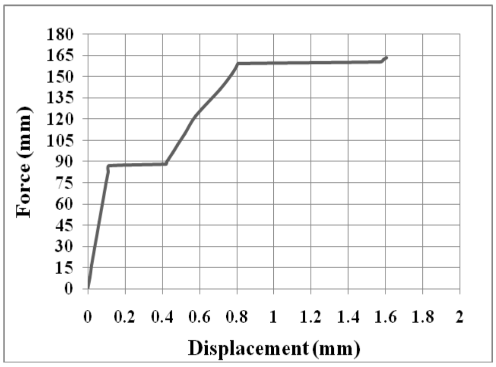




Fig. 20: Deformed shape (Macro model). Fig. 21: Crack pattern (Macro model). Fig. 22: Force- Displacement curve (Macro model).

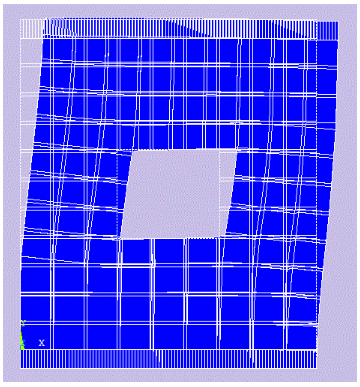


Fig. 23: Deformed shape (Micro model with opening - Stack Bond).

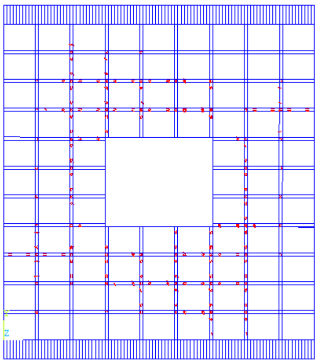


Fig. 24: Crack pattern (Micro model with opening -Stack Bond).

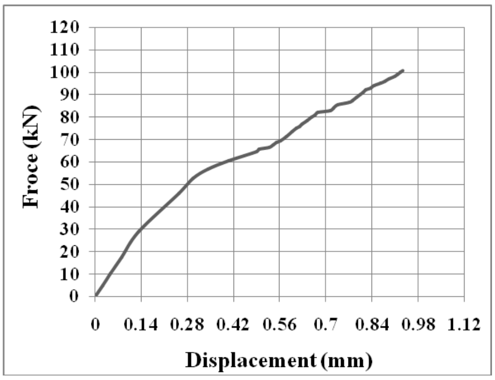


Fig. 25: Force- Displacement curve (Micro model with opennig -Stack Bond).

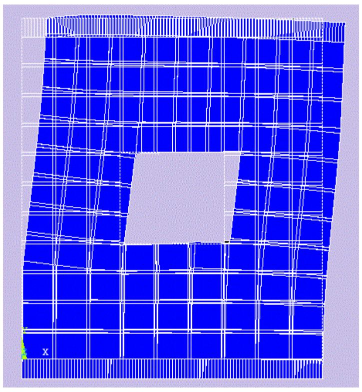


Fig. 26: Deformed shape (Micro model with opening - Running Bond).

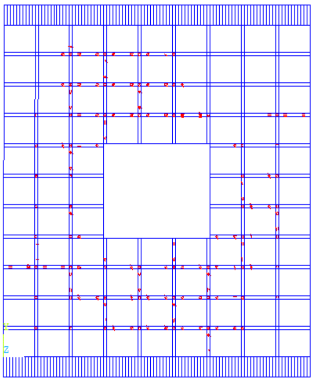


Fig. 27: Crack pattern (Micro model with opening -Running Bond).

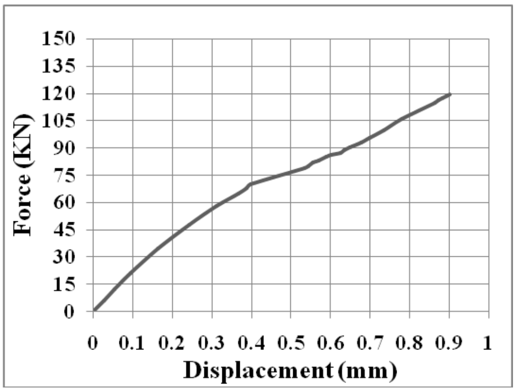


Fig. 28: Force- Displacement curve (Micro model with opennig - Running Bond).

6. CONCLUSION

In this study, evaluation of seismic resistance of masonry walls was invested by numerical analysis using nonlinear analysis computer program ANSYS. Where micro model generated marginally better results than the macro model but required more resources for analysis. Results show that, the load carrying capacity of unreinforced masonry walls also depends on the types bond & presence of opening in the walls. Unreinforced Masonry wall made with running type bond gives more robustness than stack type bond. Use of opening or window in unreinforced masonry wall, reduce the load carrying capacity nearly 50%. The proposed model showed good agreement with the results of the laboratory tests and may be considered as appropriate to represent the non-linear behavior of a single masonry wall, taking into account the alternatives of unreinforced masonry.

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SEISMIC PERFORMANCE OF FLAT PLATE STRUCTURES DESIGNED ONLY FOR GRAVITY LOADS

M R Islam[1] and T R Hossain[2]

[1]*Department of Civil & Environmental Engineering, Shahjalal University of Science & Technology (SUST), Sylhet, Bangladesh, <rafiq_cuet@yahoo.com>*
[2]*Department of Civil Engineering, Bangladesh University of Engineering & Technology (BUET), Dhaka, Bangladesh, <tahsin@ce.buet.ac.bd>*

ABSTRACT

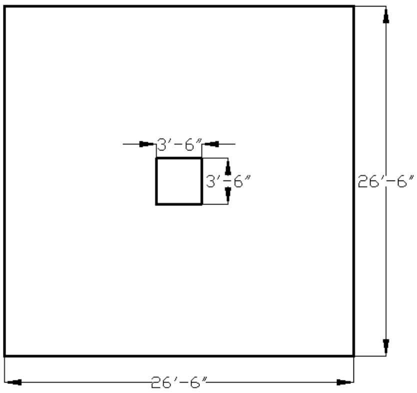
Bangladesh lies in moderate to high seismic zones. Many high-rise buildings are being constructed in different cities of the country. Bangladesh National Building Code (BNBC, 2006) provides seismic design methods to be employed in design of these high-rise buildings. As per BNBC, flat plate structures, as a part of lateral load carrying system is not permitted in high-seismic zone. However, many flat plate building have already been constructed in high-seismic zones, i.e. in Sylhet. Also, there are many flat plate buildings which have been designed without considering the seismic loads at all. For flat plates, the region around the column is always the critical location as it transfers combined gravity and lateral loads in a relatively small shallow section. Unbalanced moments generated due to seismic action need to be transferred to column from the flat plate. These moments are large, particularly for high-rise building structure having large span length and pose a problem in the design of the connection. It has been shown, for a 14-storied building without shear wall if only designed for gravity loads, is inadequate to transfer the unbalanced seismic moment for zone 2 and zone 3. Introduction of a proper shear wall reduces the unbalanced moment in column significantly and hence gravity load design proves sufficient for zone 2 and zone 3.

Keywords: Flat plate, Moment resisting frame, Shear-wall, Seismic load, ETABS Software.

INTRODUCTION

Bangladesh and the north-eastern Indian states have long been one of the seismically active regions of the world and have experienced numerous large earthquakes during the past 200 years. In geological point of view, part of Bangladesh is situated in moderate to high seismic zones. However, in the country a rapid urbanization is going on. With this, now-a-days flat plate structures are very much popular for its advantages regarding use and constructability. Flat plates, being thin members, are uneconomical from steel point of view, but they are economical in terms of formwork. However, in the design of reinforced concrete flat plates, the regions around the column always pose a critical analysis problem. When exposed to seismic loads, the performance of slab-column frames has often been less than satisfactory. Brittle punching failures of flat plates have been observed during several earthquakes as documented by AISI (1964) [1] and Mitchell and co-workers (1990 [2] and 1995 [3]). Moreover, flat plate as part of lateral load carrying system is not permitted in high seismic zone according to ACI 318 (2008) [4]/BNBC (2006) [5] as slab-column connection performance are not satisfactory in carrying seismic loads. So it is important to understand the slab-column connection at critical section of flat plates. Large research efforts have been made in the past and are still being continued to establish rather restrictive rules for flat-slab systems in earthquake prone regions. It has also inspired researchers to start extensive experimental work, and to develop new ways to make the connections stronger and more ductile in order to allow more widespread use of flat slab systems in seismic zones. In the mid-seventies, Hawkins, Mitchell and Hanna (1975) [6] were the first to research the effects of lateral loads. Numerous tests [Ghali, Elmasri and Dilger (1976) [7], Pan and Moehle (1989) [8], Cao (1993) [9], Dilger and Cao (1994) [10]] have been carried out to evaluate the behaviour of slab-column connection at critical section.

In this study, consider a high- rise building structure up to fourteenth storey having flat plate slab system of large span length. For such type of flat plate structures in regions of high seismic risk, the design considering only gravity load based on direct design method is safe and efficient up to which floor level have discussed. Better lateral load carrying system such as shear wall is discussed on flat plate high- rise building structure to resist most efficiently the various combinations of gravity and lateral loading. Computer software 'ETABS' have used for equivalent frame analysis based on elasticity. Finally investigate the results from analysis considering seismic Zone-2 and Zone-3 according to BNBC (2006) for seismic response.



MATERIALS AND METHODS

A typical interior frame of a fourteen storied flat plate commercial building was considered having 26.5 ft square slab, supported on a column of area 42″ × 42″ as shown in [Fig. 1]. The thickness of the slabs is 10 inches by ensuring that punching shear failure has not occurred according to ACI 318-08 and BNBC (2006) code provision. In this modelling the dead load with self weight is 200 psf and the live load is 80 psf as it is a commercial building. The load combination and seismic zone have considered according to BNBC (2006) code provision. The load combinations are given in "Eq. (1)", "Eq. (2)" and "Eq. (3)". Zone-2 and Zone-3 have selected to analyze the frames in both cases like without shear wall and with shear wall considering lateral loads as the above mentioned commercial building is considered at Dhaka city and Sylhet city respectively. For the modelling of this flat plate structure, concrete compressive strength and steel yield strength have considered 3500 psi and 60 ksi respectively.

$$U = 1.4\,DL + 1.7\,LL \tag{1}$$

$$U = 0.75\,[1.4\,DL + 1.7\,LL + 1.7\,(1.1\,E)] \tag{2}$$

$$U = 0.9\,DL + 1.3\,(1.1\,E) \tag{3}$$

Flat plate structures whereas the columns are cast integrally with the floor slabs behave similar to moment resisting frames under horizontal loading. The lateral deflections of the structure are a result of simple double curvature bending of the columns and a more complex three-dimensional form of double bending in the slab. The response of the structure can be studied by considering each bay-width replaced by an equivalent frame bent. The slab is replaced for the analysis by an equivalent beam with the same double bending stiffness as shown in [Fig. 2].

A shear wall structure is considered to be one whose resistance to horizontal loading is provided entirely by shear walls and floors acting as horizontal diaphragms transmit lateral loads equally to the shear walls as shown in [Fig. 3]. They are usually continuous down to the base to which they are rigidly attached to form vertical cantilevers. Their high inplane stiffness and strength makes them well suited for carrying gravity loading simultaneously. The distribution of lateral forces to the shear walls is a function of the geometrical arrangement of the resisting wall systems. It is usual to locate the walls on plan so that they attract an amount of gravity loading sufficient to suppress the maximum tensile bending stresses in the wall caused by lateral loading.

Figure 1. Plan view of interior panel of flat plate slab system

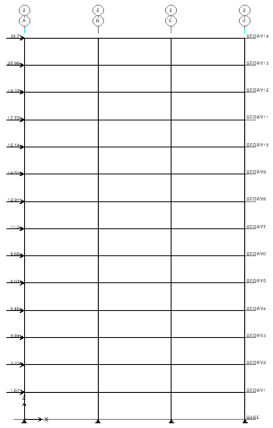


Figure 2. Flat plate structure similar to moment resisting frame under horizontal Loading

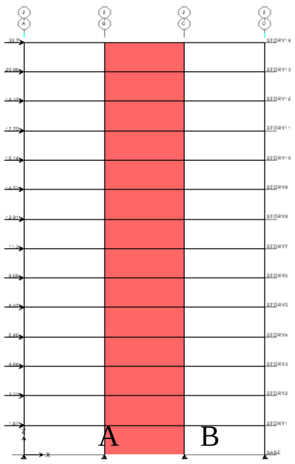


Figure 3. Flat plate structure similar to moment resisting frame with shear wall under horizontal loading

RESULTS AND DISCUSSIONS

The horizontal stiffness of the above discussed frame as shown in [Fig. 2] is governed mainly by the bending resistance of the slab-column connections. The accumulated horizontal shear above any story of a frame is resisted by shear in the columns of that story. The shear causes the story-height columns to bend in double curvature with points of contraflexure at approximately mid-story-height levels. The moments applied to joint from the columns above and below the slab are resisted by the slab both sides, which also bend in double curvature, with points of contraflexure at approximately midspan. These deformations of the columns and slabs allow racking of the frame and horizontal deflection in each story. The overall deflected shape of a moment resisting frame due to racking has a shear configuration with concavity in upward direction, a maximum inclination near the base and a minimum inclination at the top of the structure as shown in [Fig. 4].

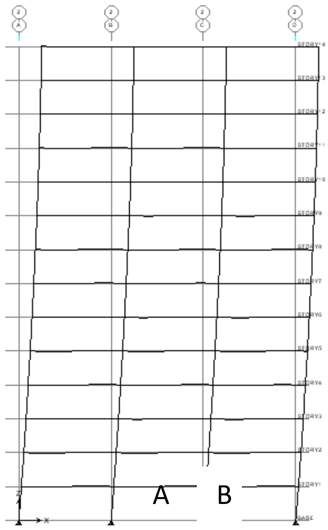


Figure 4. Overall deflected shape of moment resisting frame

Based on direct design method, flat plate slab system with interior panel column strip negative moment is 355 kip-ft due to only gravity loads considering load combination of Eq. (1). However, due to combined gravity and lateral loads considering load combinations of Eq. (2) and Eq. (3) more negative unbalanced moments and positive unbalanced moments have come at slab-column joint respectively. In Fig. 2, side 'A' and side 'B' have specified at bottom storey level of an interior panel slab-column connection whereas the unbalanced moments have compared because of maximum moment will come at bottom storey level due to lateral loads. The unbalanced moments due to only gravity loads and combined gravity and lateral loads at side 'A' and side 'B' of bottom storey level for above discussed frame considering up to fourteen storied building is given in Table 1 and Table 2 for zone 2 and zone 3 respectively. In Table 1 and Table 2, up to which storey level is safe among all thirteen buildings up to fourteen stories of flat plate structures by design only for gravity load based on direct design method can be decided easily by making a comparison in between two moments at bottom storey level due to only gravity loads and combined gravity and lateral loads.

In a moment resisting frame with shear wall as shown in [Fig. 3], the shear may be assumed to be resisted completely by the core as a first approximation. This is because its stiffness is so much greater than the lateral stiffness of the frame. The lateral rigidity is greatly improved to resist lateral forces by using not only the shear wall but also for combined shear wall and rigid frame system. The total deflection of the interacting shear wall and rigid frame systems is obtained by superimposing the individual modes of deformation as shown in [Fig. 5]. In case of moment resisting frame, the slope of the deformation is greatest at the base of the structure where the maximum shear is acting. On the other hand in case of shear wall, the slope of the deflection is greatest at the top of the building, indicating that in this region, the shear wall system contributes the least stiffness. However, the interaction of frame and shear wall is obtained by superimposing the separate deflection modes resulting in a flat-S-curve. Because of the different deflection characteristics of shear wall and frame, the shear wall is pulled back by the frame in the upper portion of the building, and pushed forward in the lower.

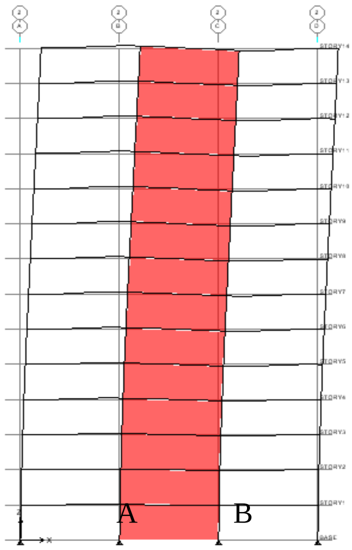


Figure 5. Overall deflected shape of moment resisting frame with shear wall

Again based on direct design method, flat plate slab system with interior panel column strip negative moment is 355 kip-ft due to only gravity loads considering load combination of Eq. (1). However, by using shear wall with flat plate structure subjected to combined gravity and lateral loads considering load combinations of Eq. (2) and Eq. (3) a very few unbalanced moments have come at slab-column joint. In Fig. 3, side ‘A’ and side ‘B’ have specified again at bottom storey level of an interior panel slab-column connection whereas the unbalanced moments have compared. The unbalanced moment due to only gravity loads and combined gravity and lateral loads at side ‘A’ and side ‘B’ of bottom storey level for above discussed frame with shear wall considering up to fourteen storied building is given in Table 3 and Table 4 for zone 2 and zone 3 respectively. After investigate the results from Table 3 and Table 4, it can be decided that by using shear wall in a flat plate structures can minimize the unbalanced moments at bottom storey level for up to fourteen storied building and can design the building only for gravity load based on direct design method of flat plate structure safely under combined gravity and lateral loads.

Table 1 Unbalanced moments due to only gravity loads and combined gravity and lateral loads at side ‘A’ and side ‘B’ of bottom storey level for zone 2

Storey	Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘A’			Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘B’		
	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks
Two Storied Building	-355	-343	Safe	-355	-191	Safe
Three Storied Building	-355	-393	Not safe	-355	-141	Safe
Four Storied Building	-355	-452	Not safe	-355	+34	Not safe
Five Storied Building	-355	-517	Not safe	-355	+99	Not safe
Six Storied Building	-355	-557	Not safe	-355	+140	Not safe
Seven Storied Building	-355	-579	Not safe	-355	+162	Not safe
Eight Storied Building	-355	-610	Not safe	-355	+194	Not safe

Nine Storied Building	-355	-639	Not safe	-355	+223	Not safe
Ten Storied Building	-355	-666	Not safe	-355	+251	Not safe
Eleven Storied Building	-355	-691	Not safe	-355	+276	Not safe
Twelve Storied Building	-355	-715	Not safe	-355	+300	Not safe
Thirteen Storied Building	-355	-737	Not safe	-355	+323	Not safe
Fourteen Storied Building	-355	-758	Not safe	-355	+344	Not safe

Table 2 Unbalanced moments due to only gravity loads and combined gravity and lateral loads at side ‘A’ and side ‘B’ of bottom storey level for zone 3

Storey	Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘A’			Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘B’		
	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks
Two Storied Building	-355	-393	Not safe	-355	-141	Safe
Three Storied Building	-355	-478	Not safe	-355	+59	Not safe
Four Storied Building	-355	-576	Not safe	-355	+159	Not safe
Five Storied Building	-355	-684	Not safe	-355	+268	Not safe
Six Storied Building	-355	-751	Not safe	-355	+336	Not safe
Seven Storied Building	-355	-787	Not safe	-355	+372	Not safe
Eight Storied Building	-355	-840	Not safe	-355	+426	Not safe
Nine Storied Building	-355	-888	Not safe	-355	+475	Not safe
Ten Storied Building	-355	-933	Not safe	-355	+520	Not safe
Eleven Storied Building	-355	-975	Not safe	-355	+563	Not safe
Twelve Storied Building	-355	-1014	Not safe	-355	+603	Not safe
Thirteen Storied Building	-355	-1051	Not safe	-355	+641	Not safe
Fourteen Storied Building	-355	-1086	Not safe	-355	+677	Not safe

Table 3 Unbalanced moments due to only gravity loads and combined gravity and lateral loads at side ‘A’ and side ‘B’ of bottom storey level for zone 2

Storey	Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘A’			Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘B’		
	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks
Two Storied Building	-355	-268	Safe	-355	-265	Safe
Three Storied Building	-355	-269	Safe	-355	-264	Safe
Four Storied Building	-355	-271	Safe	-355	-262	Safe
Five Storied Building	-355	-273	Safe	-355	-260	Safe
Six Storied Building	-355	-274	Safe	-355	-259	Safe
Seven Storied Building	-355	-275	Safe	-355	-258	Safe
Eight Storied Building	-355	-277	Safe	-355	-257	Safe
Nine Storied Building	-355	-278	Safe	-355	-255	Safe
Ten Storied Building	-355	-279	Safe	-355	-254	Safe
Eleven Storied Building	-355	-281	Safe	-355	-252	Safe
Twelve Storied Building	-355	-282	Safe	-355	-251	Safe
Thirteen Storied Building	-355	-283	Safe	-355	-250	Safe
Fourteen Storied Building	-355	-285	Safe	-355	-248	Safe

Table 4 Unbalanced moments due to only gravity loads and combined gravity and lateral loads at side ‘A’ and side ‘B’ of bottom storey level for zone 3

Storey	Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘A’			Unbalanced Moment (kip-ft) at Bottom Storey Level of Side ‘B’		
	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks	Only Gravity Loads	Combined Gravity and Lateral Loads	Remarks
Two Storied Building	-355	-269	Safe	-355	-265	Safe
Three Storied Building	-355	-271	Safe	-355	-262	Safe
Four Storied Building	-355	-274	Safe	-355	-260	Safe
Five Storied Building	-355	-277	Safe	-355	-256	Safe
Six Storied Building	-355	-279	Safe	-355	-254	Safe
Seven Storied Building	-355	-281	Safe	-355	-252	Safe

Eight Storied Building	-355	-283	Safe	-355	-250	Safe
Nine Storied Building	-355	-286	Safe	-355	-248	Safe
Ten Storied Building	-355	-288	Safe	-355	-245	Safe
Eleven Storied Building	-355	-290	Safe	-355	-243	Safe
Twelve Storied Building	-355	-292	Safe	-355	-241	Safe
Thirteen Storied Building	-355	-295	Safe	-355	-239	Safe
Fourteen Storied Building	-355	-297	Safe	-355	-236	Safe

CONCLUSION

Based on limited number of model and storey stated above, the following conclusions can be drawn. The study may provide some necessary information related to shear wall structure in flat plate high-rise building having large span length:

(1) From analysis it is found that for flat plate high-rise building having large span length, the slab-column joint is not sufficient to resist unbalanced moments under seismic load. However, combined shear wall and moment frame system have been shown better performance by minimizing the unbalanced moment at different story level to resist lateral forces.

(2) The flat plate high-rise building structure with shear wall has been designed only for gravity load based on direct design method may satisfy at seismic zone-2 and zone-3 according to BNBC (2006) under combined gravity and lateral loads.

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STUDY ON THE BEHAVIOUR OF CONCRETE STRENGTH USING RECYCLE TIRE AS A PARTIAL REPLACEMENT OF COARSE AGGREGATE

Shubhagata Roy[1*], M Shazzadul Islam[1], M Tarequl Alam[1] & M Saiful Islam[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering &Technology, Bangladesh.*
**Corresponding Author, <shubhagataroy20@gmail.com>*

ABSTRACT

Waste-Tire rubber is one of the significant environmental problems worldwide. With the increase in the automobile production, huge amounts of waste tire need to be disposed. This waste being non-biodegradable poses severe fire, environmental and health risks. Due to the increasingly serious environmental problems owing to disposal of waste tires, the feasibility of using such elastic and flexible waste particles as a partial replacement of coarse aggregate has become a research issue. The primary objective of this study is to investigate the change in strength of concrete due to the use of waste tire. Total 72 nos. of 4 inch cube were cast using waste tire chips as a partial replacement of coarse aggregate (5%, 10% and 15% by weight) and cured in plain water for 7, 28 and 90 days. Compressive and tensile strength of the specimens was measured after specific exposure period. The results of this study show that there was a significant reduction in the compressive strength and tensile strength of concrete containing waste tire rubber than normal concrete. It was also noted that the slump value was reduced as the percentage of rubber increased. But the most important fact is that unlike plain concrete, the failure state in rubberized concrete occurs gently and uniformly indicating the ductile behaviour. Finally it is recommended to use waste tires for making non-structural Portland cement concrete, such as mass concrete, partitions, back stone concrete, concrete blocks, and other non-structural uses.

Keywords: Recycle tire, Rubberized concrete, Compressive Strength, Tensile Strength.

INTRODUCTION

The disposal of waste tires is becoming a waste management problem in the world including Bangladesh particularly in Dhaka and Chittagong city. Management of waste-tire rubber is very difficult for municipalities to handle because the waste tire rubber is not easily biodegradable even after long-period of landfill treatment (Guneyisi et al. 2004). Stockpiling is dangerous, not only due to a potential negative environmental impact, but also because it presents a fire hazard and provides a breeding ground for rats, mice, vermin, and mosquitoes (Ghaly and Cahill, 2005). Also due to increase of vehicle ownership and traffic volume within the city, this eventually will increase consumption of tires over time. Current practices show that residents throw it randomly in different places such as valleys, road sides, open areas, and waste dumpsites in improper ways taking the means of open fire, and without consideration of risk on human health and environment. Uncontrolled combustion of tire tends to release significant amount of unburned hydrocarbons and noxious emissions into the atmosphere. The melting tires also produce large quantity of oil, which cause contamination of soil and ground water. Recycled waste tire rubber is a promising material in the construction industry due to its lightweight, elasticity, energy absorption, sound and heat insulating properties. Recycled waste tire rubber has been used in this study to replace coarse aggregate by weight using different percentages. The main objectives of this work are given below:

- To study the compressive strength & tensile strength of selected numbers of rubberized concrete cube.
- To observe the workability of the rubberized concrete for its application
- To observe the failure criteria of ordinary concrete and rubberized concrete.
- To minimize the risk of pollution.

- To recommend the use of such waste tire in concrete construction, thereby minimizing cost.

MATERIALS AND METHODS

CEMENT:
Cement is a cementing or binding material used in engineering construction. It is manufactured from calcareous substance (components of calcium and magnesium) and is similar, in many respects, to the strongly hydraulic limes but possessing greater hydraulic properties. Ordinary Portland cement (OPC) was used in the present study. The physical properties and chemical constituent of the used OPC is given in table 1 and 2.

Table 1: Physical properties of the OPC

SL NO.	Characteristics		Value Obtained Experimentally
1.	Fineness		92%
2.	Normal Consistency		26.8%
3.	Soundness		7mm
4.	Setting Time	Initial	140 min
		Final	182 min
5.	Compressive Strength(psi)	3 days	16.5
		7 days	21.6

Table 2: Chemical constituent of the OPC

Constituents	Oxide Composition	Composition (%)
Tri-calcium Silicate	Ca_3SiO_4	45-55
Di-calcium Silicate	Ca_2SiO_5	20-30
Tri-calcium Aluminate	$Ca_3Al_2O_6$	9-13
Tetra-calcium Aluminoferrite	$Ca_4Al_2Fe_2O_{10}$	8-20
Calcium Sulphate	$CaSO_4.2H_2O$	2-6
other		2-8

FINE AGGREGATE:
Sand and surki are commonly used as fine aggregate in Bangladesh. Stone screenings, burnt clays, cinders and fly-ash may also be used as a substitute for sand in making concrete. Locally available sand having physical properties of FM 1.60, Specific gravity 2.40, Absorption capacity 3.31%, Moisture content 2.18% was used.

COARSE AGGREGATE:
Brick Khoa (broken bricks), broken stones, gravels, pebbles, clinker cinders etc of the size of 3/16 to 2 inch are commonly used as coarse aggregate in Bangladesh. Here stone chips of 20mmф nominal size were used as coarse aggregate. Physical properties of coarse aggregate are: Dry rodded Unit Weight 1680 Kg/m^3, Bulk specific gravity(SSD) 2.84, Absorption capacity 1.28 %,Moisture content 0.90 %.

RECYCLE TIRE**:** Generally two categories of commercially available rubber products are natural rubber and synthetic rubber. Despite the competition of synthetic compounds, natural rubber continues to hold an important place in tire consumption. Physical properties of used recycle tire are Unit Weight 1150 Kg/m^3, Bulk specific gravity (SSD) 1.13, Absorption capacity 48.36%.

Fig 1: waste tire

METHODOLOGY

	7 days		28 days		90 days	
Sample Type (% replacement)	Compressive Strength (psi)	Average Compressive Strength (psi)	Compressive Strength (psi)	Average Compressive Strength (psi)	Compressive Strength (psi)	Average Compressive Strength (psi)

The following steps were followed for the completion of the study

1. As per experimental plan of the study all raw materials were collected. Waste tire was collected from a private car.(Ref. fig 1)
2. All the physical properties of the ingredient materials were tested before use
3. Mix design for M35 concrete was performed following ACI code
4. A total of 72 nos. 4 inch concrete cube specimen were cast for 0%, 5%, 10% and 15% replacement of coarse aggregate by recycle tire chips. Thus total 18 no. of cubes were used from each percentage replacement.
5. All the samples are cured in plain water for 7, 28 & 90 days. At the end of each curing period, 3 samples were tested for compressive strength and 3 samples for tensile strength to get the average strength at each test period.
6. All the tests data were analyzed critically and presented in tabular and graphical form. Comparing the test results with control specimens (0% replacement), the necessary conclusion were made

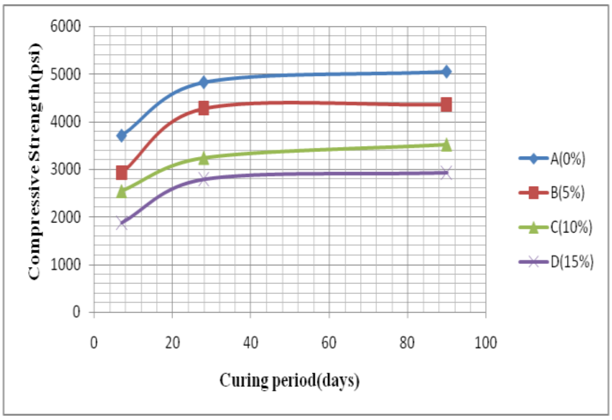
RESULT AND DISCUSSION

As per experimental program, the specimens were taken out from curing tanks periodically and tested for both compressive and tensile strength. The test results are presented in subsequent tables and figures are discussed with a view to arrive at some necessary conclusion.

Compressive strength test: Compressive strength of test specimen (from M35) made with waste tire chips as partial replacement of coarse aggregate by weight (0%, 5%, 10% and15%) are tested after different curing period of 7 days,28 days and 90 days. Table 3 show the compressive strength results of plain and rubberized concrete samples cured by 7, 28 & 90 days. Fig 2 shows the graphical presentation of compressive strength development of different concrete sample with age. Also fig 3 shows the relative compressive strength (%) as compared to normal concrete for different exposure periods. Compressive strength machine 1000 KN capacity was used to determine the compressive strength. From the relevant strength tables and graphs it is seen that the strength of rubberized concrete specimens are lower than the normal control concrete. As the tire chips content increases the strength decreases. The inclusion of tire chips content 5 to 15 % as partial replacement of coarse aggregate led to the strength reduction ranging from 14-42%.

Table 3: Compressive strength of different concrete specimens for 7, 28 & 90 days

A (0%)	3656	3712	4745	4828	5331	5052
	3488		4661		4912	
	3991		5080		4912	
B (5%)	2986	2930	4075	4270	4493	4354
	2902		4493		4326	
	2902		4242		4242	
C (10%)	2567	2539	3153	3237	3488	3516
	2651		3237		3488	
	2399		3321		3572	
D (15%)	1981	1869	2734	2790	2818	2930
	1897		2818		2902	
	1729		2818		3069	



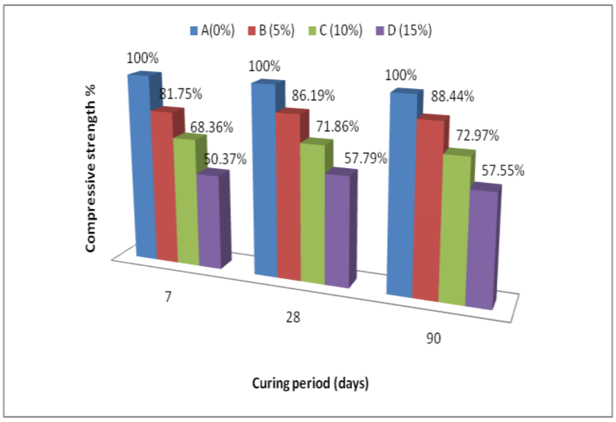


Fig 2: Compressive strength of different concrete specimens

Fig 3: Relative compressive strength (%) compared to normal concrete for different curing periods

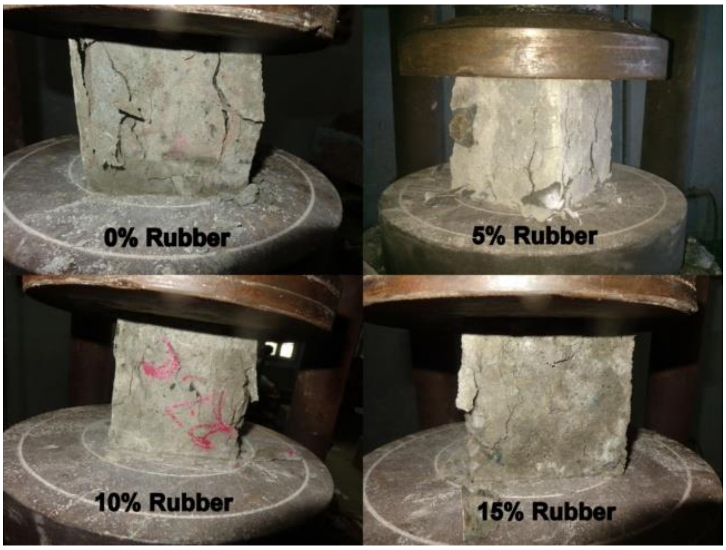


Fig 4 : Failure pattern for compressive test

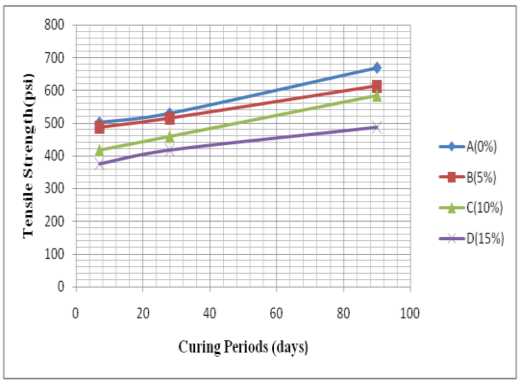
From above comparison of results, it is clear that excess use of recycle tire chips reduce the compressive strength of rubberized concrete significantly.

However concrete utilizing waste tire rubber demonstrated a ductile, plastic failure rather than brittle failure. Crack width in rubberized concrete is observed to be smaller than that of plain concrete, and the propagation of cracks and failure symptoms are more gradual and uniform (Ref. Fig 4). It was also noticed that concrete broke around the rubber particles during failure.

Tensile strength test: For the tension test, the specimens were put under compressive strength machine and a device with two rods in top & bottom was used (Ref. fig 7).The surface of the specimen was kept plain by placing a smooth plate on both top and bottom. The load was applied until the specimen failed and the corresponding reading was taken from dial gauge. Table 4 and fig 5 shows the tensile strength values for both normal and rubberized concrete specimens at different age. On the other hand fig 6 gives the relative tensile strength (%) as compared to normal concrete for different exposure periods.

Table 4: Tensile strength of different concrete specimens for 7, 28 & 90 days

	7 days		28 days		90 days	
Sample Type (% replacement)	Tensile strength (psi)	Average tensile Strength (psi)	Tensile strength (psi)	Average tensile Strength (psi)	Tensile strength (psi)	Average tensile Strength (psi)
A (0%)	557	501	557	529	724	668
	515		389		557	
	431		641		724	
B (5%)	515	487	473	515	641.0	613
	473		557		557	
	473		515		641	
C (10%)	431	417	431	459	641	585
	389		473		557	
	431		473		557	
D (15%)	389	375	389	417	473	487
	389		431		515	
	347		431		473	



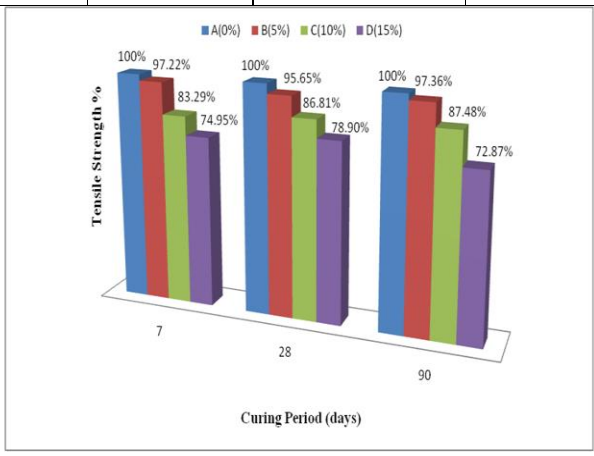


Fig 5: Tensile strength of different concrete compared specimens

Fig 6: Relative tensile strength (%) to normal concrete for different curing periods

From the relevant table and graphs, it is seen that plain concrete strength is more than rubberized concrete for all curing periods, but strength decreasing rate is lower in tensile test as compared to compressive strength test. Similar trend for strength decrease with tire chips content is also observed in case if tensile strength development. However with the use of tire chips content of 5% to15%, the decrease in tensile strength are observed to vary from 5 to 29%. The relatively lower decrease in tensile strength may be due to the random presence of tire chips particles within the concrete matrix which might act as a crack arrested. The tire particles used were not uniformly graded due to its improper shape. Making of tire chips by machine could provide uniformly graded particle size and hence better concrete strength.

Again rubber particles may not adhere with the cement paste perfectly which can be minimized by pre treatment of rubber. Moreover, during temping of concrete in mold some rubber particle came out on the surface due to its elastic property.

Fig 7: Failure pattern for tensile test

Unlike plain concrete, the failure state in rubberized concrete did not occur quickly and it was a gradual manner without separation in to two pieces which indicate the ductile behavior of tire chip concrete.

Workability: Replacing coarse aggregates by 5%, 10% and 15% of waste tire is resulted in a decreased concrete slump value. It is noticed that increasing waste tire content decreases the concrete slump and workability. Variations of slump of fresh concrete with and without waste tire aggregate are presented in fig 8. From the fig, it is clearly seen that as the tire chips content increases in the concrete mix, the slump values decreases. However up to the replacement of 10% tire content showed the slump value that can be accepted from practical consideration.

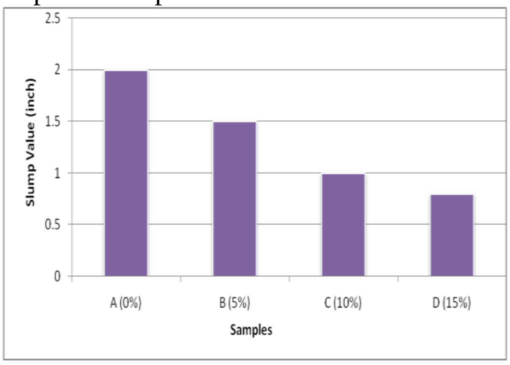


Fig 8: Slump value of fresh concrete and rubberized concrete

CONCLUSION

Based on the limited scope of study on the concrete incorporating tire chips as partial replacement of coarse aggregate, the following conclusion can be drawn

- Concrete with recycle tire chips as a partial replacement of coarse aggregate shows a significant reduction in the concrete compressive strength (14 to 42%) as compared to the plain concrete.
- Tensile test indicated a systematic reduction in strength(5 to 29%) with the increase of recycle rubber content
- Unlike plain concrete, the failure state in rubberized concrete did not occur quickly and did not cause any detachment in the specimen's elements. More ductile behaviour is observed for rubberized concrete compared to plain concrete specimens under compression and tensile testing.
- Crack width in rubberized concrete is observed to be smaller than that of plain concrete, and the propagation of cracks failure symptoms is more gradual and uniform.
- The results revealed that slump values decrease i.e workability decrease as waste tire rubber content increase from 0% to 1o%.However the mixture had acceptable workability as compared to normal concrete mixture.
- Use of recycled rubber tires as aggregate could be successful in lightweight concrete and in non-structural applications. It represents a viable alternative to recycle tires helping the conservation of the environment.

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DURABILITY OF STRUCTURAL CONCRETE IN ACIDIC ENVIRONMENT-A REVIEW

Sabrina Islam[1]*, M Rabiul Alam[1] and G M Sadiqul Islam[1]

[1] *Department of Civil Engineering, CUET, Chittagong-4349, Bangladesh.*
**Corresponding Author, <sabrina_islam107@yahoo.com>*

ABSTRACT

The versatile use of reinforced concrete in construction industry often face hazardous environmental action including acidic attack and is reported to be deteriorated leading to costly repair / rehabilitation options. Proper techniques / measures must be ensured during the selection of materials, design and construction of RC structure in such environment so as to conform the durability aspects. The paper addresses and reviews the different aspect of acidic environment and its disastrous effect on structural concrete including the factors influencing the attack, causes and possible remedial measures based on previous study / existing literature. The mechanism of reaction between the alkaline hardened cement matrix and the acidic ions are discussed together with the improvement of the microstructure of concrete and the provision of barriers against acidic attack. The state of art reports indicate that among all the acidic solution coming in contact with structural concrete, sulfuric acid is considered more harmful for its aggressive reaction with the hydrated cement matrix. The formation of expansive and leachable products leads to the creation of micro cracks and porus matrix causing degradation of concrete. The partial replacement of Cement with supplementary cementations materials including fly ash, slag, silica fume etc. and also use of geopolymer concrete are reported to increase the durability of structural concrete in acidic environment.

Keywords: Acidic attack, Acidic environment, Structural concrete, Geopolymer concrete, Durability, Permeability.

INTRODUCTION

Concrete has been proved as a well resistant construction material all over the word since past century owing to its versatile properties including strength, durability in normal as well as aggressive environments. The characteristics of concrete depend upon the properties of its ingredients, mix proportion, mixing etc. and other controls during placing, compaction and curing. The structural concretes are often subjected to different aggressive environments such as acidic environments, marine environments etc. and reported to gradual deterioration with time. Most of the concrete structures for practical application particularly in industrial region and sewer pipe lines are more or less affected by acid attack. In addition, the structures those are not directly exposed to acidic environments are also affected by different acids from acid rain. Due to climate change, acid rains are frequent phenomena now a days (Shetty, 2003).

In general, Ordinary Portland Cement (OPC) Concrete has a low resistance to chemical attack (Khitab *et al*, 2013). The common forms of chemical attack on concrete and the embedded rebar in reinforced concrete are chloride attack, sulfate attack, carbonation due to CO_2, alkali-aggregate reaction and acid attack. However it can be well resistant to chemical attack provided an appropriate mix is used and the concrete is properly compacted. Concrete containing Portland cement, being highly alkaline, is not resistant to attack by strong acids or compounds which may convert to acids (Shetty, 2003). Consequently, unless protected, structural concrete should not be used when this form of attack may occur.

Concrete is also attacked by water containing free CO_2 such as moorland water or mineral waters, which may also contain hydrogen sulphide (Shetty, 2003). Not all CO_2 is aggressive because some of it is required to form calcium bicarbonate in the solution. Flowing pure water, formed by melting ice or by condensation and containing little CO_2 also dissolves $Ca(OH)_2$, thus causing surface erosion. Peaty water with CO_2 particularly aggressive as it can have pH value as low as 4.4. This type of attack may be of importance in conduits in mountain regions, domestic/industrial sewer lines not only from the standpoint of durability but also because of the leaching out of hydrated cement that leaves behind protruding and increase the roughness of the pipe.

Concrete is generally resistant to microbiological attack because its high pH does not encourage such action; nevertheless, under certain (fortunately rare) tropical conditions, some algae, fungi and bacteria can use atmospheric nitrogen to form nitric acid which attacks concrete (Shetty, 2003).

Use of blended cements including Pozzolanas such as ground granulated blast furnace slag (GGBS), fly ash and especially silica fume, is beneficial in reducing the ingress of aggressive substances. Pozzolanic action also fixes $Ca(OH)_2$, which is usually the most vulnerable product of hydration of cement so far as acid attack is concerned. However, the performance of concrete depends more on its quality than of the type of cement used.

$Ca(OH)_2$ can also be fixed by treatment with diluted water-glass (sodium silicate). Calcium silicates are then formed, filling the pores. Treatment with magnesium fluorosilicate is also possible. The pores become filled and the resistance of the concrete to acid is also slightly increased, probably due to the formation of colloidal silicofluoric gel.

ACIDIC ENVIRONMENT:

Durability of concrete structure in acidic environment is one of the most severe problems encountered in practice. Concrete is often exposed to water which may contain domestic and industrial effluents. Various industries discharges strong acids such as sulphuric, nitric, hydrochloric acids are often cause to rapid attack on exposed concrete. The organic acids from the food processing industries namely acetic acid, lactic acid and formic acid are also in the severe category. Industries using phosphoric acid (fertilizer industries), tannic acid (leather tanneries) are subjecting concrete to moderate attacks. Carbonic acid from beverage industries solution of ammonium nitrate and ammonium sulphate, etc. encountered in fertilizing industries are also moderately corrosive to structural concrete. Table 1 shows a list of some substances which causes severe chemical attack on concrete.

Table 1: List of some substance (including acids) causing severe chemical attack of concrete

Type of Substance	Acid (Inorganic)	Acid (Organic)	Others Chemicals
Name of the substance	(a) Carbonic (b) Hydrochloric (c) Nitric (d) Phosphoric	(a) Alkali (b) Citric (c) Formic (d) Humic (e) Lactic (f) Tannic	(a)Aluminum chloride (b) Ammonium Salts (c) Hydrogen Sulfide (d) Vegetable and animal fats (e) Sulfates

The acidic wastes results in lowing the pH of waste water that react chemically with mortar and concrete materials. In most cases, the chemical reaction results in the formation of water soluble calcium compounds which are either leached away by the aqueous solution or form an insoluble salt with the aggressive acid.

Acid rain, which consists mainly of sulfuric acid and nitric acid and has a pH value between 4.0 and 4.5 may, causes surface weathering of exposed concrete. Although domestic sewage, being alkaline, does not attack concrete, severe damage of sewers has been observed in many cases especially at fairly high temperature due to reduction of sulfur compounds to H_2S by anaerobic bacteria. H_2S is dissolved in moisture films on the exposed concrete surface and undergoes oxidation by aerobic bacteria finally producing H_2SO_4. This acid is particularly aggressive because in addition to sulfate attack of the aluminates phase acid attack on $Ca(OH)_2$ and C-S-H takes place. Such attack occurs above the level of flow of the sewage and the hardened cement paste is gradually dissolved leading to progressive deterioration of concrete. Similar form of attack also occurs in offshore oil storage tanks.

ACID ATTACK ON CONCRETE

Chemical attack of concrete occurs due to the decomposition of the products of hydration and formation of new compounds which, if soluble, may leach out and, if not soluble, may be disruptive in situ. The most vulnerable hydrate is $Ca(OH)_2$, although C-S-H can also be attacked. Concrete can be attacked by liquids with a pH value below 6.5 but the attack is severe only at a pH below 5.5; below 4.5, the attack is very severe [Romben, 1978].

Mechanism of acid attack:

Concrete being very alkaline in nature, is extremely susceptible to acid attack. The mechanism for this process is very simple. The products of cement hydration are shown below.

$$\mathbf{CS + H \Rightarrow C\text{-}S\text{-}H + CH}$$

Calcium Silicate + Water → Calcium Silicate Hydrate + Calcium Hydroxide

Acid attack is caused by the reaction of an acid and the calcium hydroxide portion of the cement paste which produces a highly soluble calcium salt by product. These soluble calcium salts are easily leached out from the cement paste thus weakening the paste's structure as a whole. This basic reaction is shown below (Kenkel, 2011).

$$\mathbf{Acid\ X + CH \Rightarrow CX + H}$$

Acid + Calcium Hydroxide→ Calcium Salt + Water

A more aggressive and destructive case of acid attack occurs when concrete is exposed to sulfuric acid. The calcium salt produced by the reaction of the sulfuric acid and calcium hydroxide is calcium sulfate which in turn causes an increased degradation due to sulfate attack. This process is illustrated below.

$$\mathbf{H_2SO_4 + Ca(OH)_2 \Rightarrow Ca(SO_4) + 2H_2O}$$

Acid + Calcium Hydroxide→ Calcium Sulfate+ Water

Calcium sulfate product contributes to sulfate attack. The dissolution of calcium hydroxide caused by acid attack occurs in two phases. The first phase is the acid reaction with calcium hydroxide in the cement paste. The second phase is the acid reaction with the calcium silicate hydrate. The dissolution of the calcium silicate hydrate, is the most advanced cases of acid attack, can cause severe structural damage to concrete.

Corrosion of rebar is an electrochemical reaction that occurs at anodic spot due to the development of anodic and cathodic regions on its surface as shown in Fig.1. The transformation of metallic iron to rust is accompanied by an increase in volume as large as 600% of the original metal consumed that result in cracking, spalling and ultimately structural failure. But concrete normally provides a high degree of protection to rebar against corrosion due to highly alkaline environment (pH≈ 13). The passivity of steel is maintained as long as definite amount of aggressive ions do not reach the steel surface. In acidic environment, the rebar corrosion may be mainly initiated by the carbonation and chloride penetration process. In carbonation process the high pH value of concrete around rebar may be reduced by the ingress of acid (CO_2 in air) from the surrounding environment as per following reaction (Ref Fig.2).

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O$$

As a result, pH changes from (12-13) to neutrality and the passivity of steel is lost. The penetration depth i.e. carbonation depth when exceeds the cover depth, aggressive ions including chloride find a suitable environment leading to greater corrosion. In chloride penetration process, chloride ions from acidic environment (HCl solution) gradually diffuse into concrete with time leading to a condition when the concrete no longer able to protect rebar from corrosion. Various researchers proposed different values of threshold chloride concentration that varies from 75-3640 ppm [Kong and Orbison, 1987].

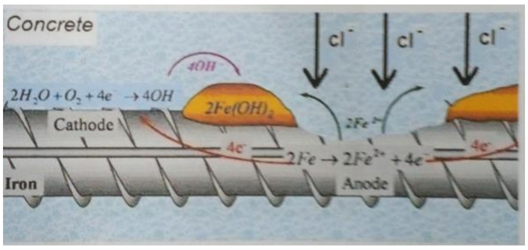

Fig 1: Schematic description of corrosion process of rebar in concrete PCA, 2002)

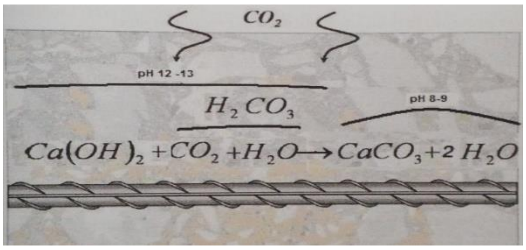

Fig 2: Concrete carbonation process (PCA, 2002)

Factors controlling acid attack:

The rate of acid attack and hence corrosion of concrete or cement paste depends on many factors such as the aggressivity of medium, the resistance of cement based materials, the choice of protective measurement, solubility of the calcium salts formed by the hydration of cement based materials etc. [Van Aardt, 1978]. The aggressivity of medium is influenced by the dissociation characteristics of acid and pH and quantity of acid. But the concentrations of aggressive species are more sensible to acid attack than pH value of the acidic solution [Thornton, 1978].

Other important factors for acid attack are quantity and action mode of acting solution. In principle, two action modes are possible: (1) static conditions indicating no practical movement of the acting solution; (2) dynamic conditions means the change of the level or flow of the acting solution. The flow rate may have a greater influence than the concentration of the solution [Khoury, 1994]. Dynamic conditions contribute to the transport of the aggressive species into the pore system of concrete and to the drainage out of the decomposition products [Grube & Rechenderg, 1989]. The corroded products may be expanded due to crystallization resulting tension cracks in the concrete. This crack helps the aggressive species to ingress in the unaffected concrete which again attacks the unaffected concrete. Alternate wetting and drying caused by repeated changes of the level in the acting solution may contribute to the severity of the processes [Thornton, 1978].

Ambient temperature and relative humidity may influence the acidic attack, especially in the presence of the changes of the level of the acting solution [Fattuhi and Hughes, 1988]. The level of the ambient relative humidity is of special significance in an attack by air pollutants like CO_2, SO_x and NO_x because the quantity of pore liquid in the attacked material is dependent on the ambient relative humidity

The type of cement has no significant effect on acid resistance. But blended Portland cement containing pozzolana such as slag, Fly ash, silica fume shows better performance against acid attack although the efficiency of performance is characterized by some other factors such as type and quantity of cement, amount, type and properties of admixture and curing conditions [Zivica and Bajza, 2002].

Effect of acid attack on structural concrete:

Acids affect the structural concretes by two ways. Firstly the acid produces calcium salts such as gypsum by the reaction with Portlandite [$Ca(OH)_2$] which in turns produces ettringite ($C_3A{\cdot}3CaSO_4{\cdot}31H_2O$) by reacting gypsum with C_3A, hydrated aluminates, or monosulfate ($C_3A{\cdot}CaSO_4{\cdot}12H_2O$).
The volume of this ettringite being higher than the reactants they replace, cracks may be developed in the structural concrete leading to greater acid attack by more ingress of acid in the concrete. As a result, the concrete losses compressive load bearing capacity due to its weak structural condition. Secondly, Thaumasite ($CaSiO_3{\cdot}CaCO_3{\cdot}CaSO_4{\cdot}15H_2O$) produced due to reaction of the calcium-silicate-hydrates (C-S-H) with this calcium salts leads to the reduction of C-S-H and ultimately

reduces the load bearing capacity of concrete. Besides these, acids including H_2SO_4 produce gypsum by reacting with C-S-H leading to the formation of more ettringite and hence lesser C-S-H content result in degradation of concrete. More aggressive acids such as hydrochloric, acetic and nitric acid produce calcium salts that are very soluble. This solubility leads to leaching out of the salts from the concrete and hence contributes to the greater porosity and permeability of concrete. Deterioration/degradation of concrete due to acid attack as reported by several researchers are shown in Fig 3.

Fig 3: Deterioration of concretes in sewage treatment plant [Moradian *et al*., 2012]

MEASURES FOR ACID ATTACK PREVENTION

Several methods/measures in the form of special materials and techniques are reported to be adopted to protect the concrete against acid attack or to retard the acid attack on structural concrete. Some of them are briefly explained below.

Use of Special concrete:

From the above discussion, it is clear that the severity of acid attack mainly depends on the amount of lime present in cement. So to reduce the acid attack, quantity of cement may be reduced by the partial replacement of cement with supplementary cementitious materials which imparts strength to concrete. Mineral admixture such as slag, fly ash, silica fume, metakaoline etc which are waste can be used as the supplementary materials with cement for making acid resistant concrete.
Silica fume (SF) is an industrial by-product obtained from an electric arc furnace process. Silica fume and fly ash have proved to be very useful additives for concrete. Using fly ash in combination with micro silica fume, results in an improved resistance against acid attack [Power *et al*, 1955; Alegre, 1978]. Fly ash produces a densely packed mixture of cement paste and aggregates, while silica fume reacts pozzolanically and transforms the calcium hydroxide into Calcium-Silicate-Hydrate (CSH) gel in accordance with the following equation (Langan *et al*, 2002):

$$SiO_2 \text{ (solid)} + Ca^{2++}\ 2OH^- \longrightarrow CaO.SiO_2.H_2O \text{ (CSH gel)}$$

The CSH gel is the source of strength in concrete. It increases the bond between the cement paste and aggregates and thereby the compressive strength and chemical resistance of the concrete. The additional CSH produced by silica fume is more resistant to attack from aggressive chemicals than the weaker CH. Therefore, the combination of silica fume and fly ash results in a denser concrete having lesser quantity of calcium hydroxide, which considerably increases the acid resistance. Silica fume without fly ash produces micro cracks in concrete, which increase the path of acid attack (Langan *et al*, 2002). Blast Furnace Slag is formed when iron pellets, coke and limestone/dolomite flux are melted together in a blast furnace. Blast furnace slag significantly increases the durability of the concrete. It has been found to be very effective against chloride and sulphate attack; however it does not provide any considerable improvement in acid resistance [Hewayde *et al*, 2007].

The Geopolymer concrete (GPC) is reported to be more acid resistant in terms of dimensional stability i.e. loss in weight and strength as compared to identical portland pozolanic cement concrete (PPCC).

The geopolymers are found to produce high grade structural concrete by self-curing mechanisms. Geopolymers are a novel binder that relies on alumina-silicate rather than calcium silicate hydrate bonds for structural integrity and have been reported as acid resistant. GPC utilize industrial wastes in the form of fly ash (FA) and GGBS which are activated by alkaline medium to produce ambient temperature cured inorganic polymeric binder (called as geopolymers) in the form of aluminosilicate (Rajamane *et al*, 2012).

Air entraining concrete:

Although, air entrainment is mainly practiced to increase freeze-thaw resistance of the concrete, it can also be used as a means of acid resistance. The air entrainment increases the acid resistance because the air voids block micro capillaries and prevent the acid from invading the concrete through these canals. Air-entraining agents are available as additives as well as admixtures. As additives, the agents are interground with cement in fixed proportions. As admixtures, Vinol resin and fatty acid salts are used as air-entraining agents, which have now been largely replaced by synthetic admixtures: Alkyl sulphates, olefin sulphonates, Diethanolamines etc. [Moradian *et al*, 2012].

High performance concrete:

High performance concrete (HPC) is defined as a concrete having properties much superior than an ordinary concrete. Along with other properties such as strength and durability, it is supposed to have higher resistance against chemical attack. High-performance concretes are made with carefully selected high-quality ingredients and optimized mix design. Typically, such concretes will have a low water-cement ratio of 0.25 to 0.4. Various admixtures are usually used to make these concretes fluid and workable. Owing to lesser porosity, HPCs also offer significant acid resistance. The low porosity is achieved by cement contents in excess of 500 kg/m^3 in concrete, low water to cement ratio, adequate compaction and cuing and the incorporation of a super plasticizing admixture.The aggregate to be used in the mix design should be well graded, chemical ion free and having low alkalinity as acid attack is accelerated by the alkalinity of concrete. The mixing water must be free from objectionable salt ions.

As penetration of aggressive spices is regarded as an important cause of concrete deterioration, to prevent the ingress of acid into the concrete, a layer with sufficient thickness of coating may be provided. Antibacterial additives can be incorporated in the coating mix to reduce the formation of acids in some structures like sewers lines [Haile *et al.* 2010]. Adequate concrete cover must be provided in case of reinforced concrete structures to achieve greater level of corrosion protection.

Although durability of concrete depends on several factors including aggregate type, cement type, water-binder ration, air entrainment, period of procuring, quality of mixing water, proper quality control at every stages of concrete making is mandatory in order to get denser, less permeable, good quality concrete for the use as structural concrete in the acidic environment.

Use of nanotechnology for acid attack prevention:

Nanotechnology is a new branch of material sciences that promotes the use of nano particles in different domains. As obvious from its name, nano particles have their sizes in nano meters. Materials at nano scale display properties somewhat different from those at micro or macro scale. For example, inert titania is used for pigmentary purposes while a nano titania is a photo-catalyst. Similarly, opaque copper becomes transparent when ground to nano size [Parrot, 1991]. Knowing that, attempts were made to apply nanotechnology in different fields including construction engineering. Apart from other beneficial effects, nanotechnology has also found its importance in increasing the acid resistance of concrete as given below.

(a) Nano cementitious materials:

Cementitious materials including concrete, mortar and cement paste were developed due to strong bonding characteristics and very rapid hardening and setting of cement. Several new cement composites have been developed during the recent years by combining it with nano-Titania (TiO_2), carbon nanotubes, nano-silica (SiO_2), nano clay and nano-alumina (Al_2O_3), which have significantly improved the performance of these materials. Besides, efforts are also going on to introduce nano-cement that might lead to superior strength due to its very fine particle size and faster chemical reactions with water [Raki *et al*., 2010]. Or in other words nano materials might enhance resistance of concrete against acid attack.

(b) Nano coating materials:

With the help of nanotechnology, coatings with a molecular structure which simply rejects adhesion by foreign bodies have been created. Many nano-coatings are available in the market, which provide effective resistance against acid attack. Appropriate nanotechnology can be used to create eco-friendly sealers and coatings that deliver pure Performance on concrete. These coatings can be applied by spraying, rolling or brushing (Khitab *et al*, 2013).

(c) Nano food additives:

New concretes with nano food additives are being developed at National Institute of Standards and Technology (NIST), USA. The technique is named as VERDiCT (Viscosity Enhancers Reducing Diffusion in Concrete Technology). With nano additives in concrete pores, the viscosity of the concrete pore solution can be increased, which will slow down the ingress of external species (including acids) (Khitab *et al*, 2013).

CONCLUDING REMARKS:

In this review paper, attempts have been made to present the state of art knowledge on structural concrete exposed to acidic environment. A lot of studies have been conducted on the ill effects of acidic environment on structural concrete used for various applications. Existing literatures covers the effectiveness of supplementary cementitious materials as partial replacement of cement to produce durable concrete as remedial measures. However, on the basis of the previous research/existing literature discussed, the following findings/recommendations can be noted.

(a) Concrete being very alkaline in nature (pH: 12-13) is extremely susceptible to acid attack.
(b) Aggressive and destructive case of acid attack occurs when structural concrete is exposed to sulfuric acid. HCl leads to rebar corrosion and is initiated by carbonation and chloride penetration from acidic environment.
(c) The rate of acid attack and hence corrosion of concrete depends on several factors including the aggressivity of the of the medium, the resistance of the cement based materials, solubility of calcium salts formed during hydration, dissociation characteristics of acid i.e. pH value, quantity of acid etc.
(d) Alternate wetting and drying caused by repeated changes of the level in the acting solution of the environment may contribute to severity of the corrosion/deterioration process.
(e) Portland cement containing pozzolana including fly ash, slag shows better performance against acid attack.
(f) Acid reacts with hardened cement matrix and form complex products including Friedel's salts, ettringite etc. which are being leachable/expansive in nature causes degradation of structural concrete.
(g) The geopolymers are reported to produce high grade structural concrete by self-curing mechanism and is found to be more acid resistant in term of dimensional stability and durability.
(h) Use of nanotechnology in developing cement composites, nano coatings, ecofriendly sealers etc. may lead to produce acid resistant concrete.
(i) Addition of mineral admixture (fly ash, slag, silica fume etc.) with cement, proper protective coating, adequate cover to rebar, good quality concrete having low permeability, use of chemical free ingredient materials for concrete, proper quality control etc. are the suggested remedial measures for making structural concrete in acidic environment.

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USE OF PAPER INDUSTRY WASTE IN MAKING LOW COST CONCRETE

S. Chakraborty[1*], T. Mahmud[1], M. M. Islam[1] & M. S. Islam[1]

[1] *Department of Civil Engineering, CUET, Chittagong-4349, Bangladesh.*
**Corresponding Author, <sudipto_ounshu@yahoo.com>*

ABSTRACT

Among different waste materials produced in different factories of Bangladesh, paper industry waste, hypo-sludge is a noticeable one as it poses problems of health hazards and disposal. Hypo-sludge is a growing problem in Bangladesh as landfill spaces are used up every year causing the decrease of cultivable lands. In some cases, crop lands are used for dumping sludge which reduces fertility and a threat to human health. Disposal of sludge into rivers and canals causes severe water pollution. Some paper mill companies try to get rid of it by using incinerators to burn it causing air pollution. It is reported that hypo sludge has pozzolanic as well as cementitious properties which may be used as replacement of cement clinker in concrete production. This paper deals with experimental investigations to evaluate the optimum percentage of hypo sludge to be used for making concrete. Three different grades of concrete M15, M20 and M25 were used in this study. Five different replacement level of cement with hypo sludge i.e. 10, 20, 30, 40 and 50% were used and OPC concrete of 0% cement replacement level was also made for comparison. Compressive and tensile strength of concrete were tested at a curing age of 7 and 28 days. Overall result reveals that use of hypo sludge as partial replacement of cement can improve the strength of lower grade concrete upto 20% replacement level. Use of hypo sludge as partial replacement of cement also markedly reduces the cost of construction which otherwise been dumped making environmental hazard.

Keywords: Hypo-sludge, Pozzolanic property, Compressive strength, Tensile strength.

INTRODUCTION

The paper industry in Bangladesh went into operation when Karnaphuli Paper Mill (KPM) was established by DAUD group at Chandraghona in Chittagong in 1953 [Saha et. al.,1997]. Three more paper mills were established by the government : Khulna Newsprint Mill(KNM),North Bengal Paper Mill(NBPM) and Sylhet Pulp And Paper Mill(SPPM).Among them, only KPM is now in operating condition and currently producing 30,000 tonnes of paper per annum [www.bcic.gov.bd/kmp.php]. Afterwards private sectors were also raised in for paper production. In the mid 90s, private companies started to take part in this **SECTOR** due to loss in state owned mills for shortage of raw materials and other problems. Currently it is seen that there are about 55 paper and board mills in our country which produce approximately 550,000 tonnes of paper and board per annum [Haroon,2010] These paper mills depend upon imported pulp and waste papers. Based on environmental impact effect, Bangladesh government has included paper and board mills in the "red category"[Environmental Conservation Rule 1997, Schedule 1, Ministry of Environment and Forest, Government of People's Republic of Bangladesh, 1997].Across the world, sustainability is a major issue now-a-days. Unsustainable trends in waste generation of waste can be regarded as a symptom of environmentally inefficient use of resources [OECD 2011].Transforming such otherwise unusable products may mitigate but not solve present and environmental challenges [Naik and Kraus 2000].About 6 kg of sludge is produced per ton of paper produced [Gabriele et al.,2011]. From this, we can sum up the total production of hypo-sludge (550,000*6)=3,300,000 kg per year in our country. Different studies are going on to use this hypo-sludge as supplementary cementious materials. Experiments show that when paper mill sludge ash was used to replace up 10% of the Portland cement; positive results were obtained on the mechanical performance of mortars [Gabriele et al.,2011]. Again it is assumed that to produce 4 million tons of cement,1 million ton green house gases are emitted. So it is encouraged to find an alternative for OPC although could be a small extent. In the context of low availability of renewable energy resources coupled with the requirements of large quantities of energy for Building materials like cement, the importance of using industrial waste cannot be under estimated. This study included M15, M20, M25 grade concrete with five partial replacement of hypo-sludge to investigate compressive strength and tensile strength. Curing period taken was 7 and 28 days, specimen were 100 mm × 100 mm × 100 mm cubes and were made from concrete with five partial replacement of level of cement (0%,10%,20%,30%,40%,50%) hypo-sludge.

This main objectives of the study is to investigate the use of paper industry waste known as hypo sludge as partial replacement of cement in making concrete. The specific objectives are to provide guide line for making low-cost concrete, disposed of hypo-sludge in a useful manner, find the optimum level of sludge for the partial replacement of cement and minimize the cost of concrete production.

MATERIALS

a) Hypo-sludge : Hypo-sludge was collected from KARANAPHULLY PAPAER MILL(KPM).(Fig.1.).Table-1 shows the comparison between the chemical composition of cement and hypo-sludge as obtained from chemical test data. **(**Experimental investigation in developing low cost concrete from paper industry waste by R.Srinivason,K. Sathiya and M. Palanisamy,2010)

Table : 1

Serial no.	Constituents	Cement(In %)	Hypo-sludge(In %)
1.	Lime(CaO)	62	37.97
2.	Silica(SiO_2)	22	11.92
3.	Alumina (Al_2O_3)	5	0.671
4.	Magnesium(MgO)	1	1.899
5.	Calcium sulphate	4	0.565

b) Cement : Ordinary Portland Cement Type-I is the most common type of cement in use around the world. It is a basic ingredient of concrete, mortar, stucco and most non-specialty grout cement. It is a fine powdered material produced by grinding clinker (95%) and a limited amount of Gypsum which controls the setting time. It conforms with the Bangladesh Standard BDS EN 197-1:2003 CEM-I 42.5 N, European Standard EN 197 type CEM I, and American Standard ASTM C 150 Type-I mark [Ref.(5)] Ordinary portland cement, from Premier Cement company was used as binding material in concrete test specimens.

Fig.1.Hypo-sludge

Fig. 2. Cement [Ref.(5)]

c) Coarse aggregate: The aggregate fractions ranging from 80 mm to 4.75 mm are termed as coarse aggregate. 20 mm downgraded crushed stone chips were used as coarse aggregate in making the concrete specimens.

Fig 3.Coarse aggregate

d) Fine aggeregate: Aggregate fractions ranging from 4.75 mm to 150 micron are termed as fine aggregate. Sylhet sand of FM 2.2 was used as fine aggregate in this study.

Fig. 4. Fine aggregate

d)Water : The quality of water was assured by using portal tap water. W/C ratios of 0.4,0.43 and 0.45 were used for M25,M20 and M15 grade concrete respectively**.**

RESULTS AND DISCUSSIONS

Compressive and tensile strength test : Standard metallic cube moulds(100*100*100 mm) were used to cast for compressive and tensile strength test specimen. The specimens were demoulded after 24 hours and subsequently immersed in water for 7 and 28 days curing age.For each age, 4 specimens were tested to determine averege compressive and tensile strength. Compression testing machine of 250 MT capacity was used for the strength test (Ref. Fig.5 and Fig.6).

The strength test results are presented both in tabular as well as graphical forms. Table 2 shows the 7 days and 28 days compressive strength for concrete specimens made from three different grades of concrete while Table 3 contains the corresponding tensile strength test results.

Fig. 5. Compressive strength testing

Fig. 6.Tensile strength testing

Table.2: Compressive strength (N/mm^2)

Partial replacement	7 DAYS			28 DAYS		
	M15	M20	M25	M15	M20	M25
0%	9.04	25.79	35.03	13.66	33.30	42.54
10%	20.01	24.06	28.68	25.21	29.25	30.99
20%	13.08	13.66	19.43	16.55	18.28	25.79
30%	7.88	5.57	10.77	10.77	7.309	14.81
40%	7.30	7.88	11.93	8.46	9.04	15.39
50%	3.26	4.99	7.30	3.84	6.15	7.88

Table.3: Tensile strength (N/mm^2)

Partial replacement	7 DAYS			28 DAYS		
	M15	M20	M25	M15	M20	M25
0%	1.53	3.84	4.42	2.11	5.57	4.99
10%	2.11	3.26	3.26	2.68	3.84	4.42
20%	1.53	2.11	2.68	2.11	2.68	3.26
30%	0.95	0.95	2.11	2.11	1.53	2.11
40%	0.95	1.53	2.11	1.53	1.11	2.68
50%	0.21	0.95	1.53	0.37	0.95	2.11

Fig.7 and Fig.8 shows the graphical presentation of compressive and tensile strength results reproducing for various replacement level of cement by hypo sludge waste materials.

From the test result it is seen that for cement replacement level upto 20%, the compressive strength of specimens made from relatively lower grade concrete i.e. M15 increases as compared to OPC concrete while the corresponding strength values decrease for high grade concrete specimens (Ref. Fig.7). However the 10% replacement level shows the maximum strength development and hence can be considered as the optimum level amongst the various replacement levels used in the study. The tensile strength results also show the similar trends regarding strength gaining with age against the cement replacement level. 10% cement replacement level can also be considered as the optimum level as per strength data.

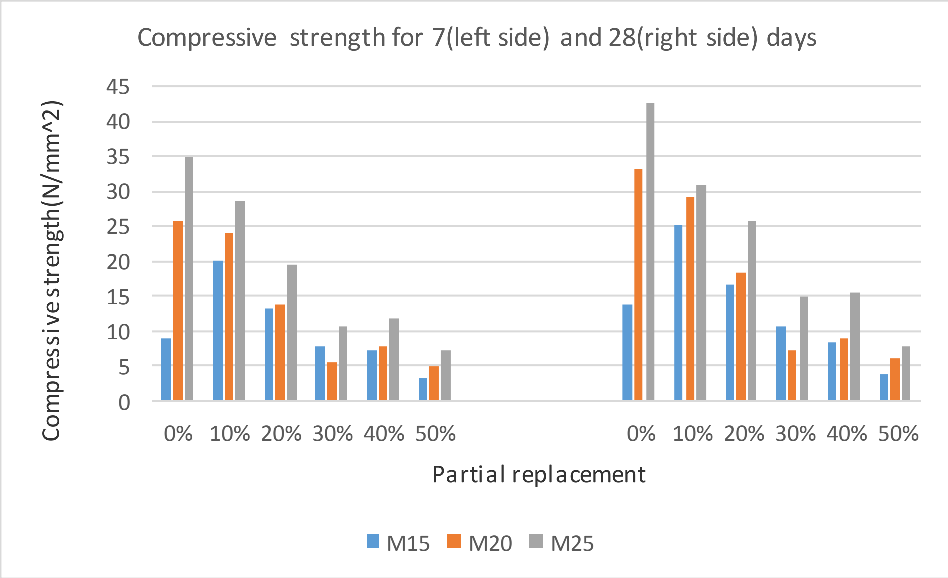

Fig.7 :Compressive strength for 7,28 days(M15,M20,M25)

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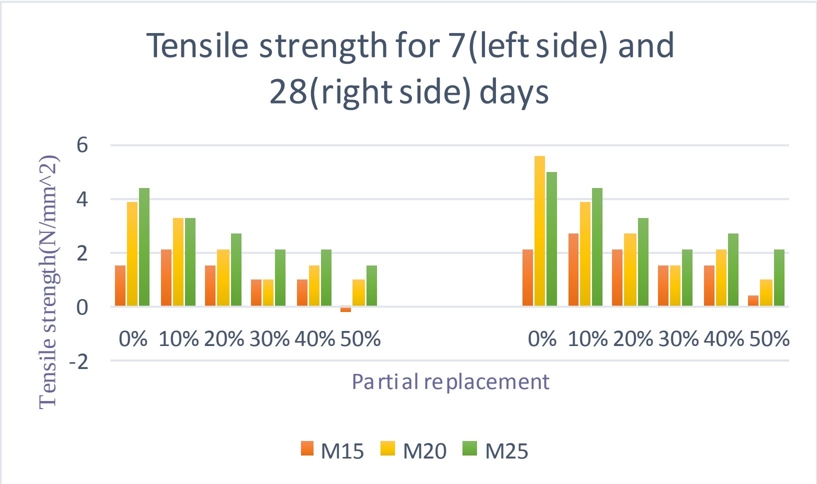

Fig.8 : Tensile strength for 7,28 days(M15,M20,M25)

From the above experimental studies, it is observed that for low strength concrete(M15), hypo-sludge replacement is successful but is found unsatisfactory for relatively higher strength concrete. Considering the cost of every cement bag as 500 taka which is now a standard market price, then for every bag of cement saving upto 50 taka can be made. Furthermore around 66000 bags of cement per year can be produced by utilsing hypo-sludge. However, further studies are requied to utilise hypo-sludge as a cement replacement material in different concrete construction working and to enhance the replacement level and also save our environment from pollution.

CONCLUSION

On the basis of the investigation carried out on the use of industrial waste (hyper-sludge) as partial replacement of cement in making three different grades of concrete the following conclusions can be drawn:

(a) The use of hypo-sludge as partial replacement of cement is found successful in case of lower grade concrete specimens but found unsuitable for higher grade concrete.

(b) Both compressive and tensile strength of concrete is found to be increased upto 20% cement replacement level as compared to the OPC concrete.

(c) Of the various replacement levels, the maximum strength gain is observed at 10% replacement level and hence can be considered as optimum level.

(d) The use of hypo-sludge can be used in low grade concrete works which otherwise been dumped over lands making environmental hazards.

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TRANSFORMATION OF BLIGHTED SPACE TO VIBRANT SPACE : A RESILIENT APPROACH

Abu Yousuf Swapan[1*]
[1] *Department of Architecture, Premier University, Chittagong, Bangladesh.*
**Corresponding Author, <swapan4794@gmail.com >*

ABSTRACT

Cities are losing usable spaces as blighted spaces are emerging within the central business district. These blighted spaces also termed as ‘Lost spaces’ or ‘Neglected spaces’ that deteriorating our urban environment. This paper is an attempt to show a resilient strategy for better mixed living environment termed as ‘Third place’. A holistic urban regeneration proposal in this paper indicates a complex transformation process from ‘Lost space’ to ‘Third place’. This paper works to achieve a holistic strategy for resilient urban design process; to achieve a better urban environment. Identifying the deteriorating process of urban decay and lessons from the case study analysis enlighten the revitalization strategy.

Keywords: Mono-function, Public Space, Lost space, Third Place, Revitalization.

INTRODUCTION

Central business district is the centre of gravity for a city. High concentration of retail and commercial buildings, close proximity of amenities and services makes an inner area as the focus of attention all over the world. City of Chittagong is expanding to its periphery without limit and creating new problems rather than solving any. Chittagong City has a population of 3.3 million with an area of 100 sq mile (260 sq km), total Open space of 200 acres (0.80 sq km), available Open area average is 0.06 acres per 1000 people, required area average is 4 acres per 1000 people (CDA standard 1961). Chittagong district possesses no natural lakes. The Assam Bengal Railway dug two artificial lakes in 1920 and 1924 named ‘Deba’ at Agrabad and ‘Jordighi’ or ‘Horseshoe pond’ at Pahartali which are under threat due to illegal encroachment. Water body (Deba) is situated at the south of Agrabad downtown area which has great potentiality to become a successful open space for the area as well as for the city. Besides, Agrabad downtown and adjacent water body is facing the problem of ‘mono-function’ due to lack of community activities and night life.

MATERIALS AND METHODS

Agrabad area of Chittagong city has been selected here for urban regeneration. Existing urban park DC Hill has been studied for the evaluation of criteria’s for urban design proposal. For this paper, analogy is drawn from local context, to learn from their experience on how to tackle the problem. This paper is based on SWOT analysis for assessing and identifying the key criteria of urban design strategy to propose an appropriate revitalization strategy. Traditional urban renewal methods usually do not consider socio-cultural aspects beside climatic considerations. Cultural resilience might work as catalyst for urban design process besides climatic concerns. This research paper therefore provides a resilient analysis evolved from the local aspects, derived from Matthew Carmona’s ‘Values of urban design’. SWOT method used in an analogical manner to justify the socio-cultural aspect; while ECOTECT and DEPTHMAP software used to achieve optimized result to estimate expected climatic conditions for a site specific.

LITERATURE REVIEW

Oldenburg (1991) introduced "third place" where people can gather, put aside the concerns of work and home, and hang out simply for the pleasures of good company and lively conversation - are the heart of a community's social vitality and the grassroots of democracy. Oldenburg (1991) argues that third places are important for civil society, democracy, civic engagement, and establishing feelings of a sense of place. He calls one's "first place" the home and those where one lives. The "second place" is the workplace, where people may actually spend most of their time. Third places, then, are "anchors" of community life and facilitate broader, more creative interaction. The contribution (Trancik, 1943; Freidmann, 1987; Oldenburg, 1991) summarizes that the combined effect of common interest, social interaction and cultural value can contribute to a great extent to achieve dynamic third place or public realm. Design proposals need to restore traditional values and search for the meaning of 'lost space' to develop a dynamic urban design. Five major factors contributing to the lost urban space are the automobile, the modern movement in design, urban renewal and zoning policies, the dominance of private over public interests and changes in land use in the inner city. Prognosis for the future is that a more efficient use of urban land will make necessary a tighter and integrated urban form and that will offer the opportunity to recapture the lost spaces, and trigger regeneration of public realm. Urban design is the process by which better urban environment comes about (Carmona, 2001).

Table 1: Chittagong facts.

Population	3.3 million
Area	100 sq mile (260 sq km)
Total open spcae	200 acres (0.80 sq km)
Available Open area average:	0.06 acres per 1000 people
Required area average	4 acres per 1000 people

Source: Chittagong Development Authority standard 1961.

Fig. 1: Deba, Agrabad, Ctg.

Source: Author

Fig. 2: Panorama of concerned site Deba(Water body) from west (19.3 Acre).

Source: Author.

Values of third place in urban design (Carmona, 2001): Character- A place with its own identity. Character in the townscape and landscape is ensured by responding to and reinforcing locally distinctive patterns of development, landscape and culture; Continuity and Enclosure- A place where public and private spaces are clearly distinguished. The continuity of street frontages and the enclosure of space by development define private and public areas; Public realm- A place with attractive and successful outdoor areas. The public spaces and routes that is attractive, safe, uncluttered and work effectively for all in society, including disabled and elderly people; Ease of Movement- Accessibility and local permeability by making places that connect with each other and are easy to move through, putting people before traffic and integrating land uses and transport; Legibility- A place that has a clear image and is easy to understand. Legibility is ensured by development that provides recognizable routes, intersections and landmarks to help people find their way around; Adaptability- A place that can change easily. Development that can respond to changing social, technological and economic conditions and context; Diversity- A place with variety and choice can be promoted through a mix of compatible developments and uses that work together to create viable places that respond to local needs; Social Learning- Interactive skills of Social Learning- a)

Openness, b) Trust, c) Willingness to appreciate other points of view, d) Search for ways of accommodating all interests, e) Planners act as challenging intermediates between communities and powerful structures of the society (Freidmann, 1987). On the whole, a responsive and dynamic urban design can be achieved by judicious development, appropriate conservation and ensuring public participation.

CASE STUDY: DC Hill Park, Chittagong, Bangladesh

Fig. 3: South west view of DC Hill Park.

Source:http://www.thedailystar.net/locals-oppose-2-resident-govt-officials-resistance-24038

Information was gathered through interviews, personal experience and print media. ‘DC Hill Park’ (open space for national and cultural gathering) was constructed on Buddhist Temple Road; open space-cum-park for taking stroll or holding programmes marking different festivals or national days in the city. But, the park can hardly make room for the huge crowd and audiences during the national days like Ekushey February, Pahela Baishakh, Independence Day, Victory Day or different religious festivals. Despite having an area of 5.2 acres of land, the park is facing space shortage since a huge valley on the eastern side of the DC Hill has been left detached and unused. This public place has been studied and analysed to achieve criteria’s for urban design proposal.

Table 2: SWOT analysis of DC Hill Park, Chittagong, Bangladesh.

Urban Design Objectives	**Strength**	**Weakness**
1. Character	Successful cultural image. Targeted middle income group. Economic: Considers general people benefit, makeshift wet market. Overwhelming public engagement. Sociocultural: Sense of community.	Unsocial activities create unwelcome image.
2. Public Realm	Presence of natural surveillance. Expanded territory of middle class recreation with respect about local culture and values. Combination of soft and hard pave.	Lack of safety and security within the boundary.
3.Ease of Movement	Free of access; inadequate parking facility; multiple entry and exits facilitates high level permeability.	Central location creates uncontrollable accessibility. Creates traffic congestion late evening; Combination of soft and hard pave.

4. Sustainability	Mixed paving allows precipitation.	Biodiversity disturbed by overcrowd.
5. Legibility	Perfect place for citizen to mingle and to engage; recognizable routes and landmarks.	Compact design amidst busy urban fabric. Open spaces and routes are unrecognizable.
6. Adaptability	Highly adaptable hub for cultural activities, social gathering and economic gain.	Lack of control and balance over makeshift daily market.
7. Diversity & Density	Agenda supports dense development.	Mainly for leisure and recreation, open access for lower income group in business, lack of coordination and support.
8. Sustainability	Improved drainage system, Less emission, ensured through redevelopment.	Imposing approach lacks sustainability. Lack of renewable energy consumption.
9. Decision making	Public opinions valued through protest by culture class.	Top down and bottom up opinions are often in conflict. No public participation in decision making. No equity in terms of economic engagement.
10. User Participation	Socio-cultural: active engagement of Local community enhanced social cohesion.	Target group is not focused.

Learning

DC Hill Park successfully engaged local community; open access created an image of barrier free access to the citizen. Physical aspect: 1) Enhanced public realm, 2) No consistency between aim and goal as the physical development is combined with social and environmental agenda, 3) Imposing establishment disturbed the biodiversity, 4) Urban design respected the site forces. Criticism: Lack of security complicated the issue of natural surveillance. Social aspect: 1) Social equity is neglected due to lack of public engagement, 2) Cultural value respected, 3) People not valued by each other, 4) Sense of community developed, 5) Attracts visitors from other parts of the city, 8) Seems to acquire long term goal in terms of sustainability. Criticism: 1) Overcrowded with visitors, 2) Peak hour (late evening) invites access traffic that creates traffic congestion to the main traffic flow. Economic aspect: Failed to achieve long term gain. Criticism: Vulnerable Lower income group accessibility. Public-private interdependence facing lack of coordination.

Check List derived from the case study for Evaluation

Character: a) Attractive image, b) Sense of identity; Public Realm: a) Accessibility (visual & physical), b) Variety of options (Vehicular & pedestrian), c) Safety & security (Natural Surveillance), d) Maintenance, e) Micro-climate (Comfort ; Daylight, wind & temperature); Legibility: a) Easily recognizable place, b) Sense of place; Diversity: a) Mixed use (small scale retail & Third place), b) Difference (building layout, form and tenure type); Adaptability: a) A place should allow changes, b) Changing direction derive from existing trends, c) Cultural involvement brings social cohesion and community engagement into the context, d) Urban design should incorporate open public space to achieve equity.

Table 3: Problem, vision and action.

What to change?	Aim?	How?
Image	Public realm	Responsive Public space
Current trend	Culture	Adapt local trend

Responsive Urban Regeneration Proposal

Evaluation Criteria derived from the case studies have been grouped under two heads i.e. Public Realm and Adaptability and evaluated for a responsive regeneration proposal.

Public Realm: Attributes that contribute towards developing public realm are, Accessibility: visual and physical; Variety of options: Vehicular and pedestrian; Microclimate and comfort.

Adaptability: Attributes that contribute towards developing adaptability are: Value culture; respect current trend; Engage community.

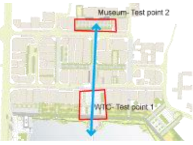
Fig. 4: Test point 1(museum) & 2(proposed site for World Trade Center).

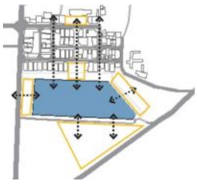
Fig. 5: Accessibility (visual & physical).

Fig. 6: Rural house unit around water body(pond) or common interest.


Fig. 7: Internal court as local trend or common interest.


Fig. 8: front door spaces

Thermal comfort analysis: Micro-Climate

Summer analysis: The Test point 2 (see Fig.4) is a controversial site as there is a 20 story structure (World Trade Centre) under construction and would invite enormous traffic flow. Daylight factor analysis has been generated to find the future effect of this high rise building in the site area. Result shows the preference of design for the plaza with or without the WTC tower. Analysis has done both for summer and winter to find the optimal solution for test point 2. Daylight factor calculation is based on a scale ranged from 0 to 100 where the colour variation from blue to yellow shows the gradual increase in intensity. The blue colour means least shaded zone and yellow colour means most lighted zone. From the summer analysis it is clearly visible that Test point 1 is getting better daylight without the tower. Test point 2 (Fig.4) is in average showing the similar result. The existence of the tower is creating more shaded zone in point 3 and surrounding. Point 4,5,6 have also potentiality to become successful public space. Winter analysis: Winter is not severe in Chittagong as the city is having moderate temperature due to close proximity to sea.

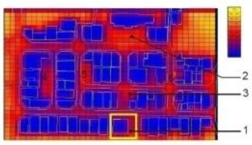

Figure 9: Summer, 15 April, with Tower

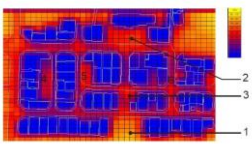

Figure 10: Summer, 22 April, without Tower

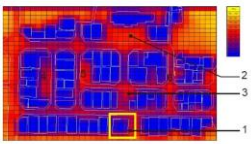

Figure 11: Winter, 15 December, with Tower

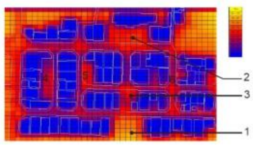

Figure 12: Winter, 15 December, without Tower

Shadow analysis: Daylight factor

Shadow analysis simulation(Fig.13-20) tested in summer on 15 April at three different times (9.00 am, 02.00 pm, 05.00 pm) of the day with and without the tower.

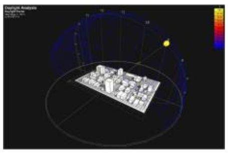

Figure 13: Summer, 15 April, 9.00 am with tower

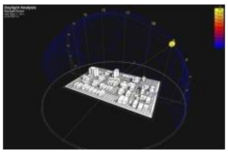

Figure 14: Summer, 15 April, 9.00 am without tower

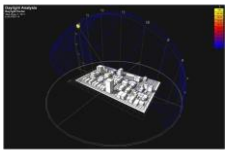

Figure 15: Summer, 15 April, 2.00 pm with tower

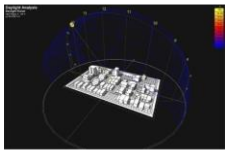

Figure 16: Summer, 15 April, 2.00 pm without tower

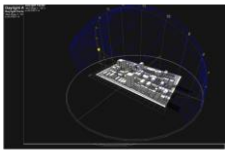

Figure 17: Summer, 15 April, 5.00 pm with tower

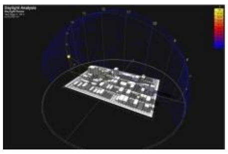

Figure 18: Summer, 15 April, 5.00 pm without

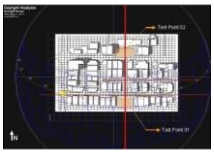

Figure 19: Summer, 15 April, 02 pm, without Tower

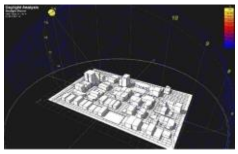

Figure 20: Summer, 15 March, 02 pm

Thermal impact simulation: Simulation generated for two test points to figure out the type of pave prior to construction. Thermal impact on Test point 1; Az 01- Applied zone; Cz 01- Comfort zone; Human Comfort level: 18°C to 26°C; Human deep body temperature: 37°C; Material assumed for Test point 01 is hard type pave. The result shows that the Az 01 is exceeded the human comfort zone level (Cz 01). The result indicates that the proposed pave material of the test zone 01 should be softer and less heat absorbing. Vegetation might be applied to decrease the heat absorption rate. For winter the test result is negligible for pave consideration.

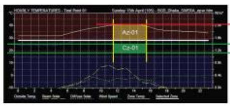

Figure 21: Summer 15 April 2.00 pm average temperature 25.9°C

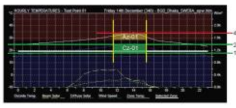

Figure 22: Winter: 14 Dec, 2.00 pm average temperature 19.2°C

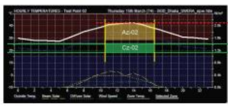

Figure 23: Summer 15.03.07, 2.00 pm average temp. 25.9°C

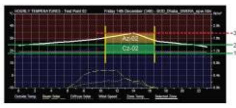

Figure 24: Summer 14.12.07, 2.00 pm average temp. 19.2°C

Thermal impact on Test point 02; Az 01- Applied zone; Cz 02- Comfort zone; Human Comfort level: 18°C to 26°C; Human deep body temperature: 37°C; Material assumed for Test point 01 is hard type pave. The result shows that the Az 02 is exceeded the human comfort zone level (Cz 02). The result indicates that the proposed pave material of the Test zone 02 should be softer and less heat absorbing. Vegetation might be applied to decrease the heat absorption rate. For winter the test result is negligible for pave consideration. Evaluation: For modelling and its parameters analysis- the ECOTECT 5.2v is used for this analysis. The detailed precise analysis according to the surrounding, trees, material assign, the daily cloud overcast is not applicable here but for overall result it is very useful. These test results helps designer to predict about the future situation and opens up the opportunity to find out the way for better optimized solution.

RESULT AND DISCUSSION

Outcomes from micro-climatic analysis and case study analysis clearly indicate the need for a public space for the local community. Ray Oldenburg (1989, 1991) argues that third places are important for civil society, democracy, civic engagement, and establishing feelings of a sense of place. Oldenburg calls one's "first place" the home and those that one lives with. The "second place" is the workplace — where people may actually spend most of their time. Third places, then, are "anchors" of community life and facilitate and foster broader, more creative interaction. All societies already have informal meeting places; what is new in modern times is the intentionality of seeking them out as vital to current societal needs. Oldenburg suggests these hallmarks of a true "third place": free or inexpensive; food and drink, while not essential, are important; highly accessible: proximate for many (walking distance); involve regulars – those who habitually congregate there; welcoming and comfortable; both new friends and old should be found there. Built environment should posses a socio-cultural emotion for the longer achievement of urban design strategy which might be termed as revitalization.

CONCLUSION

This paper shows that lost spaces can be transformed into third place for the regeneration of otherwise deteriorating area. Regeneration of Deba (water body) is not simply an urban design project but stands strategically against ongoing trends of encroachment with holistic consideration of the interest of all. The dilapidated negative spaces are vanishing in the name of development without any concern to our socio-cultural values. Respecting local trend through internal courtyard (See Figure 7,8) as cultural root of traditional rural house unit around central water body; works as common interest for the community. Cultural values can act as driving forces to encourage community mixing and social cohesion. Besides establishing physical connections to the site concerned, cultural values reflected within modern forms are important in making these urban spaces to fit into the context. Third place is a setting beyond home and work where people can often relax in good company or in a regular basis. Agrabad needs to change its mono-function image and the lost spaces lying beneath have great potentiality to get it with. Reclaiming the lost spaces of Agrabad and transform those into third place can give a relief to the city people. This exercise shows that culture could inspire the urban design process.

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ID: SEM 017

POTENTIAL OF RECYCLED GLASS AS CEMENTITIOUS MATERIAL IN CONCRETE

M. H. Rahman[1]*, A. A. Shames[1]& G. M. S. Islam[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.*
**Corresponding Author, <mhrahman.cuet@gmail.com>*

ABSTRACT

Million tons of waste glass is being generated annually in the Bangladesh. A considerable part of these is disposed as landfills which is unsustainable considering the environment. This paper aims to gather background information on use of recycled glass in concrete. These include use as aggregates and cementitious materials. Efforts have been paid to recycle waste glass in concrete as replacement of aggregate. These initiatives neglected the reactive nature of glass in concrete, because of slow reaction due to the relatively large (millimetre-scale) size of glass particles. To overcome the drawbacks of recycled glass aggregate in concrete the material, if milled down to micro-scale particle size, is expected to undergo pozzolanic reactions with cement hydrates, forming secondary calcium silicate hydrate (CSH).
The present review shows that waste glass, if ground finer than 100µm could show pozzolanic behaviour. The presence of the Pozzolanic Glass Powder in concrete could also intensify the plastic properties of concrete. Milled waste glass was also found to suppress alkali-silica reactions hence increase durability of concrete. The air content of concrete required for resistance to freezing and thawing was not disturbed with the addition of milled waste glass. Use of milled waste glass in concrete as partial replacement of cement represents an important step towards development of sustainable (environmentally friendly, energy-efficient and economical) concrete-based infrastructure systems.

Keywords: Waste glass; recycling; supplementary cementitious material; energy; environment

INTRODUCTION

Disposal of waste glass in landfills is costly. The non-biodegradable nature of glass further complicates the environmental impact of its disposal in landfills. On the other hand, manufacturing of cement, a key ingredient used for the production of concrete, is a major source of greenhouse gas emissions. Manufacturing of one ton of cement results emission of approximately one ton of carbon dioxide (CO_2) to the atmosphere. Cement production also involves emission of moderate quantities of NO_x, SO_x, and particulates. The use of supplementary cementitious materials (SCMs) to offset a portion of the cement powder in concrete is a promising method for reducing the environmental impact of the industry. The most influential environmental concerns in the use of concrete for construction are the production of greenhouse gases during the manufacturing of cement powder, and the consumption of non-renewable resources as raw materials.

Several industrial by-products have been used successfully as SCMs, including silica fume (SF), ground granulated blast furnace slag (GGBFS), and fly ash. These materials are used to create blends of cement which can improve concrete durability, early and long term strength, workability, and economy (Detwiler, Bhatty, & Bhattacharja, 1996). One material which has potential as a SCM but which has not yet achieved the same commercial success is waste bottle glass. Glass has a chemical composition and phase which is comparable to traditional SCMs. It is abundant, can be of low economic value and is often land filled (Byars, Meyer & Zhu, 2003).

It is realized that mixed-color waste glass offers desired chemical composition and reactivity for use as a supplementary cementitious material (SCM) for enhancing the chemical stability, pore system characteristics, moisture resistance and durability of concrete. Previous efforts to recycle waste glass in concrete have focused on the use of crushed glass as replacement for aggregate in concrete. These efforts neglected the reactive nature of glass in concrete, which was slowed down due to the relatively large (millimetre-scale) size of glass particles (Nassar & Soroushian, 2012). Milling of glass to micrometer scale particle size, for accelerating the reactions between glass and cement hydrates, can bring major energy, environmental and cost benefits when cement is partially replaced with milled waste glass for production of concrete. Cattaneo *et al.* (2008) reported that recycling of each ton of glass saves over one ton of natural resources, and recycling of every six tons of container glass results in the reduction of one tone of CO_2 emission.

Fig. 1 Empty raw glass bottle

Fig. 2 Milled glass powder

This research aims to explore various factors associated with the use of the milled waste glass as partial replacement of cement in concrete. In This regard review the merits of using powdered waste glass as a supplementary cementitious material, replacing a portion of the cement powder used in concrete in order to improve the environmental impact of the concrete industry by reducing the green-house gases produced and raw materials consumed in cement production, and by diverting a waste material from landfills is carried out. In addition, causes of alkali silica reactivity versus pozzolanic reactivity with powdered waste glass as a SCM, including cement alkalinity and chemical composition is also studied.

WASTE GLASS AS POZZOLANA (COMPOSITION)

The lower reactivity of waste glass compared to cement powder limits its use as pozzolans in concrete. Overcoming this limitation requires activation methods for increasing the reactivity of SCMs. A comparison of the methods by Shi and Day indicated that the most effective method for developing reactivity in natural pozzolans was chemical activation, which improved both the initial reaction rate and the final strength. The reactivity of the treated pozzolanic material was measured in terms of the compressive strength and total hydration of the material. Comparison between chemical compositions of various pozzolainc materials are given in Table 1. With the exception of Al_2O_3, Na_2O and CaO, the percentages of the main constituents are similar. It is reasonable to expect that success may be achieved by applying similar treatment to waste glass in order to improve its pozzolanic properties. The properties which influence the pozzolanic behaviour of waste glass, and most pozzolans in general, are fineness, chemical composition, and the pore solution present for reaction. Based on observed compressive strengths, Meyer *et al,* (1996) postulated that below 45 µm, glass may become pozzolanic. The pozzolanic properties of glass are first notable at particle sizes below approximately 300 µm, and below 100 µm, glass can have a pozzolanic reactivity which is greater than that of fly ash at low percent cement replacement levels and after 90 days of curing (Shi *et al.*, 2005). Table 2 gives comparison between chemical compositions of various pozzolanas in light of ASTM C618-12a. It has been noted from the literature that the chemical composition of waste glass conforms to that required by ASTM to declare the product as a pozzolana to use in concrete.

Table 1. Comparing Chemical composition of pozzolans with OPC

Compound	Waste Glass (Nassar & Soroushian, 2012)	Volcanic Ash (Mostafa, 2005)	Volcanic Pumice (Warren, 1994)	Slag (Warren, 1994)	Silica Fume (Binici, 2007)	Fly Ash (Islam *et al*, 2011)	OPC (Ryou, 2006)
SiO_2	68	73.7	65.7	35	90.9	59.2	20.3
Al_2O_3	7	12.3	16.7	12	1.12	25.6	4.7
Fe_2O_3	<1	2.2	3.6	1	1.5	2.9	3.0
CaO	11	1.1	3.3	40	0.7	1.1	61.8
MgO	<1	0.2	0.9	-	0.78	0.3	3.3
K_2O	<1	3.9	3.1	-	-	0.9	0.6
Na_2O	12	3.6	4.5	.3	-	0.2	0.2
SO_3	-	0.3	0.7	9	0.4	0.3	3.6
LOI	-	3.1	2.4	1	3.0	1.4	-

Table 2. ASTM C618-12a criteria for SCM and their composition

ASTM C 618-12a Requirements		Waste Glass	Slag	Silica Fume	Fly Ash
$SiO_2+Al_2O_3+Fe_2O_3$, min%	70	68.4	48	93.48	62.04
SO_3, max %	4	0.17	9	0.38	4.3
Moisture Content, max %	3	-	-	-	-
LOI, max %	10	-	1	3	2.1

POZZOLANIC REACTIVITY OF WASTE GLASS

The pozzolanic reaction occurs when amorphous silica dissolves into a solution with a high pH in the presence of calcium. Excess $Ca(OH)_2$ exists in the highly alkaline pore solution of hydrating cement. If a material provides a readily soluble form of silica with high surface area, the pozzolanic reaction will take place. The dissolved silica and $Ca(OH)_2$, along with alkalis and aluminates, will form a reaction product that can be chemically and structurally similar to C-S-H. The process of hydration is essentially the formation of minute crystals of calcium and gels from the solution of cement and water and continues for a long period. The hydration of different constituent compounds of portland cement is illustrated as follows:

For tricalcium silicate (C_3S):

$$2C_3S+6H_2O = C_3S_2.H_2O + 3Ca(OH)_2$$

For tricalcium silicate (C2S):

$$2C_2S+4H_2O = C_3S_2.H_2O + Ca(OH)_2$$

With this readily available $Ca(OH)_2$ and SiO_2 the following equilibrium was proposed by researches (Greenberg *et al.* 1961; Urhan *et al.* 1987)

$$SiO_2(s) + Ca_2+(aq) + 2OH^-(aq) = n_1CaO\cdot SiO_2\cdot n_2H_2O(s) \quad (1)$$

As with C-S-H the product is also variable (n_1 and n_2). The Ca/Si ranges from 0.75 to 1.75 (Massazza, 1998). It is prudent to identify the factors which influence the pozzolanic reaction in the system; for example particle size of the pozzolanic material. The particle size of the pozzolan is one factor which is known to affect its reactivity and therefore the hydration of the system. A decreased particle size leads to an increase in surface area. The reactivity of most SCMs can be enhanced through mechanical treatment, where grinding is used to reduce the particle size of the material, increasing the surface area available for reaction.

CHALLENGE IN THE USE OF WASTE GLASS AS POZZOLANS

Use of waste glass in concrete first focused on aggregate replacement. Phillips *et al.* (1972) attempted to introduce waste glass as a partial replacement of the fine and coarse aggregate in concrete masonry block, the production and use of which is less conducive to ASR gel production, and which allow early age monitoring. They observed that the main challenges in utilizing waste glass would be the removal of contamination, processing, and cost. Johnston *et al.* (1974) also considered waste glass as a coarse aggregate, and found that only by using low alkali cement or high percentages of pozzolans such as fly ash could satisfactory strength and expansion performance be achieved to one year.

Experimentation with the addition of waste glass in concrete has been closely related to the study of alkali-silica reactivity, where production of ASR gel in the presence of reactive aggregates causes damaging expansion in concrete. It is important to impart only that the mechanism by which a reactive aggregate can form ASR gel, leading to expansion and ultimately causing damage to concrete, is dependent on the presence of amorphous silica, which is the major component of waste glass, the presence of alkali hydroxides in pore solution, and cement reaction products (Diamond *et al.* 1989). For most cases, both the local maximum and minimum expansion occurs at a very small particle size, typically less than 100 µm (Diamond *et al.* 1974).

The use of any supplementary cementing material will depend on its performance, namely its strength, durability, and volumetric stability over time. In the case of waste glass, this performance is further challenged by the tendency for ASR to occur. The controlling process between a beneficial pozzolanic reaction, which would improve the performance of glass as an SCM, and a damaging ASR is the production of either a stable or swelling product. Depending on several factors, including calcium content, particle size, and alkalinity, the dissolved silica will re-polymerize into expansive gel, hydrate into C-S-H, or a combination of both (Buchwald *et al.* 2003). Availability of calcium ions in combination with a relatively high rate of C-S-H formation will favour the pozzolanic reaction, and over time, any ASR product will take on the texture of C-S-H. Following the reaction in Eq. (1) proposed by Urhan *et al.* (1987) a type of C-S-H is formed. When the reaction of glass results in ASR gel, the chemical equation is similar, however, sodium, potassium, or other alkalis may be substituted for calcium as shown by Eq. (3) (Glasser *et al.* 1981).

$$SiO_2+ 2Na^+(K^+) + 2OH^-\rightarrow Na_2(K_2)SiO_3\cdot H_2O \quad (2)$$

The final product is more likely similar to a precipitate of the composition given in Eq. (3), existing within a sol/gel matrix of calcium silicate hydrate with a Na2O/SiO2 ratio near 0.19.

$$0.16Na_2O \cdot 1.4CaO \cdot SiO_2 \cdot XH_2O \tag{4}$$

This system may be the cause of variability in the swelling properties of ASR products (Helmuth *et al.* 1993) have suggested that the mechanism which differentiates between pozzolanic and ASR products could be simply the degree of aggregation or particle size of the silica source. According to the requirements of ASTM standards (ASTM C618-12a) for the use of natural pozzolans, Table 2., glass has the potential to acceptably function as an SCM. However, proper guidelines must be developed to control the ASR/pozzolanic reaction and influence a non-destructive, non-swelling product. The form of this product has not been identified. It may be a pozzolanic form of C-S-H, which has the potential to contribute additional strength.

BENEFIT/COST ANALYSIS

In local market the price of waste glass is approximately 2 tk/kg. In addition, the processing and grinding may increase the cost up to 2.5 tk/kg. Considering price of a 50-kg cement bag as 450 tk, replacing 10 and 20% cement by glass powder can reduce its price by 7.22 and 14.44% respectively. Table 3 gives a comparison in cost analysis considering the product gives same performance as OPC.

Table 3. Comparison of price considering cement is replace by milled glass powder

Cement bag of 50 kg	10% glass replacement	20% glass replacement
Price 450 Tk.	Price 417.5 Tk.	Price 385 Tk.

CONCLUSIONS

The use of milled waste glass as partial replacement of cement in recycled aggregate concrete may results in enhanced durability characteristic such as sorption, chloride permeability, and freeze–thaw resistance through improvement in pore system characteristics, filling effect of glass particles, and conversion of CH to C–S–H available in the concrete pore water. Milling of waste glass to sub-micron particle size is a key to benefit from its pozzolanic reaction. The high surface area of milled waste glass changes the kinetics of chemical reaction towards beneficial pozzolanic reaction utilizing the available alkalis before production of a potential ASR gel. The ASR is an issue to consider carefully and needs further study. In general, considering the similar performance with replaced material glass addition can reduce significant cost of cement production. In addition, glass replacement can save the environment by reducing green-house gas and particulate production.

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POTENTIAL OF RECYCLED GLASS AS CEMENTITIOUS MATERIAL IN CONCRETE

M. H. Rahman[1]*, A. A. Shames [1] & G. M. S. Islam[1]

[1]Department of Civil Engineering, Chittagong University of Engineering and Technology (CUET), Chittagong-4349, Bangladesh.
**Corresponding Author, < mhrahman.cuet@gmail.com>*

ABSTRACT

Million tons of waste glass is being generated annually in the Bangladesh. A considerable part of these is disposed as landfills which is unsustainable considering the environment. This paper aims to gather background information on use of recycled glass in concrete. These include use as aggregates and cementitious materials. Efforts have been paid to recycle waste glass in concrete as replacement of aggregate. These initiatives neglected the reactive nature of glass in concrete, because of slow reaction due to the relatively large (millimetre-scale) size of glass particles. To overcome the drawbacks of recycled glass aggregate in concrete the material, if milled down to micro-scale particle size, is expected to undergo pozzolanic reactions with cement hydrates, forming secondary calcium silicate hydrate (CSH).

The present review shows that waste glass, if ground finer than 100µm could show pozzolanic behaviour. The presence of the Pozzolanic Glass Powder in concrete could also intensify the plastic properties of concrete. Milled waste glass was also found to suppress alkali-silica reactions hence increase durability of concrete. The air content of concrete required for resistance to freezing and thawing was not disturbed with the addition of milled waste glass. Use of milled waste glass in concrete as partial replacement of cement represents an important step towards development of sustainable (environmentally friendly, energy-efficient and economical) concrete-based infrastructure systems.

Keywords: Waste glass; recycling; supplementary cementitious material; energy; environment

INTRODUCTION

Disposal of waste glass in landfills is costly. The non-biodegradable nature of glass further complicates the environmental impact of its disposal in landfills. On the other hand, manufacturing of cement, a key ingredient used for the production of concrete, is a major source of greenhouse gas emissions. Manufacturing of one ton of cement results emission of approximately one ton of carbon dioxide (CO_2) to the atmosphere. Cement production also involves emission of moderate quantities of NO_x, SO_x, and particulates. The use of supplementary cementitious materials (SCMs) to offset a portion of the cement powder in concrete is a promising method for reducing the environmental impact of the industry. The most influential environmental concerns in the use of concrete for construction are the production of greenhouse gases during the manufacturing of cement powder, and the consumption of non-renewable resources as raw materials.

Several industrial by-products have been used successfully as SCMs, including silica fume (SF), ground granulated blast furnace slag (GGBFS), and fly ash. These materials are used to create blends of cement which can improve concrete durability, early and long term strength, workability, and economy (Detwiler, Bhatty, & Bhattacharja, 1996). One material which has potential as a SCM but which has not yet achieved the same commercial success is waste bottle glass. Glass has a chemical composition and phase which is comparable to traditional SCMs. It is abundant, can be of low economic value and is often land filled (Byars, Meyer & Zhu, 2003).

It is realized that mixed-color waste glass offers desired chemical composition and reactivity for use as a supplementary cementitious material (SCM) for enhancing the chemical stability, pore system characteristics, moisture resistance and durability of concrete. Previous efforts to recycle waste glass in concrete have focused on the use of crushed glass as replacement for aggregate in concrete. These efforts neglected the reactive nature of glass in concrete, which was slowed down due to the relatively large (millimetre-scale) size of glass particles (Nassar & Soroushian, 2012). Milling of glass to micro-meter scale particle size, for accelerating the reactions between glass and cement hydrates, can bring major energy, environmental and cost benefits when cement is partially replaced with milled waste glass for production of concrete. Cattaneo *et al.* (2008) reported that recycling of each ton of glass saves over one ton of natural resources, and recycling of every six tons of container glass results in the reduction of one tone of CO_2 emission.

Fig. 1 Empty raw glass bottle

Fig. 2 Milled glass powder

This research aims to explore various factors associated with the use of the milled waste glass as partial replacement of cement in concrete. In This regard review the merits of using powdered waste glass as a supplementary cementitious material, replacing a portion of the cement powder used in concrete in order to improve the environmental impact of the concrete industry by reducing the green-house gases produced and raw materials consumed in cement production, and by diverting a waste material from landfills is carried out. In addition, causes of alkali silica reactivity versus pozzolanic reactivity with powdered waste glass as a SCM, including cement alkalinity and chemical composition is also studied.

WASTE GLASS AS POZZOLANA (COMPOSITION)

The lower reactivity of waste glass compared to cement powder limits its use as pozzolans in concrete. Overcoming this limitation requires activation methods for increasing the reactivity of SCMs. A comparison of the methods by Shi and Day indicated that the most effective method for developing reactivity in natural pozzolans was chemical activation, which improved both the initial reaction rate and the final strength. The reactivity of the treated pozzolanic material was measured in terms of the compressive strength and total hydration of the material. Comparison between chemical compositions of various pozzolainc materials are given in Table 1. With the exception of Al_2O_3, Na_2O and CaO, the percentages of the main constituents are similar. It is reasonable to expect that success may be achieved by applying similar treatment to waste glass in order to improve its pozzolanic properties. The properties which influence the pozzolanic behaviour of waste glass, and most pozzolans in general, are fineness, chemical composition, and the pore solution present for reaction. Based on observed compressive strengths, Meyer *et al,* (1996) postulated that below 45 µm, glass may become pozzolanic. This size can be achieved by using a grinding operation with the help of “Ball Mill” which is generally used in Cement industry to grind cement clinker. The pozzolanic properties of glass are first notable at particle sizes below approximately 300 µm, and below 100 µm, glass can have a pozzolanic reactivity which is greater than that of fly ash at low per cent cement replacement levels and after 90 days of curing (Shi *et al.,* 2005). Being strength as a key factor to evaluate concrete performance, several research shows that, at the higher age recycled glass concrete (15% to 20% of cement replaced) with milled waste glass powder provide compressive strengths exceeding that of control concrete (Nassar & Soroushian, 2011). Table 2 gives comparison between chemical compositions of various pozzolanas in light of ASTM C618-12a. It has been noted from the literature that the chemical composition of waste glass conforms to that required by ASTM to declare the product as a pozzolana to use in concrete.

Table 1. Comparing Chemical composition of pozzolans with OPC

Compound	Waste Glass (Nassar & Soroushian, 2012)	Volcanic Ash (Mostafa, 2005)	Volcanic Pumice (Warren, 1994)	Slag (Warren, 1994)	Silica Fume (Binici, 2007)	Fly Ash (Islam *et al*, 2011)	OPC (Ryou, 2006)
SiO_2	68	73.7	65.7	35	90.9	59.2	20.3
Al_2O_3	7	12.3	16.7	12	1.12	25.6	4.7
Fe_2O_3	<1	2.2	3.6	1	1.5	2.9	3.0
CaO	11	1.1	3.3	40	0.7	1.1	61.8
MgO	<1	0.2	0.9	-	0.78	0.3	3.3
K_2O	<1	3.9	3.1	-	-	0.9	0.6
Na_2O	12	3.6	4.5	.3	-	0.2	0.2
SO_3	-	0.3	0.7	9	0.4	0.3	3.6
LOI	-	3.1	2.4	1	3.0	1.4	-

Table 2. ASTM C618-12a criteria for SCM and their composition

ASTM C 618-12a Requirements		Waste Glass	Slag	Silica Fume	Fly Ash
$SiO_2+Al_2O_3+Fe_2O_3$, min%	70	68.4	48	93.48	62.04
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POZZOLANIC REACTIVITY OF WASTE GLASS

The pozzolanic reaction occurs when amorphous silica dissolves into a solution with a high pH in the presence of calcium. Excess $Ca(OH)_2$ exists in the highly alkaline pore solution of hydrating cement. If a material provides a readily soluble form of silica with high surface area, the pozzolanic reaction will take place. The dissolved silica and $Ca(OH)_2$, along with alkalis and aluminates, will form a reaction product that can be chemically and structurally similar to C-S-H. The process of hydration is essentially the formation of minute crystals of calcium and gels from the solution of cement and water and continues for a long period. The hydration of different constituent compounds of portland cement is illustrated as follows:

For tricalcium silicate (C_3S):

$$2C_3S+6H_2O = C_3S_2.H_2O + 3Ca(OH)_2$$

For tricalcium silicate (C_2S):

$$2C_2S+4H_2O = C_3S_2.H_2O + Ca(OH)_2$$

With this readily available $Ca(OH)_2$ and SiO_2 the following equilibrium was proposed by researches (Greenberg *et al.* 1961; Urhan *et al.* 1987)

$$SiO_2(s) + Ca_2+(aq) + 2OH^-(aq) = n_1CaO\cdot SiO_2\cdot n_2H_2O(s) \qquad (1)$$

As with C-S-H the product is also variable (n_1 and n_2). The Ca/Si ranges from 0.75 to 1.75 (Massazza, 1998). It is prudent to identify the factors which influence the pozzolanic reaction in the system; for example particle size of the pozzolanic material. The particle size of the pozzolan is one factor which is known to affect its reactivity and therefore the hydration of the system. A decreased particle size leads to an increase in surface area. The reactivity of most SCMs can be enhanced through mechanical treatment, where grinding is used to reduce the particle size of the material, increasing the surface area available for reaction.

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PROSPECTS OF POROUS CONCRETE IN CIVIL ENGINEERING APPLICATIONS

M. F. Zaman[1*] & G. M. S. Islam[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong, 4349, Bangladesh.*
**Corresponding Author, <ferdouszaman85@yahoo.com>*

ABSTRACT

Porous concrete has gained attention of engineers due to its environment friendly nature. Potential application includes road construction, riverbank protection using porous concrete block. This review is accomplished to report recent advances in the knowledge base which are relevant to the potential application of porous concrete for Civil Engineering construction considering its different properties. Compressive strength and porosity plays an important role in the behaviour of porous concrete. In the current review, compressive strength and porosity of porous concrete as well as the relationship between them are discussed on the basis of existing research carried out so far. To seek the feasibility of porous concrete for using in the road surface, its response to abrasion and impact loading are also explored. In addition, this paper includes the assessment of hydraulic conductivity of porous concrete in relation to infiltration capability of this sustainable material to suggest application in Civil Engineering Applications.

Keywords: Porous Concrete, Compressive Strength, Porosity, Abrasion, Permeability

INTRODUCTION

Porous Concrete consists of same basic components as conventional concrete but is characterized to have high porosity and permeability. Using a gap-graded aggregate with a very low proportion of fine aggregates the porosity is achieved (Pindado *et al.*, 1999). Porous concrete may be used in various applications such as permeable concrete for pavement, base course, noise absorbing concrete, concrete bed for vegetation or living organism, thermal insulated concrete and various other Civil Engineering and Architectural applications (Chindaprasirt *et al.*, 2007)

It is well known that the mechanical behaviour of a building material is dependent on its structure. The presence of pores can have adverse effect on the material's mechanical properties such as crushing strength (Lian *et al.*, 2011).Besides strength and permeability, the abrasion durability and raveling under moving traffic loads are also concerns about pervious concrete pavement. For reduced strength and contact area between neighbouring aggregate particles associated with high voids, PCPC is more vulnerable than conventional Portland cement concrete (PCC) to cracking and ravelling under traffic loads (Dong *et al.*, 2013). Porous concrete has a moderate static Strength compared to normal concrete due to its high intentional meso-size air pore content, while its fragmentation behaviour under dynamic loading is considerably different from that of normal concrete(Ozbek *et al.*, 2013). Seeking relationship between porosity and compressive strength of porous concrete was set as aim of this study.

MATERIAL COMPOSITION

The compositions that are used to prepare porous concrete consist of coarse aggregates, Ordinary Portland Cement (OPC) and water. However, admixtures such as quarry sand, silica fume and super plasticizer are also used in some of the mixes to give variation in strength and fresh properties (Lian *et al.*, 2011). Some other admixtures used in porous concrete are thickening (cohesive) agent (water-soluble cellulose based polymer powder), Water reducing agent, set retarder etc (Bhutta *et al.*, 2011, Chindaprasirt *et al.*, 2007, Ozbek *et al.*, 2013). Generally in porous concrete 5-13 mm, 2–4 mm or 4–8 mm or both size rages (50% each), 13.2–4.75 mm; 9.5–6.7 mm; 9.5–4.75 mm aggregate sizes are used(Chindaprasirt *et al.*, 2007, Lian *et al.*, 2011and Ozbek *et al.*, 2013). Uniformly graded aggregates viz. 13–20 mm, 9.5–13 mm, 5–13 mm and 2.5–5 mm aggregates are also used in many cases (Bhutta *et al*., 2011 and Sumanasooriya *et al*., 2011).

EVALUATION OF POROUS CONCRETE

Void Ratio Test

Void ratio of hardened concrete is usually measured using test method for porous concrete as suggested by the Committee for Eco-concrete Research. Each of the specimens is kept in water for 24 hours and then the mass is taken in water (M_1). Specimen is then taken out of water and kept for air drying for 24 hours. After that final mass is taken (M_2). The total void ratio is obtained by dividing the difference between the initial mass (M_1) of the cylinder specimen in the water and the final mass (M_2) measured following air drying for 24 h with the specimen volume (V), where as ρM is the density of water. Equation 1 is used to obtain the total void ratio (A)..

$$A\% = 1 - \left(\frac{M2 - M1}{\rho M}\right) \times_{100}$$ ....Eq. (1) (Chindaprasirt *et al.*, 2007; Bhutta *et al*., 2011)

Compressive Strength

Compressive strength of concrete is determined in accordance with ASTM C39 (Chindaprasirt *et al.,* 2007). Approximately 300 kN load is applied per minute to concrete specimen in order to determine compressive strength. Three specimens are used in case of cylindrical specimen whereas in case of cubic specimen five specimens are used for compressive strength test.

Drop Weight Impact Testing

A steel impactor is used for the drop weight tests. Due to its high dynamic impedance, steel is selected as the impacting material. The longitudinal wave velocity of the steel used in the impactor is calculated adopting Equation (2)

$$C_i = \sqrt{\frac{K + \frac{4}{3}G}{P}}$$ ………………………...Eq. (2) (Ozbek *et al.*, 2013 and Ozbek *et al*., 2012)

Where, C_i= The longitudinal wave velocity of the steel used in the impactor

P= Density (g/cm^3)

K= Bulk modulus (GPa)

G= Shear modulus (GPa)

In the experiment, the specimen is placed vertically on a steel base structure, which also functions as a wave sink. Impactor is dropped from approximately 1.2 m striking the target specimens at velocities ranging between 4.0 and 4.7 m/s. A horizontal laser beam sent from a laser head is reflected from a 45^0 mirror so that it is then directed upwards in the vertical direction. The reflected vertical beam which hits the retro-reflective sticker attached in the bottom surface of the impactor rim reflects again and follows the same path back to the laser head. The particle velocity time histories of the interface between the target and the impactor are captured and afterwards analyzed to determine the impact stress applied on the target. A detailed view of how the laser beam is reflected and the testing system are schematically presented in Fig. 1. (Ozbek *et al.*, 2013 and Ozbek *et al*., 2012)

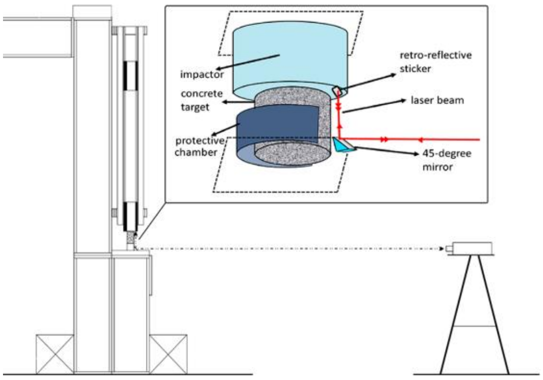

Fig 1: Drop Weight Impact Testing Setup (Ozbek *et al*., 2012 and Ozbek *et al*., 2013)

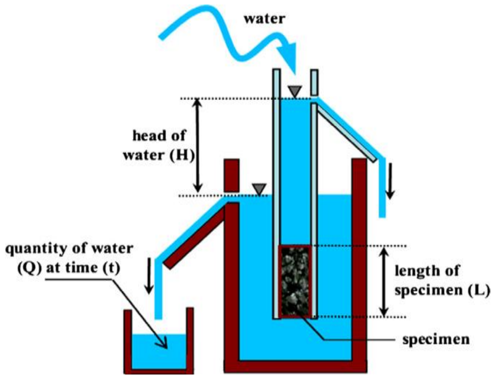

Fig 2: Permeability Testing Setup (Tho-in *et al*., 2011)

Abrasion Resistance Testing

a) Cantabro Abrasion Test: A Los Angeles (LA) abrasion machine without steel ball charges is used to conduct the Cantabro test in accordance with ASTM C131., the cylindrical specimen is weighed before the test and placed in the steel drum and tested at the rotating speed of 30 rpm. To characterize the abrasion resistance of PCPC the weight loss of the specimen before and after the test is used. Weight loss values are calculated usually every 50 revolutions (Wo *et al.,* 2011).

b) APA Abrasion Test: The APA is a type of multifunctional loaded-wheel testing system, in which, repeated loading can be applied to the beam specimens by three controllable loaded wheels to simulate actual situations on pavements. When a vehicle moves forward, its tires exert a horizontal force on aggregate particles on the pavement surface. These particles, if not strongly bonded to a pervious concrete body, they will ultimately be worn off by the moving vehicle tires and thus spalling/raveling distress will occur. During the testing the specimens are subjected to 5,000 cycles of repeated loads at the frequency of 2 cps (Wo et al., 2011 and Dong et al., 2013).

c) Surface Abrasion Test (ASTM C944): The surface abrasion test (ASTM 2012c) is a standard test method to determine the surface abrasion resistance of conventional concrete or mortar using the rotating cutter method and is potentially capable of evaluating abrasion resistance of PCPC. The surface abrasion test uses a rotating cutter mounted in the abrasion device to apply an abrasion force on the surface of PCPC specimens as shown in Fig. 4. After the test, the specimen is brush cleaned and its weight loss is recorded and used as an indicator of the abrasion resistance of PCPC (Dong et al., 2013).

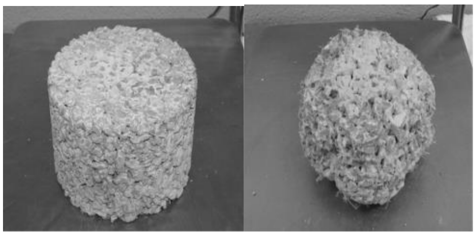

Fig.3. Specimens before and after Cantabro loss test

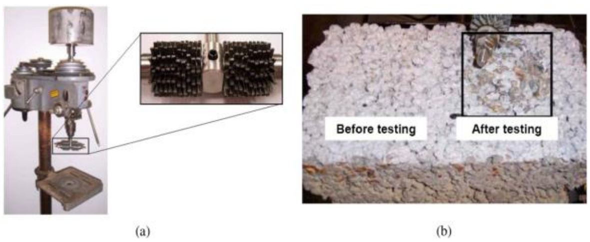

Fig.4. Surface abrasion test: (a) rotating-cutter drill press;

(Dong *et al.*, 2011) (b) specimen Before and after test (Dong *et al.*, 2011)

Permeability

The water permeability of Porous Concrete is tested using the constant head method. The schematic diagram of the experimental test set-up is shown in Fig. 2. The coefficient of water permeability (k) is calculated following Darcy's law as shown in Equation (3).

k=QL/Hat..........................Eq. (3)

Where k is the coefficient of water permeability (cm/s), Q is the quantity of water collected, (cm^3) over time t (s), L is the length of specimen (cm), H is the water head, (cm), A is the cross sectional area of specimen (cm^2) (Tho-in *et al*., 2011 and Hossain *et al*., 2011)

PROPERTIES OF POROUS CONCRETE

Compressive Strength

Compressive strength of porous concrete normally ranges between 12-18 MPa (Chindaprasirt *et al.*, 2007,Amde *et al., 201* ,Bhutta *et al*., 2011, Kevern *et al*., 2006 and Lian *et al*., 2011). In case of high performance concrete this range increases to 15-25 MPa (Bhutta *et al*., 2011). Compressive strength of porous concrete is dependent on various factors such as aggregate grading, flow value, compaction, void ratio, aggregate type etc. It is seen that compressive strength of porous concrete increases in a significant rate with the increase of flow value. Porous concrete which has a compressive strength of 15 MPa at a flow value of 150 mm is observed to attain 18 MPa at a flow value of 190 mm and 22 MPa at a flow value of 230 mm (Chindaprasirt *et al.*, 2007).

Aggregate grading has a substantial effect on the compressive strength of porous concrete as compressive strength of porous concrete decreases with increase in the size of aggregate used. Where compressive strength of porous concrete with aggregate size 13-20 mm is 7-12 MPa; this value increases to 10-15 MPa for aggregate size 5-13 mm and 13-20 MPa for aggregate size 2.5-5 mm (Bhutta *et al*., 2011, Hossain *et al*., 2011 and Sumanasooriya *et al*., 2011). Compaction energy also has an impact on the compressive strength of porous concrete. It is seen that porous concrete to which 10 kN-m/m^2 vibrating energy has a compressive strength of 8-10 MPa which is increased to 12-17 MPa at 40 kN-m/m^2 and 13-22 MPa at 90 kN-m/m^2. Compressive strength of porous concrete is also dependent on the aggregate material used. For stone chips this value normally ranges between 7-22 MPa which is as low as 3-7 MPa in case of brick chips (Hossain *et al*., 2011). In a case it is seen that compressive strengths of porous concretes are 11.8 MPa, 15.5 MPa and 15.8 MPa serially for Quartzite, limestone and dolomite being used as coarse aggregate (Lian *et al*., 2011). Compressive strength of porous concrete decreases with cement to aggregate ratio. It is found the range of 19-20 MPa for cement to aggregate ratio 0.42-0.44 whereas this value is in the range of 12.25-12.5 MPa for cement to aggregate ratio 0.17-0.20 (Sumanasooriya *et al*., 2011).

Impact Strength

Impact or Dynamic strength of porous concrete ranges 20-90 MPa depending upon various factors such as aggregate, cement paste composition, compaction of concrete and admixtures used. In a case it is seen that when river gravel is used as coarse aggregate dynamic strength is around 60 MPa which increases to around 75 MPa when crushed basalt is used as coarse aggregate. This value reaches to around 85 MPa when these two coarse aggregates are used in equal composition. Compaction plays a very important role in dynamic strength of concrete as dynamic strength may be as low as 20 MPa in case of low compaction (Ozbek *et al*., 2013 and Ozbek *et al*., 2012).

Abrasion Strength

Where Cantabro loss is no more than 20% for OGFC mixtures, the Cantabro loss values for Porous concrete mixtures fell within the range of 35 to 80% .But in case of loaded wheel abrasion test this value is as low as 0.5 to 2%and in case of surface abrasion test 0.2 to 0.5% .The possible reason for this difference is that some of the specimens fall apart because of their low strength during the first several impacts rather than aggregate particles are abraded away from the specimen surface during the Cantabro test where during the loaded wheel abrasion test and surface abrasion test the area under abrasion is very small in comparison with the area of the specimen (Wo *et al.*, 2011 and Dong *et al.*, 2013). But using different types of Latex and Fiber can be beneficial in case of abrasion strength of porous concrete by limiting the cantabro loss within 15 to 30%, loaded wheel abrasion loss within 0.5 to 2.5% and surface abrasion loss within 0.2 to 0.3% (Dong *et al.*, 2013). Using granite as coarse aggregate instead of limestone increases abrasion strength of porous significantly. In some cases loss due to abrasion of porous concrete with granite is even reduced to half of that of limestone (Wo *et al.*, 2011).

Permeability

Water permeability of porous concrete normally ranges between 0.07 to 1.22 cm/sec depending upon void ratio (Mccain and Dewoolkar., 2010, Wo *et al.*, 2011 and Hossain *et al.*,2011). Permeability of porous concrete increase with void ratio. In a case it is seen that where permeability is only 0.07 cm/sec at 10% void ratio it increases to 0.37 cm/sec at 28.2% void ratio. It is found in a study that for same void ratio, stone aggregate concrete produce higher permeability than that of brick aggregate concrete (Hossain *et al*.,2011).However pervious high-calcium fly ash geopolymer concrete poses a huge permeability as high as 2-6 cm/sce (Tho-in *et al*., 2011).

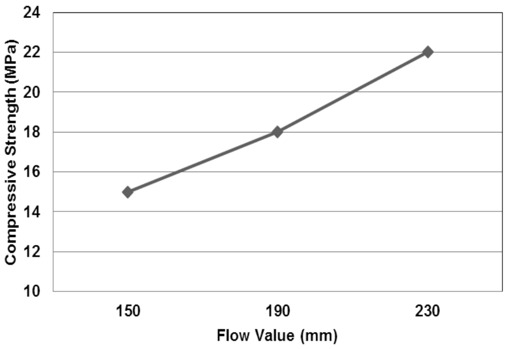

Fig 5: Relationship Between Compressive Strength and Flow value (Chindaprasirt *et al.*, 2007)

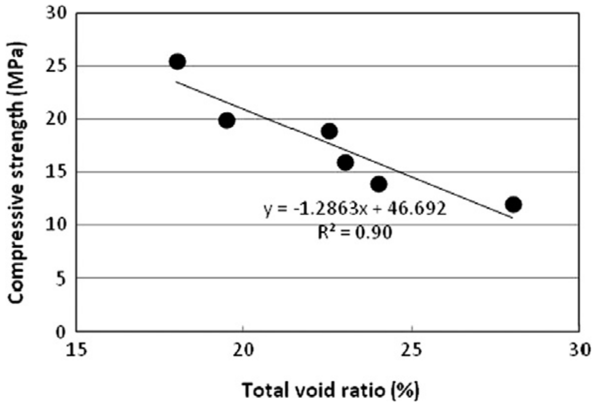

Fig 6: Relationship Between compressive strength and void ratio (Bhutta *et al.*, 2011)

Void and w/c Ratio

Most of the porous concretes are observed to have total void ratio ranging from 18 to 28% regardless of aggregate size. The acceptable ratio ranges between 15 and 25% (Bhutta *et al*., 2011). Aggregate gradation exhibits a significant effect on the total void ratio. With decreasing aggregate size, the total void ratio decreases (Bhutta *et al*., 2011).In order to obtain the required continuous void and sufficient strength, the most important condition is to keep the continuity of cement paste with coarse aggregate embedded so that continuous void is maintained. This can be achieved with the use of cement paste with relatively low w/c and sufficiently high workability. In general, water cement ratio is used within range of 0.20-0.36 for porous concrete (Chindaprasirt *et al.*, 2007, Ozbek *et al.*, 2013, Bhutta *et al.*, 2011, Sumanasooriya *et al*., 2011 and Sumanasooriya *et al*., 2011).

Relationship between Void ratio and Compressive strength

The relationship between total void ratio and compressive strength of porous concrete is represented in Fig. 4. The compressive strength of all porous concretes was increased with a decrease in void ratio regardless of types of porous concrete and aggregate size (Bhutta *et al*., 2011).The relationship between compressive strength and void ratio of a porous brittle material suggested by Ryshkewitch and Duckworth is used for porous concrete (Chindaprasirt *et al.*, 2007).

$\sigma = \sigma_0 e^{-bV}$...............................Eq. (4)

Where σ is compressive strength (MPa), σ_0 is compressive strength at zero void (MPa), V is void (%) and b is experimental constant.

CONCLUSION

This study shows that compressive strength of porous concrete is in the range of 12-18 MPa which can be increased up to about 22 MPa with use of admixture proper mix design and other factors. This value shows its potentiality to be used in Civil Engineering structures. However, as a new product, the importance of strength for pervious concrete design is still under research. Therefore, its primary applications have been limited to walkways, bike lanes and parking lots. In these applications the pervious concrete is usually subjected to relatively light and low frequency loading. Although pervious concrete has been used for some low-traffic roads and shoulders, it is not widely used as a street paving material (Goede, 2009). With impact strength of up to 90 MPa and abrasion loss of less than 2% in loaded wheel abrasion test and surface abrasion test porous concrete poses its effectiveness to be used in the pavement surfaces. Permeability of porous concrete which can be up to 1.22 cm/sec is good enough to drain away the rain water from its surface.

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STRENGTH BEHAVIOUR AND DURABILITY OF SILICA POWDER BLENDED CONCRETE IN ACIDIC ENVIRONMENT

Suborno Debnath[1*], Sajib Khan[1], Bipul Chandra Mondal [1] & Md.Saiful Islam[1]

[1] *Department of Civil Engineering, CUET, Chittagong-4349, Bangladesh.*
**Corresponding Author <subornodebnath@gmail.com>*

ABSTRACT

Durability in terms of strength and dimensional stability is the prime demand to concrete in acidic environment such as sewer pipe lines, industrial waste water drainage system etc. Hydrogen sulphide gas which may be evolved from septic sewage in sewer or sludge digestion tank through by itself is not harmful, may promote the formation of sulphuric acid which can attack the concrete surface. Moreover sulphuric acid is used in industries in their production process thereby affecting the sewerage system by the effluent of the industries. So, it is necessary to construct durable structures in such environment. "Silica powder" is an important pozzolana which improves both strength and durability of concrete by retarding the acid intrusion and chemical reaction with free lime of cement. In this study, cement was partially replaced with silica powder by 0% (control mix), 10%, 20%, 30% and 40% by weight of OPC. 100 mm cubic specimens were cast and pre-cured in normal water for 28 days. Then the specimens were placed in four different types of environments i.e. normal water, 1% H_2SO_4, 2% H_2SO_4 and 3% H_2SO_4 solution. The different tests such as Compressive Strength, Splitting Tensile Strength, and Dimensional Stability were performed on test specimens at the end of 3, 7, 28, 90 & 120 days exposure. From the test results, it can be concluded that silica powder blended concrete shows better durable performance in sulphuric acid environment.

Keywords: Silica fume, Compressive strength, Splitting tensile strength, Durability, Acidic environment.

INTRODUCTION

Concrete is the most widely used construction material for civil engineering constructions including industrial outlets and sewer structures. These structures are supposed to carry acidic water as industrial effluent. These acid causes serious damage to structural concrete (Kejin *et al*.; 2006). Several researches were carried out to overcome this deterioration. Fiber glass reinforced lining, high performance coating, special mortars and high proportions of polymer modified binder can be more acid resistant but they are costly for most practical application (Saricimen *et al*.;2003, Vipulanandan *et al*.;2002, Vincke 2002). Therefore the use of low cost pozzolanic material such as silica fume is suggested. It is known that porosity of the cement paste is the important parameter for durability and other mechanical properties (Rahmani *et al*.; 2005). These porosity can be reduced by using ultrafine supplementary material such as silica fume with cement. It reacts with the free lime in the concrete matrix and consequently improve its performance (Safwan *et al*.; 2013). Concrete is generally well resistant to chemical attack provided an appropriate mix is used and the concrete is properly compacted. Concrete containing Portland cement, being highly alkaline is not resistant to attack by strong acids or compounds which may convert to acid. Consequently, unless protected, concrete should not be used when this form of attack may occur. Use of blended cement which include ground granulated blast furnace slag, fly ash and especially silica fume is beneficial in reducing the ingress of aggressive substances. Pozzolanic action also fixes $Ca(OH)_2$ which is usually the most vulnerable product of hydration of cement so far as acid attack is concerned. However, the performance of concrete depends more on its quality than that of the type of cement used. Silica fume is a by-product resulting from the reduction of high purity quartz with coal or coke and wood chips in an electric arc furnace during the production of silicon metal or silicon alloys (Vikas *et al*, ;2012). The addition of silica fume to concrete is effective for increasing the compressive strength, decreasing drying shrinkage, increasing the abrasion resistance, increasing the bond strength with the reinforcing steel and decreasing permeability. However, silica fume causes workability loss which is a barrier against proper utilization of silica fume concrete (Yunsheng. & Chung.; 1999). It contains SiO_2 around 85-97% and is profoundly used in Canada, United States, and Japan, rarely in India but it is not available in our country. Silica powder which is a by-product contains around 40% SiO_2 and other cementitious compounds. It is produced from rice husk in the process of thermal treatment which increase the relative amount of silicon oxide by reduction of carbonaceous materials that present in the samples, as well as to burn out other undesirable components detected by chemical analysis. (Patil *et al.*; 2014) Very few literature are available regarding the use of silica powder in concrete which needs detail investigation for its suitability.

Objectives of the study:

The main objective of the study is to observe the strength and durability aspect of silica powder blended concrete in acidic environment. The experimental programme was designed to understand the mechanism of acidic attack, dimensional stability, strength characteristics of concrete and also the resisting effect of silica powder incorporation as blending material with cement in making concrete to acidic environment.

EXPERIMENTAL PROGRAM

Materials

Silica Powder:

Silica powder was collected from KALLANI TRADERS (Shop # 3/1, Armenian Street, Armanitola Dhaka-1100) which is imported from India. Table 1 shows the constituents of silica powder.

Table 1. Constituents of silica powder

Serial no.	Constituents	Silica powder(In %)
1.	Silica (SiO_2)	38.80
2.	Lime (CaO)	2.52
3.	Alumina (Al_2O_3)	0.01
4.	Magnesium (MgO)	1.48
5.	Feric oxide (Fe_2O_3)	0.565
6.	IR	55.43

Cement:

Ordinary Portland Cement (OPC) Type-I is the most common type of cement used around the world. It is a basic ingredient of concrete, mortar, and is used as binding material. PREMIER BRAND OPC CEMENT produced by grinding 95% clinker and 5% Gypsum was used in this study. It conforms with the Bangladesh Standard BDS EN 197-1:2003 CEM-I 42.5 N, European Standard EN 197 type CEM I, and American Standard ASTM C 150 Type-I mark.

Table-2 shows the physical properties and constituents of cement used in the study.

Table 2. Physical properties and Constituents of OPC

Serial no.	Physical Properties	Test Results	Constituents	Test Result
1.	Fineness	96%	Lime(CaO)	62%
2.	Specific Gravity	3.15%	Silica(SiO_2)	22%
3.	Standard Consistency	25.40%	Alumina (Al_2O_3)	5%
4.	Initial Setting Time	65 min	Magnesium(MgO)	1%
5.	Final Setting Time	190 min	Calcium sulphate	4%

Coarse aggregate: The stone aggregate fractions from 20 mm to 4.75 mm with FM 7.5 specific gravity 2.60 was used as coarse aggregate.

Fine aggeregate: Sylhet sand of FM 2.4 and size fractions from 4.75 mm to 150 micron was used as fine aggregate.

Water: Potable water was used for mixing and curing of concrete specimens.

VARIABLE STUDIED:

(a) Cement replacement levels: 10%, 20%, 30% and 40% by weight with Silica Powder.
(b) Acidic Environments : 1%, 2%, and 3% H_2SO_4 solution.
(c) Exposure Periods : 3, 7, 28, 90 and 120 days.
(d) Nature / type of tests : Visual inspection, volume change, compressive strength and tensile strength.

A total of 480 No’s 100*100*100 mm cubical specimens were cast from M35 grade concrete using four different replacement levels of cements.0% replacement level, control specimens were also cast for comparison. The specimens were demoulded after 24 hours and immersed in normal water for 28 days normal curing. After procuring, the specimens were exposed to different acidic environments as per test program. After specific periods, the specimen were taken out and subjected to various tests as stated above. Test data from three similar specimen was considered to get average result at each test point. The test results are critically analysed, interpreted and presented in tabular and graphical form so as to arrive at some useful conclusions.

RESULT AND DISCUSSION

Visual Examination:

Visual examination was carried out on specimen immersed in different acidic environments. It is seen that control specimen show no colour change but the specimens in acidic environment show the change in colour from original grey to yellowish due to the acidic attack on hydrated cement product (Ref Fig.1).

Fig.1 Concrete specimen exposed to 3%, 2%, 1% H_2SO_4 and plain water (left to right).

Volume Change:

The results of volume change for concrete specimen in acidic environment are presented in table 3. The specimens in normal water show marginal increase in volume whereas the specimen in acidic environment show a decrease in volume. In 3% H_2SO_4 environment, the normal concrete (0% replacement) losses its volume around 11% whereas the 40% replaced silica powder concrete losses its volume only by 2%. Thus silica powder concrete shows better dimensional stability. The decrease in volume may be associated with the surface erosion / crumbling as evidenced from visual inspection report. (Ref Fig 2).

Fig.2 Effect of 3% H_2SO_4 on 40%, 30%, 20%, 10%, 0% silica powder concrete (left to right)

Strength Test Result:

Table 3 and Fig 3-7 shows the compressive and tensile strength result of concrete specimen exposed to acidic environment for various exposer periods. The strength of the concrete specimen decrease with the increase of the cement replacement level with silica powder in plain water environment. But in acidic environment, control specimen suffers maximum strength loss as compared to silica powder blended concrete. For example, in 3% H_2SO_4 **,** the normal concrete (0% replacement) losses its strength up to 24.83% whereas the 40% replaced silica powder concrete remain stable in strength even with marginal increase strength up to 0.704% (Ref Table 4).

Thus it can be noted that, silica powder concrete, although attain the lower strength at normal environment, offer higher resistance regarding strength deterioration in acidic environment.

Particularly at 10% replacement level the strength of blended concrete is almost same as that of control specimen. Also it shows noticeable resistance to strength deterioration in acidic environment. Thus among the limited no of replacement level, 10% may be considered as optimum for the replacement of cement with silica powder in making concrete for acidic environment.

Table 3. Volume change (%), Compressive & Tensile strength (MPa) at various ages

		7 DAYS			28 DAYS			90 DAYS			120 DAYS	
Environments	Silica Powder (%)	Volume Change (%)	Com. Strength (MPa)	Tensile Strength (MPa)	Volume Change (%)	Com. Strength (MPa)	Tensile Strength (MPa)	Volume Change (%)	Com. Strength (MPa)	Tensile Strength (MPa)	Com. Strength (MPa)	Tensile Strength (MPa)
N. W	0	1.89	38.21	3.84	2.15	40.05	4.42	2.94	41.97	4.99	42.25	5.28
	10	1.32	33.30	4.02	1.86	34.10	4.42	1.99	32.74	4.99	33.3	4.99
	20	1.06	31.28	3.84	1.38	31.96	3.84	1.49	29.26	4.02	30.12	4.42
	30	1.03	25.22	3.56	1.27	27.18	3.27	1.21	27.53	3.84	28.68	4.42
	40	0.8	22.04	2.69	0.85	22.98	3.27	0.95	23.19	3.84	23.48	3.84
1% H_2SO_4	0	-0.55	36.77	3.27	-2.74	37.01	4.42	-4.37	33.02	4.42	35.34	3.84
	10	-0.87	36.48	3.27	-2.13	35.28	3.27	-2.63	30.70	4.42	32.63	3.86
	20	0.13	28.39	3.84	-1.59	28.50	4.42	-2.16	28.68	4.99	30.54	3.42
	30	1.27	16.26	2.69	0.87	20.08	2.69	-1.69	21.75	2.69	27.41	3.30
	40	3.08	24.63	3.26	1.58	24.80	2.69	-1.25	24.93	2.69	23.87	2.69
2% H_2SO_4	0	0.57	36.78	3.27	-3.58	38.56	3.27	-6.99	39.66	4.42	41.36	3.27
	10	1.65	31.86	2.69	-1.32	27.42	3.27	-4.56	26.94	4.42	34.27	3.24
	20	2.08	30.41	3.27	-0.97	26.45	4.42	-3.45	22.62	4.42	29.83	3.48
	30	3.01	22.61	4.42	1.25	22.80	4.42	-2.39	22.91	3.26	25.13	3.26
	40	4.12	17.99	4.42	2.48	18.08	3.26	-1.98	18.86	2.69	21.94	2.69
3% H_2SO_4	0	-1.11	36.77	4.13	-5.63	30.21	3.84	-10.44	24.64	3.27	22.71	2.69
	10	-0.93	30.99	4.42	-3.19	28.42	3.84	-6.77	23.77	3.84	24.14	3.27
	20	-0.17	22.90	3.84	-1.87	21.22	3.55	-3.29	20.88	3.27	22.23	3.27
	30	0.46	21.75	3.84	-0.25	20.63	3.27	-2.49	18.57	3.07	21.75	2.85
	40	1.93	20.02	2.69	0.83	20.45	2.69	-1.39	19.44	2.69	20.59	2.69

Table 4. Increase or decrease rate of compressive strength compared to 28 days strength in 3% H_2SO_4.

DAYS	0% REPLACEMENT		40% REPLACEMENT	
	Compressive Strength	Increase or Decrease Rate %	Compressive Strength	Increase or Decrease Rate %
7	36.77	21.714	20.02	-2.10
28	30.21	------	20.45	------
90	24.64	-18.44	19.44	-4.93
120	22.71	-24.83	20.594	+0.704

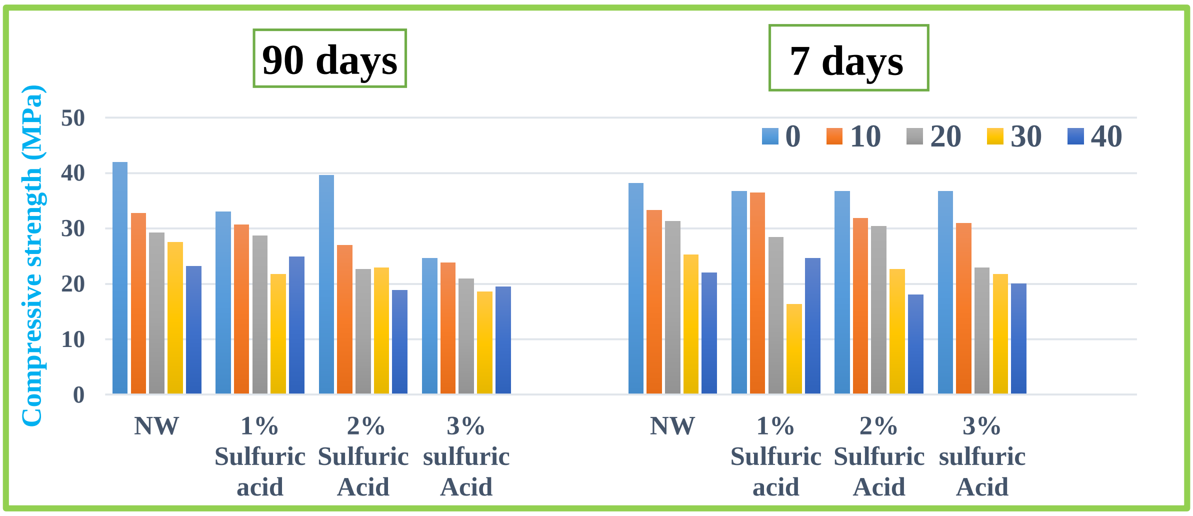

Fig.3. Compressive strength of silica powder blended concrete (age at 90 & 7 days)

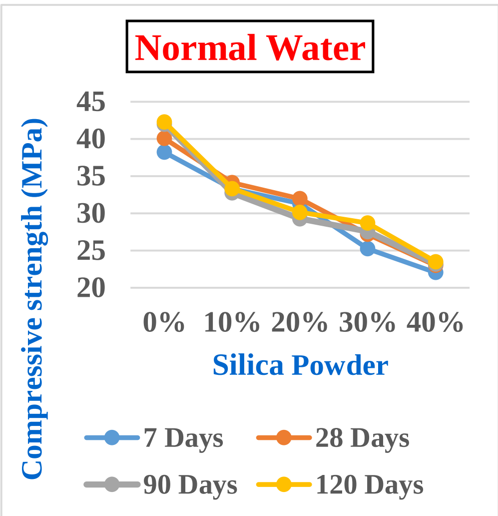

Fig.4. Compressive strength of specimens in normal warter

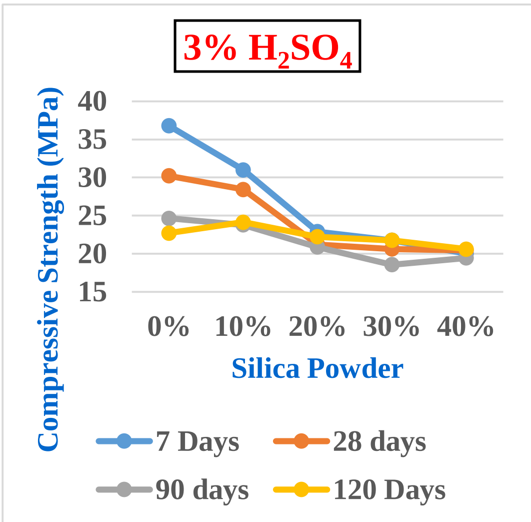

Fig.5. Compressive strength of specimens in 3% H_2SO_4

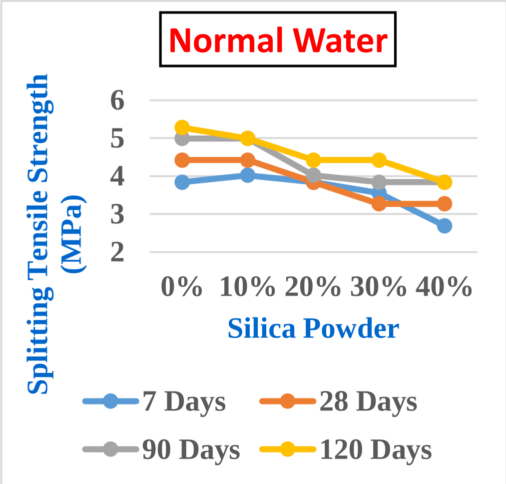

Fig.6. Splitting tensile strength of specimens in normal water

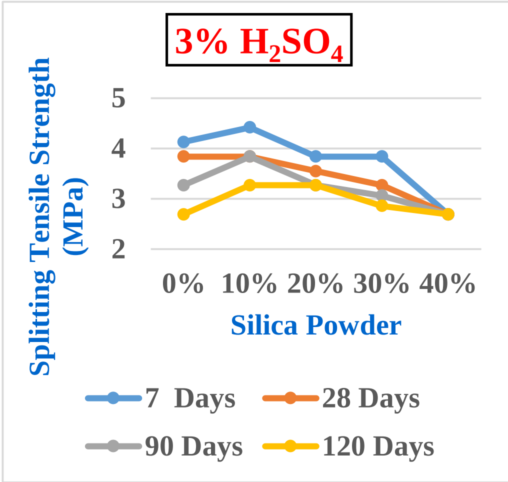

Fig.7. Splitting tensile strength of specimens in 3% H_2SO_4

CONCLUSION AND RECOMMENDATION

Based on the limited number of variable studied on the durability of silica powder blended concrete in acidic environment, the following conclusion can be drawn:

- Silica powder may be used as partial replacement of cement in making structural concrete in acidic environment. However the replacement level depends on durability requirements.
- Incorporation of silica powder leads to the reduction of concrete strength in plane water environment but in acidic environment, silica powder concrete offers significant resistance to strength deterioration.
- In 3% H_2SO_4 environment, the normal concrete (0% replacement) losses its strength up to 24% whereas the 40% replaced silica powder concrete remain stable in strength even with marginal increase strength up to 1%.
- 30% and 40% silica powder concrete shows least strength deterioration in acidic environment although the 28 days plain water cured strength of those concrete are markedly lesser than control concrete.
- In 3% H_2SO_4 environment, the normal concrete (0% replacement) losses its volume around 11% whereas the 40% replaced silica powder concrete losses its volume only by 2%. Thus silica powder concrete shows better dimensional stability.
- Considering dimensional stability, strength gaining, deterioration, 10% cement replacement with silica powder in concrete is found optimum.
- Similar investigation can be carried out taking more as well as smaller increment of replacement level and also over longer periods so as to get more specific information about the use of silica powder blended concrete in acidic environment.

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AN INVESTIGATION ON COMPRESSIVE STRENGTH OF CONCRETE WITH RECYCLED AGGREGATES

M.A. Mohiuddin[1*], S. Das[2], K.M. Islam[1], M.A. Hasan[1]

[1] *Dept. of Civil Engineering, Southern University Bangladesh, Chittagong, Bangladesh.*
[2] *Dept. of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.*
**Corresponding Author, <engrareef@gmail.com>*

ABSTRACT

A significant amount of natural resources can be saved if the demolished concrete is recycled for new construction. In addition to the saving of natural resources, recycling of demolished concrete will also provide other benefits, such as creation of additional business opportunities, saving cost of disposal, saving money for local government and other purchasers etc. Recycled aggregates are comprised of crushed, graded inorganic particles processed from the materials that have been used in the constructions and demolition debris. The objective of this work is to achieve the target strength of concrete made by recycled aggregates used fully or partially with the natural fresh stones. Local sand of different fineness modulus (FM value 1.1, 1.2, 1.25, 1.30, 1.40, 1.50) and Sylhet sand (FM value 2.90) are used as fine aggregates. In this study, ACI 211-91was used for mix design & conducted compressive strength test for 15 Cylinders (also 15 compositions) with 15 different mix ratios.

From the results obtained, it can be concluded that recycled aggregate used for low strength concrete is able to achieve target strength and local sand of FM 1.25 is more suitable than the other samples.

Keywords: Recycled Aggregate, Concrete, Local Sand, Compressive Strength, Fineness Modulus.

INTRODUCTION:

For a variety of reasons, reuse of construction & demolished (C & D) waste by the construction industry is becoming day by day. In addition to environmental protection, conservation of natural aggregates resources, shortage of waste disposal land & increasing cost of waste treatment prior to disposal are the principal factors responsible for the growing interest in recycling C&D waste (Metha, 1999, Topcu et al; 2004, Terro, 2005).

The concrete rubble has the largest proportion and hence its recycling is most important among the inert C&D waste. Many laboratory and field test studies have shown that the size fraction of the concrete rubble corresponding to coarse aggregate and the recycled concrete aggregate shows that the later would give at least two third of the compressive strength and the elastic modulus of the natural aggregate (Metha, et al 1993; Poon et al;2004).

Due to the modernization of the civil society through industrialisation present world is changing rapidly. One of the representative changes is the rapid penetration into the use of construction works to contain the waxing inhabitants. To get optimum benefit, many buildings and other structures that are old and deteriorated have been demolishing in recent years. Consequently large amount of concrete wastes are generated, which created deposition problems. Beside to make new massive structures on demand large amount of concrete is prepared from natural aggregates. If the use of natural aggregates can be replaced by demolished concrete then we can hope for better environment for our living through effective construction. This study has been intended to know whether recycled aggregate possesses that much ability to take the position of new concrete in construction.

WHY CONCRETE RECYCLING IS NECESSARY FOR BANGLADESH?

In Bangladesh, the volume of demolished concrete is increasing due to the deterioration of concrete structures as well as the replacement of many low-rise buildings by relatively high-rise buildings due to the booming of real estate business. Disposal of the demolished concrete is becoming a great concern to the developers of the buildings. If the demolished concrete is used for new construction, the disposal problem will be solved, the demand for new aggregates will be reduced and finally consumption of the natural resources for making aggregate will be reduced. In some project sites, it was also found that a portion of the demolished concrete is used as aggregate (after breaking into aggregate) in foundation works without any research on the recycled aggregates. In most of the old buildings, brick chips were used as coarse aggregate of concrete in Bangladesh. Studies related to the recycling of demolished concrete are generally found for stone chips made concrete [Alan 1977 and Gomez-Soberon et al 2002]. Therefore, investigations on recycling of brick made concrete are demolished necessary.

ADVANTAGES OF RECYCLING OF DEMOLISHED CONCRETE:

The advantages associated with the recycling of demolished concrete are summarized below:

- Saving of natural resources.
- Creation of additional business opportunities.
- Saving cost of disposal of demolished concrete.
- Saving money for local governments and other purchaser.
- Saving energy when recycling is done at site.
- Helping local government to meet their goal of reducing disposal. Minimize hazards to collect coarse aggregate from different natural resources.

EXPERIMENTAL PROGRAM:

Local sand of different fineness modulus (FM value 1.1, 1.2, 1.25, 1.30, 1.40, 1.50) and Sylhet sand (FM value 2.90) are used as fine aggregates. For our investigation, demolished concrete blocks were collected from demolished Flyover sites and broken into pieces as aggregates. Before making concrete, the aggregates were investigated for absorption capacity, specific gravity, unit weight & moisture content. Standard grading of the aggregates were controlled as per ASTM C33. Cylinder concrete specimens of diameter 150 mm and height 300 mm were made and tested for compressive strength. The workability of concrete was also measured by slump test. In this study, ACI 211-91was used for mix design & conducted compressive strength test for 15 Cylinders (also 15 compositions) with 15 different mix ratios.

RESULT:

Table-1: Variation of Water – Cement ratio with fineness modulus of Fine aggregate

FM of Fine Aggregate	Aggregate Type	Water Cement ratio	Strength (psi)
1.10	100% Recycled	0.46	2241.12
1.20	100% Recycled	0.44	2281.14
1.25	100% Recycled	0.44	2761.38
1.30	100% Recycled	0.46	2081.04
1.40	100% Recycled	0.45	2505.25

1.50	100% Recycled	0.44	2161.08

Table-2: Variation of Water – Cement ratio with Composition of Coarse aggregate

FM of Fine Aggregate	Aggregate Type	Water Cement ratio	Strength (psi)
1.25	100% Fresh	0.44	3001.50
1.25	75% Fresh+25% Recycled	0.43	3337.67
1.25	50% Fresh+50% Recycled	0.44	3065.53
1.25	25% Fresh+75% Recycled	0.44	2321.16
1.25	100% Recycled	0.44	2761.38

Table-3: Variation of Water – Cement ratio with Composition of Coarse aggregate

Type of Fine Aggregate	Aggregate Type	Water Cement ratio	Strength (psi)
Sylhet Sand	100% Fresh	0.45	3889.94
Sylhet Sand	75% Fresh+25% Recycled	0.45	3297.65
Sylhet Sand	50% Fresh+50% Recycled	0.45	3361.68
Sylhet Sand	25% Fresh+75% Recycled	0.45	3121.56
Sylhet Sand	100% Recycled	0.46	1920.96

Compressive strength for different FM values of F.A. with recycled aggregate after 3, 7 & 28 days:

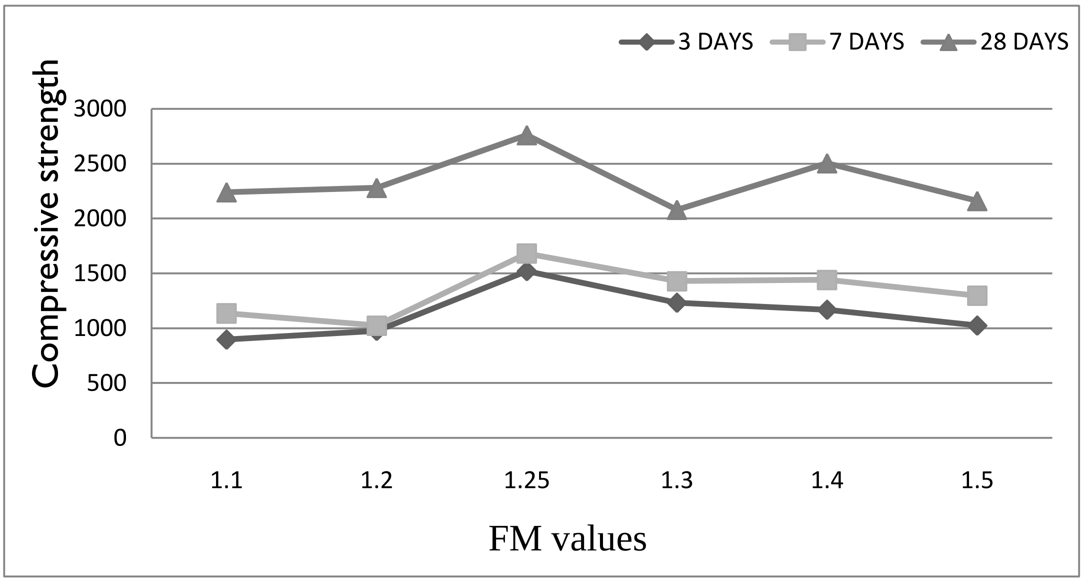

Fig-1: Comparison of Compressive strength with Curing period.

Compressive strength for fm 1.25 of FM with different types of coarse aggregate after 3, 7 & 28 days:

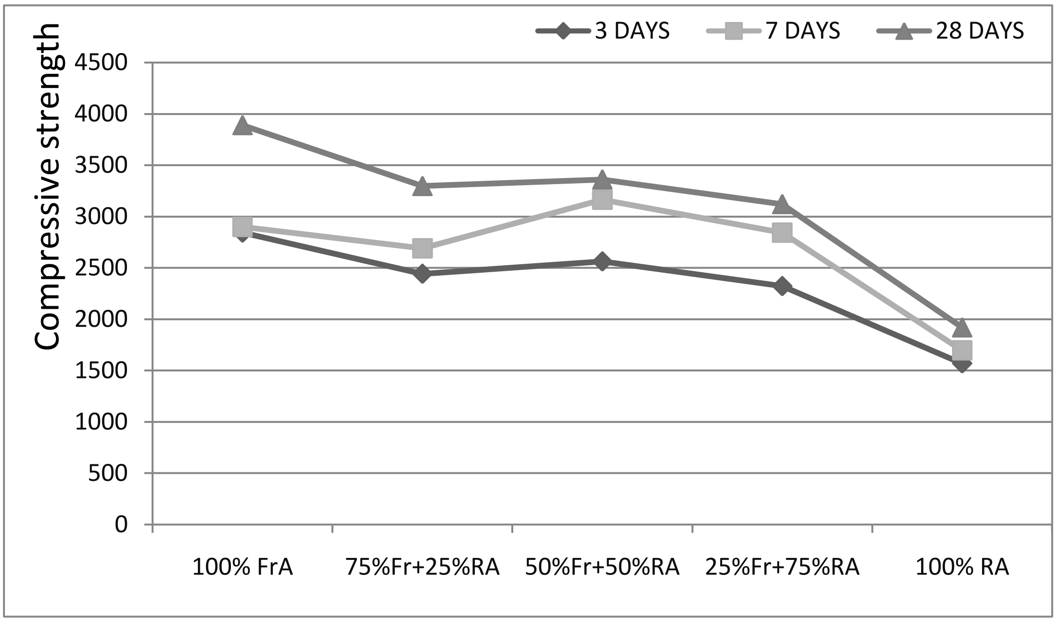

Fig-2: Comparison of Compressive strength with Curing period.

Compressive strength for sylhet sand with different types of coarse aggregate after 3, 7 & 28 days :

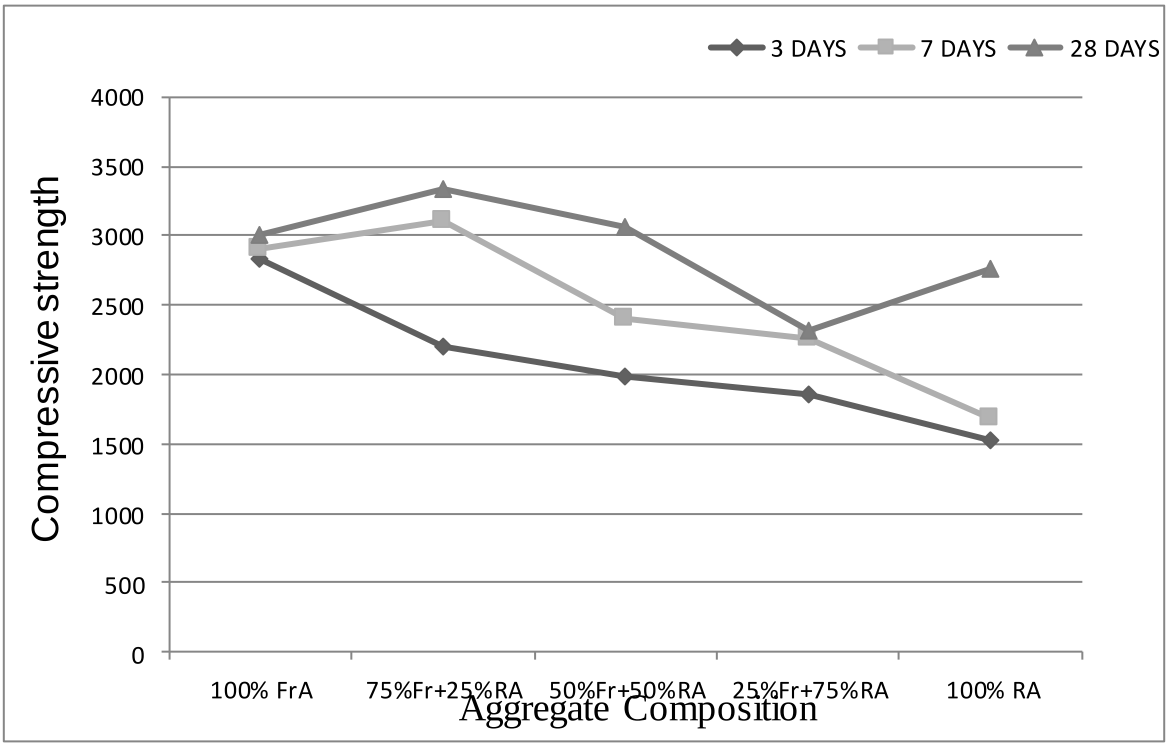

Fig-3: Comparison of Compressive strength with Curing period.

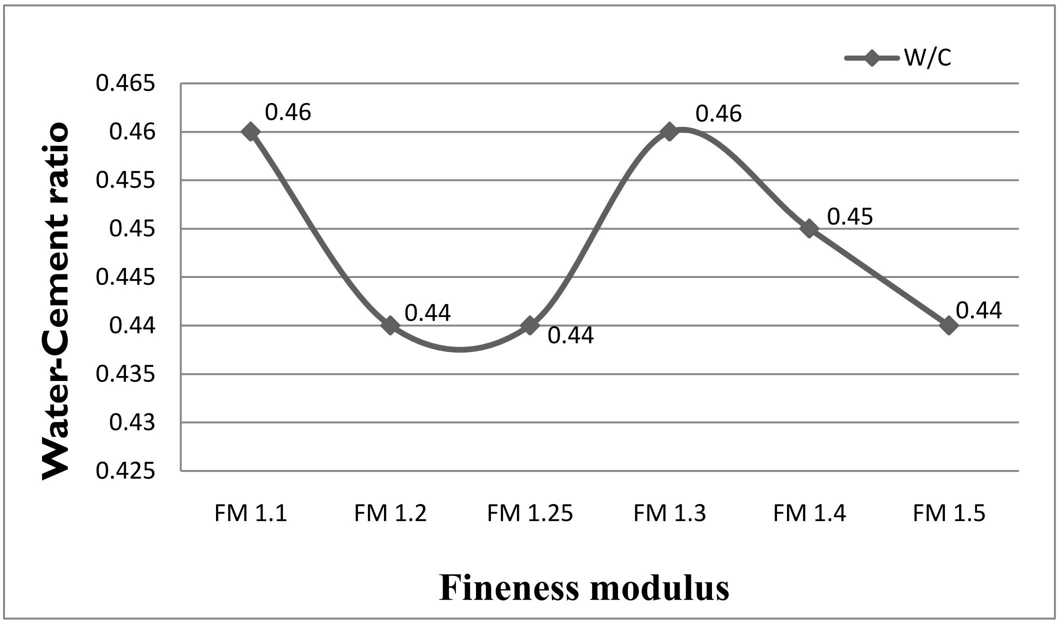

Fig-4: Variation of W/C ratio with the variation of Fineness modulus of Fine Aggregate.

Table-4: Concrete Compressive Strength for 3000 psi

FM of Fine Aggregate	Coarse Aggregate Type	Mix ratio	Strength (psi)		
			3 Days	7 Days	28 Days

1.10	100% Recycled	1:1.94:2.88:0.46	896.45	1136.57	2241.12
1.20	100% Recycled	1:1.97:2.85:0.44	976.49	1024.51	2281.14
1.30	100% Recycled	1:2:2.8:0.46	1232.62	1428.72	2081.04
1.40	100% Recycled	1:2.1:2.77:0.45	1168.58	1440.72	2505.25
1.50	100% Recycled	1:2.1:2.73:0.44	1024.51	1296.65	2161.08
1.25	100% Fresh	1:1.69:3.11:0.44	2833.42	2897.49	3001.50
1.25	75% Fresh+25% Recycled	1:1.76:3:0.43	2193.10	3105.55	3337.67
1.25	50% Fresh+50% Recycled	1:1.9:2.91:0.44	1984.99	2401.20	3065.53
1.25	25% Fresh+75% Recycled	1:1.83:2.97:0.44	1856.93	2258.73	2321.16
1.25	100% Recycled	1:1.97:2.84:0.44	1520.76	1680.84	2761.38
Sylhet Sand	100% Fresh	1:2.39:2.42:0.45	2841.42	2897.49	3889.94
Sylhet Sand	75% Fresh+25% Recycled	1:2.44:2.37:0.45	2441.22	2689.34	3297.65
Sylhet Sand	50% Fresh+50% Recycled	1:2.50:2.32:0.45	2561.28	3161.58	3361.68
Sylhet Sand	25% Fresh+75% Recycled	1:2.55:2.27:0.45	2321.16	2841.42	3121.56
Sylhet Sand	100% Recycled	1:2.60:2.22:0.46	1568.78	1696.85	1920.96

DISCUSSION & CONCLUSION:

The test results show that the local sand of FM 1.25 with coarse aggregate compositions [100% Fresh, (75% Fresh + 25% Recycled) & (50% Fresh + 50% Recycled)] gives the compressive strength higher than the design strength 3000 psi. Also, all compositions consisting of sylhet sand with coarse aggregate in various proportions [100% Fresh, (75% Fresh + 25% Recycled), (50% Fresh + 50% Recycled) & (25% Fresh + 75% Recycled)] shows higher compressive strength than the design strength of 3000 psi. Using Local sand (FM 1.25) is more convenient than Sylhet sand, because of its availability. From the results obtained, it can be concluded that recycled aggregate used for low strength concrete is able to achieve target strength and local sand of FM 1.25 is more suitable than the other samples.

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COASTAL COMMUNITIES COULD SURVIVE IN CYCLONE: A CASE STUDY ON WATER GYPSY (BEDE)

Irteja Hasan[1]*, Ibrahim Khalil[1], Ali Adnan[1] & S. Hossain[2]

[1] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dhumki, Patuakhali-8602, Bangladesh,*
[2] *Institute of Water Modelling, New DOHS, Mohakhali, Dhaka-1206, Bangladesh*
[*] *Corresponding author, <irteja07@gmail.com>*

ABSTRACT

Water gypsy usually known as Bede or Bedey is a nomadic racial community of Bangladesh ancestrally lives, travels and earns their livings on the river. As they mainly resided in the boat and embankment of the river and also waterways is the main communication system therefore they get easily affected by devastating cyclone. The interest of this paper are to find out the perception and prediction indicators used by the Bede people during cyclones; examine their survival strategies at the face and pace of cyclones; assess the problems they frequently face for being a marginalized group of the society; and highlight important findings that can be used by the disaster management programmers/ planners of NGOs and GO line agencies. The study mostly relied upon qualitative data and information with the close accordance of objectives. The key finding of the study includes survival strategies and relies upon cumulative experience of the earlier generations are the structural adjustment to reduce cyclone damage, specific forms of housing; and the prediction of cyclones using the state and level of the river-water, wind direction, weather, behaviour of some living organisms; the problems they mainly face are poor house structure, nonexistence of proper education, don't get informed cyclone warning signals just in time, even don't know the exact location of cyclone shelter, absence of pure drinking water, don't have the right to vote, lack of support from GO's and NGO's and they don't get space in policy making decisions. One of the objectives of this paper is to make suggestions to increase the effectiveness of the counter measures in the future, incorporating the indigenous knowledge of cyclone prediction and disaster management.

Keywords: Bangladesh, Cyclone, Coastal Area, Water Gypsy

INTRODUCTION:

Dashmina, coastal belt of Bangladesh is prone to natural Disasters including cyclone, Flood due to storm surge, Salinity intrusion, and River bank erosion. Severe cyclones and storm surges are quite common in the study area (Wisner et al. 2004; Ali 1999; Paul 2009a). On average, annually 2/3 cyclone strikes Bangladesh each year (Mooley 1980; Haque 1997; Paul2009a, b). The coastal water gypsy people have made use of several powerful practices to lessen the negative impacts of cyclones. Apart from modern cyclone forecasting, Bede people can understand forthcoming danger by looking at natural signs (Gregg et al. 2006). Coastal *water gypsy* of Dashmina, Patuakhali can predict impending cyclones by using age-old indigenous knowledge gained from nature and their families through their experiences of frequent cyclones. Such knowledge varies significantly from person to person and to survive from the cyclone, they face some common problem.

This technical paper provides of best practices and available tools for surviving cyclone and also highlights the best practices and available tools for identifies gaps in, as well as recommends possible actions to enhance the appropriate application of disaster risk reduction for cyclone. The objectives of the study as follows

- To find out the perception and prediction indicators used by the Bede people during cyclones;

- To examine their survival strategies at the face and pace of cyclones;
- To assess the problems they frequently face for being a marginalized group of the society

SELECTION OF THE STUDY AREA:

For the selection of the study area authors considered three main priorities: (a) Indigenous marginalized communities (b) coastal region; and (c) survival capacity in terms of cyclone. Keeping these in context, on the basis of secondary information, *Water Gypsy* of Dasshmina upazilla under the Patuakhali district was selected for intensive in-depth investigation. Besides this selected coastal area was having the experience of cyclones repeatedly for long time. Almost every cyclone that passes Bangladesh damages the selected area. Among the indigenous marginalized communities of Bangladesh *Water Gypsy* live in the coastal region with a considerable number and are also dependent on the sea for their living, and this made them more experienced in reading sea behavior before and during cyclones.

METHOD OF DATA COLLECTION:

The study mostly carried out based on qualitative data and information. Data have been collected from both primary and secondary sources, in order to accomplish the objectives of the study. Primary data have been collected through field visits which are customarily qualitative. Secondary data are obtained from reports, journals, research papers, and book. Material on pertinent concerns is collected from websites available on the internet. Relevant statistics on the concerned issue has been collected through in-depth, key informant interviews and group discussions with the local people were also carried out. Primary data and information collected through questionnaire have been summarized and analyzed for the fruitful completion of the study.

RESULTS AND DISCUSSIONS:

Perception and Prediction Indicators of Cyclone:

Cyclone is very common and frequent natural disaster among all disaster. They also cause maximum damage than any other ones. So, the people in this area are more conscious about cyclone and they adopt some techniques by using natural indicators to predict cyclone. Those indicators are given in **Table-1**

Table-1: Cyclone Prediction by using Indigenous Indicators in the study area

Predicting Indicators	Respondents (N)	%
Increase of water in the river, while cyclone move towards the coast	26	20
Drizzling and gloomy sky and abnormal wind circulation	20	15.39
Strong wind circulation from south or south-east	18	13.85
Huge roar of the sea/ river	23	17.70
Restless and continuous barking of house pets like dogs, cats etc.	10	7.70

Birds fly without destination	8	6.16
Ants climb towards the roof of house	8	6.16
Sea birds, pigeons move towards the inland	6	4.62
Abnormal behavior (jumping) of fish in the river	7	5.32
Gigantic waves of water in the sea	4	3.09
Total	N=130	100

Source: Field survey and author's calculation, 2014

Cyclone Surviving Strategies:

The research revealed that, generally, the water gypsi of Dashmina have a variety of Indigenous Knowledge, beliefs and practices and they use these for the purpose of surviving cyclone. Those indigenous knowledge and beliefs of water gypsi in Dashmina is proved as an important basis for facing the even greater challenges of natural disasaters like cyclone. Those surviving strategies are given in **Figure-1**

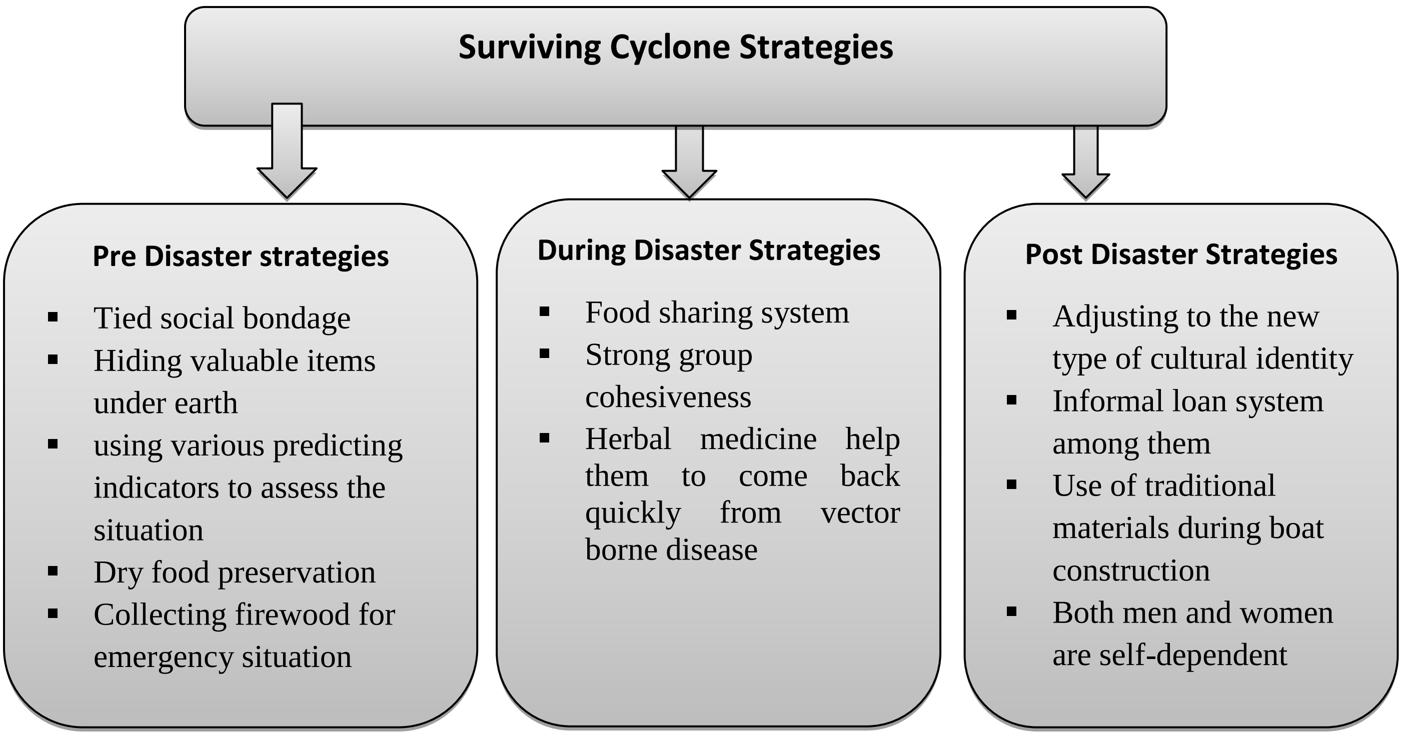

Fig 1: water gypsies surviving cyclone strategies

PROBLEMS THEY FREQUENTLY FACE:

Poor house structure:

The semi tubular shaped tents, covered in black oilcloth or plastic, seldom remain in one place for more than a couple of months, except, perhaps in the winter season. As they live temporally they don't put emphasis on the structure of house (tent) or its strength to cope up such storm or cyclone.

Nonexistence of proper education:

Although there is no caste-system in Bangladesh, bedeys are on the bottom rung of society and almost all are illiterate and desperately poor. Many find it difficult to integrate because they lack education and skills. About 98 percent of Bedes live below the poverty line, and about 95% of Bede children cannot attend school.

Don't get informed cyclone warning signals just in time:

As Bedyes are the negligible community in the society, respectable organizations for disaster forecasting and warning, even CPP would not inform them immediately in the times of cyclone warning phase.

Don't know the exact location of cyclone shelter:

As the cyclonic storms are unpredictable and they are frequently roaming from here to there, so in the time of emergency they become helpless. They don't even know the exact location of cyclone shelter to make a safe room for them in the time of disasters.

Absence of pure drinking water:

Many rivers, canals and streams are drying up because of human extraction of water, large-scale dumping of industrial waste and unplanned building of hundreds of dams. All these factors lead to a problem of access of pure drinking water.

Don't have the right to vote:

Existence information denotes that in 2007 when the Caretaker Government was in power, they took the initiatives to bring the Bede people under the right to vote. But unfortunately the coastal Bede people were completely deprived from this program which acts as a curse to them and they frequently run down from their basic rights.

Lack of support from GO's and NGO's:

According to Grambangla Unnayan Committee, a Dhaka-based charity, Bangladesh's bedey community could disappear within a few decades as they abandon their annual migration between land and water. Finding jobs on the mainland is tough. Most of the Bedes are not skilled at anything but being bedeys. Of course they want a better life than on the water, but they are always deprived from the support of Government as well as Non-Government organizations.

Don't get space in policy making decisions: As Bedes are not eligible for voting, political leaders have no headache for them. Consequently their thoughts, views and interests never get any space in the society. Illiteracy and living under poverty line simultaneously destitute them from the representation in the decision making level.

RECOMMENDATIONS:

On the basis of the findings of this research the following recommendations for taking structural and non-structural measures are made, which can be utilized to reduce cyclone damages:

Special Cyclone Forecasting and Warning Model:

One of the most effective countermeasures for the reduction of cyclone is the establishment of special cyclone early warning model. The cyclone warning system, which is prevailing in Bangladesh, based on signal numbers up to 10. This system is cumbersome. People had no clear idea of the meaning of the signals. The warning system on the basis of forecast should be disseminated in a language that can be easily understood by the Bede people.

Adequate Cyclone Shelters:

After the cyclone of 1971 a number of cyclone shelters were constructed in the coastal areas.

These are not sufficient in number and are not properly designed and located (although nowadays situation is improving). But many people could not use these shelters because the access roads were flooded. But the Bede people face problems in these too. For being a marginalized group they could not take shelter along with the Bengali Muslims in many cyclone centers, private cyclone resistant houses and mosques. So, considering these problems responsible organization should take prior action to establish adequate cyclone shelters as an option for the safety of Water Gypsy.

Provision of pure Drinking Water and proper Sanitation:

Similar to other infrastructures the cyclone normally causes serious contamination of the water supply and damages to the sanitation system. The Water Gypsy's of the study area traditionally use pond and river water for drinking and household activities. Therefore, after the cyclone there is a serious crisis of drinking water and an outbreak of waterborne diseases is very common.

A large number of casualties occur from the post cyclone water borne diseases. To mitigate the post cyclone sufferings and loss of lives the water and sanitation sector should be given proper care. Water purification tablet can be provided by NGOs as well as govt.

Alternate their Livelihoods:

Water Gypsys usually manage their livelihoods by snake charming which is useful for villagers seeking to remove resident reptiles from village homes, and entertaining in Haats and Bazaars, selling traditional medicines and remedies to the villagers, for whom a visiting Bede encampment is usually welcome. Diving for the river cultured pearls, especially the distinctive pink, which are a specialty of the country, is another of their demand skills. But in the recent era the ornaments they sell are no longer in demand. Orthodox medicine is preferred so there is no demand for their products. Finding jobs on the mainland is tough. Most of us are not skilled at anything but being bedeys. In this typical situation it is essential to alternate their livelihoods for protecting this historically cultured marginalized group.

Educate them in the Floating Boat:

Proper education makes a human being oneself to prepare for any infortunes condition. At least the basic concept regards hygiene, sanitation or other issues should provide these community people to lead life. Both GOs & NGOs can conduct this journey with the vulnerable people like "Food for learning" & it's a demand of time to ensure their safety. Organizations who work for disaster management should consider this issue.

Post Cyclone Relief and Rehabilitation:

Bangladesh needs a better plan to incorporate all the sufferers from cyclones under a well-managed relief and rehabilitation programme. And the higher officials should take into account two more facts in relief distribution and programming: (a) when there is lot to give through relief to the sufferers it has to be properly monitored as it was found that local NGOs were accused of taking bribes in distributing relief products; (b) the other this which need to be taken into account that all the ethnic and marginalized communities. So to keep protecting the cultural tradition of the Water Gypsy the government needs to take proper initiatives.

Proposed Model for Desseminating Effective Early Warning Signal During Cyclone:

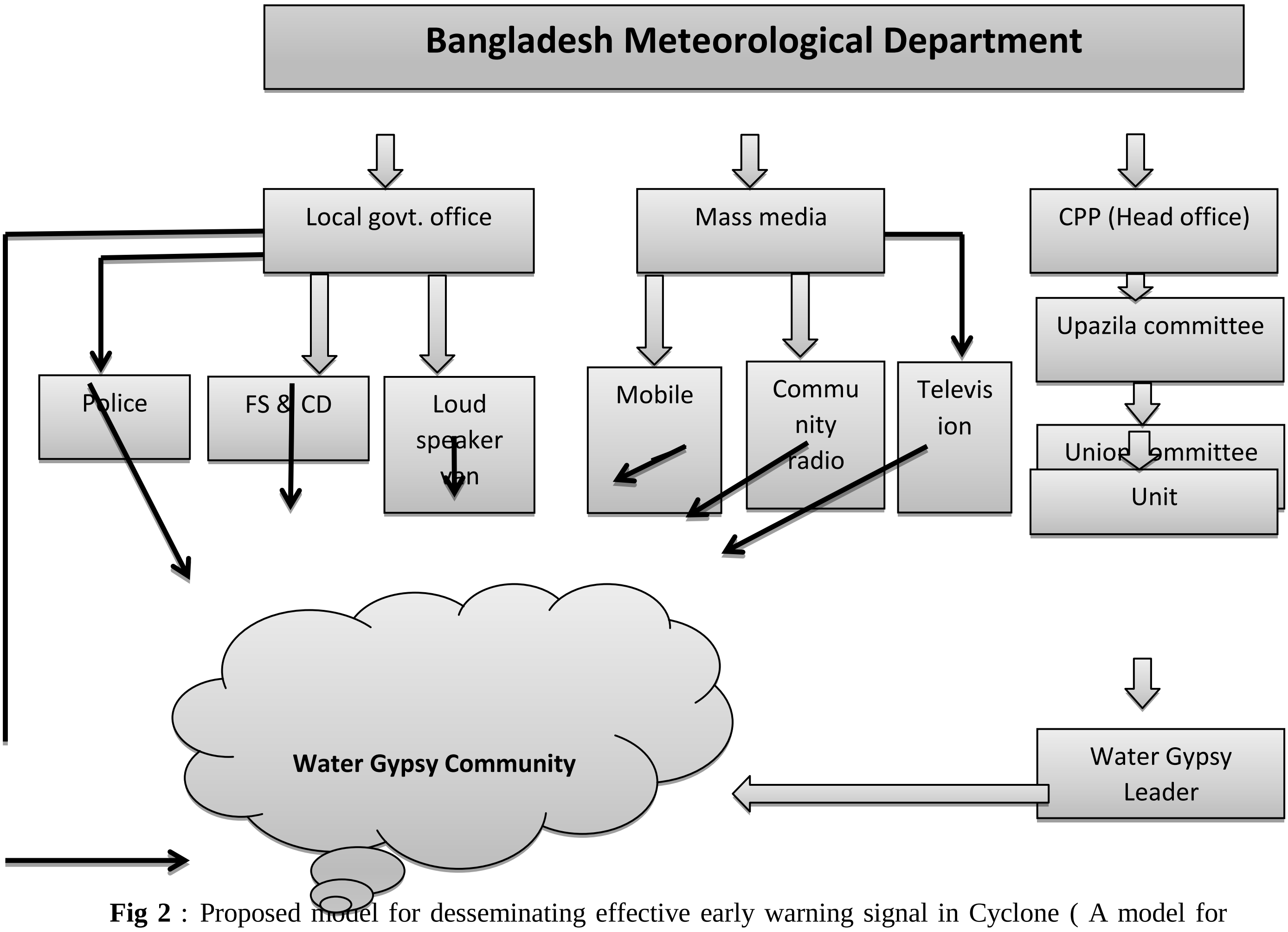

Fig 2 : Proposed model for desseminating effective early warning signal in Cyclone (A model for coastal water Gypsy)

CONCLUSION:

As the natural disasters make people vulnerable, every society perceives the natural disastrous events with its capacity to cope and peoples' interaction with their surroundings is important for their living in a particular area. Water Gypsy in the coastal region is no longer an exception here. Their social system responses in the crisis situation and the temporary adaptive processes play an important role to stand still at the pace and face of any cyclonic storms. Having close contact with the nature, Bede community of the study area have developed an indigenous perception and prediction strategy for cyclones and, there by possess effective survival strategies. Water gypsy community of the selected study area take all aspects of their communal, racial and conservational environments into account as well as a chain of risk-related dynamics in responding to cyclones. And their perceptions have terrestrial, social, fiscal, traditional and factors such as the extent of the consequences of the hazardous event, the degree of the perceived control over the consequences, the degree of personal exposure and other social costs and benefits. So, for effective disaster management planning and programming input at both local and national level, it is essential to explore the rationale of their actions.

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ID: SEM 025

REDUCTION OF ENERGY CONSUMPTION, CO_2 EMISSION & CONSTRUCTION COST OF A MODEL BUILDING

M. A. Imran[1*] & M. T. Islam[1]

[1] *Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi, Bangladesh.*
**Corresponding Author, <ashraful.ruet@yahoo.com>*

ABSTRACT

The manufacturing of building materials contributes to the huge amount of CO_2 emission, specially in the cement industry and in brick burning process. The cement industry is responsible for 5% of global anthropogenic CO_2 emission and the burning of 100000 bricks emits 50 tons CO_2 in Fixed Chimney Kiln (FCK) (ernst worrell et al., 2004; Report No. 60155-BD, Introducing Energy-efficient Clean Technologies in the Brick Sector of Bangladesh., June, 2011). The aim of the present paper is to propose and depict the reduction techniques of energy consumption and consequently, CO_2 emission for the manufacturing of materials used in construction purpose of a building. About 40% of the world's energy use is in the form of electricity; the rest is used for heating and manufacturing. And at least 40% of the world's electricity comes from coals which leads to huge CO_2 emission. In this paper, a reduction technique in the amount of electric energy consumption is proposed and natural ventilation & lightification system using day-light and also using temperature and light effect controlled sensor is discusssed. The review idea of fly ash bricks in construction purpose is also discussed. The CO_2 emission mitigation options include energy efficiency improvement, new processes, a shift to low carbon fuels, application of waste fuels, increased use of additives in manufacturing of building materials. Finally, a comparison between the general building and our proposed building is made.

Keywords: Energy consumption, Climate Change, CO_2 emission.

INTRODUCTION

The world's climate is a great concerning issue now-a-days and rising of CO_2 gas is the major threat for this. The increment of CO_2 is mainly responsible for the combustion of fossil fuel and also due to deforestation. For this reason, the world's temperature is increasing day by day and the height of sea water level is also increasing. The world's main construction materials are bricks and cement which are produced in virtually all countries because of their geographic abundance. The manufacturing of cement emits CO_2 from the combustion of fossil fuel (i.e. coal) and also due to the use of electricity which involves the burning of coal. The burning of clay bricks emits CO_2 by the combustion of fossil fuel and deforestation due to excessive cutting of trees. Energy consumption by the cement industry is estimated about 2% of the global primary energy consumption or almost 5% of the global industrial energy consumption. Also the burning of 100000 bricks emits 50 tons of CO_2 in FCK kiln .For the reduction of Co_2 various kind of structure such earthen structure, wooden structure etc. have got too much popularity but today the main requirement of people is beauty whether it may cost too much or others. So in the age of highly development of technology it is too much of shame that our environment is degrading day by day but we are just only watching. The first objective of this paper is to discuss different ways of mitigation of the CO_2 emission for the manufacturing of building materials.

About 40% of the world's energy is used in the form of electricity. Because of the limitation of this source, the use of electricity for lightening and ventilation purpose is a great concern all over the world. So the second objective of this paper is to propose a reduction technique in the amount of electric energy consumption by providing natural ventilation & lightening system using day light and also using temperature & light effect controlled sensor. Finally, a comparison of construction cost between the typical building (Fig.1) and our proposed model building is reported.

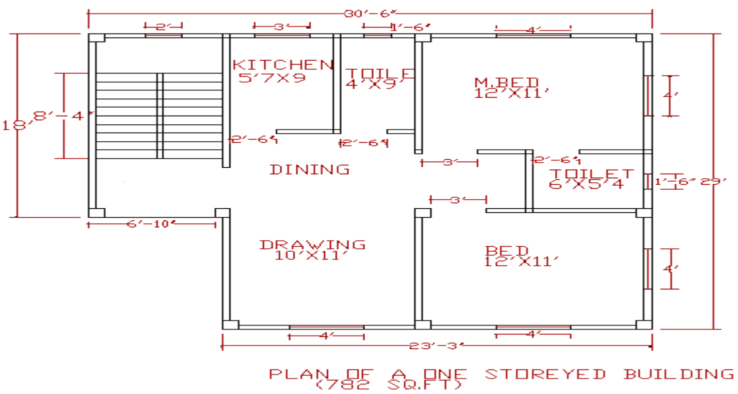

Fig.1: Plan of a typical building

CO_2 EMISSION OF THE TYPICAL BUILDING

As cement and brick are the main materials of building so firstly we have estimated for the typical building (fig.1) and then we have accounted for cement (Table 1) and brick (Table 2) only.

Table 1: CO_2 emission for bricks used in typical building plan

Item	Amount(ft^3)	Total amount in no. of bricks	Equivalent co_2 emission (50 tons per lakh of bricks)
Brick	1730	20760	10.38 tons

Table 2: CO_2 emission for cement used in typical building plan

Item	Amount(ft^3)	Total amount of cement in kg.	Equivalent co_2 emission (0.8 kg. co_2 per kg. of cement)
Cement	251	10040	8032 kg.

PARTICULARS OF OUR PROPOSED BUILDING

Our environment is degrading day by day too much because of industrialization. Many much discussions are going worldwide but no fruitful thing is achieved yet. So we have tried to propose such kind of building that will reduce CO_2 emission, it may be seemed that our proposed building will reduce small amount of Co_2 emission but if we imagine about huge project that will be understood how much amount it will reduce.

1. Fly ash bricks as a replacement of clay bricks

2. Techniques for the reduction of CO_2 emission in cement factory

3. Modified lightening and ventilation

Fly ash bricks as a replacement of clay bricks

Fly ash is a waste material obtained from the combustion of coal. Fly ash brick consists of cement, water and fly ash (fly ash averagely 60%) which is stronger than clay bricks. The properties of a typical fly ash brick is presented in Table 3.

Table 3 : Properties comparison between clay bricks and fly ash bricks

Properties	Clay brick	Fly ash brick
Drying time	7 days	3 days
Length of firing time	1-7 days	Few hours(subject to provisional confidentiality)
strength Compressive	12-40 M Pa	43 M Pa
Average density	1800-2000 kg/m^3	1450 kg/m^3
Absorption capacity	5-20%	10%
Modulus of rupture	1 M Pa	10.3 M Pa
Cost	6 Tk.	20Tk.

Bonding is also good in case of fly ash bricks compared to clay bricks and it requires only a few hours for burning which is better for environment. From statistical data, fly ash bricks cut energy and CO_2 emissions by 90% (Cal Star Products; San Francisco, Cal Star's process). A fly ash brick of sizes 400mm × 200mm×150mm is nearly 7 times of the size of a conventional clay brick of sizes 230mm × 75mm×100mm. Due to this reason where the price of a fly ash brick is 20 Tk. but the price of 7 clay bricks is 42 Tk. So by using fly ash brick half of the total brick budget will be reduced. The comparison of the cost and CO_2 emission between clay and fly ash bricks are shown in Table 4.

Table 4: Comparison of cost and CO_2 emission between clay & fly ash bricks

Items	Amount (nos.)	Cost (Tk.)	Equivalent CO_2emission
Clay brick	20,760	1,24,560	10.38 tons
Fly ash brick	2,966	59,320	1.038 tons

Techniques for the reduction of co_2 emission in cement factory

Many processes have existence in the reduction of CO_2 emission. We will review some processes in the following sections:

- Energy Efficiency Improvement

- Replacing High-Carbon Fuels with Low-Carbon Fuels
- Blended Cements

Energy efficiency improvement

Improvement of energy efficiency reduces the emissions of CO_2 from fuel and electricity uses which can be attained by using more energy-efficient equipment and by improving fuel efficiency. In general, the dry process is more energy-efficient than the wet process. The main opportunities in the kiln are the conversion to more energy-efficient process variants (e.g., from a wet process to a dry process with preheaters), improvement of preheating efficiency, improved burners as well as process control and management systems. Electricity use can be reduced through improved grinding systems, high-efficiency classifiers, and process control systems.

Replacing high-carbon fuels with low-carbon fuels

One option for lowering CO_2 emissions is to reduce the carbon content of the fuel, e.g., shifting from coal to natural gas. An important opportunity to reduce the long-cycle carbon emission is the application of waste-derived alternative fuels. This could at the same time diminish the disposal of waste material and reduce the use of fossil fuels. Alternative fuels may be gaseous (e.g., landfill gas), liquid (e.g., halogen-free spent solvents distillation residues, waste oils), or solid (e.g., waste wood, dried sewage sludge. Waste may reduce CO_2 emissions by 0.1–0.5 kg/kg of cement produced compared with current production techniques using fossil fuels.

Blended cements

In blended cement, a portion of the clinker is replaced with industrial by-products, such as coal fly ash, blast furnace slag, or other pozzolanic materials. These products are blended with the ground clinker to produce a homogenous product: blended cement. Blended cement has different properties than Portland cement, e.g., setting takes longer but ultimate strength is higher. The global potential for CO_2 emission reduction through producing blended cement is estimated to be at least 5% of total CO2 emissions from cement making (56 Mt of CO2) but may be as high as 20%. The potential emission reduction varied between 0% and 29%. The average emission reduction for all countries (producing 35% of world cement in the reference year1990) was estimated at 22.

Modified lightening and ventilation

For the reduction of energy consumption we will discuss mainly for electricity .As we cannot think the existence of civilization without this and because of the limitation of this energy source, we have tried to reduce the use of this. So for reducing the use of this our first ideas about modified lightening and ventilation are-

i. Temperature sensor

ii. Light controlled sensor

Temperature sensor

The use of this sensor is so simple and it is so economical .For the implication of this, we will have to just fit a temperature sensor in the room in a connection with the electric fan or others and the sensor will automatically control the speed of the fan or others. Actually when the temperature will be high the sensor will increase the speed and when the temperature will be low the sensor will decrease the speed.

Light controlled sensor

Similarly for the implication of this ,we will have to just fit a Light controlled sensor in the room in a connection with the electric light and the sensor will automatically control the brightness of the light . Actually when the natural brightness will be high the sensor will decrease the brightness of the light and when the natural brightness will be low the sensor will increase the brightness of the light. This is also too simple and economical.

For reduction of energy consumption our idea is about ventilation system. Our experimental methodology is discussed below.

i. Natural ventilation system
ii. Building orientation arrangement for ventilation

Natural ventilation system

For this we have worked with several types of building .Our main purpose is to make comfortable temperature in the room. Our experimental building no.1 views are shown in fig 1.3 which is covered by lot of trees. We have tried to show how much the temperature changes from outer face to inner face.

Table 5: Temperature Measurement chart in three faces of mosque (Kazla, Rajshahi)

Point	North Face(Temp. ^{0}C)		South Face(Temp. ^{0}C)		Front Face(Temp. ^{0}C)	
	Inner	Outer	Inner	Outer	Inner	Outer
Point -1	28.4	30	29	29.8	29.4	31.1
Point-2	28.5	30.3	28.5	30	29.4	31
Point -3	28.8	30.1	28.2	29.9	29.2	30.9
Point -4	29.1	30.2	27.9	29.5	28.9	30.7
Point -5	29	30	28.1	29.6	28.8	29.6

From the above data it is seen that there is a certain amount of changes in temperature from outer face to inner face especially for north and south faces.

Fig. 1.1:North face

Fig. 1.2:south face

Fig. 1.2:Front face

Fig. 1.3:Mosque (kazla,Rajshahi,Bangladesh)

Fig. 2: Infrared thermometer (KYORITSU, KEW 5515)

Building orientation arrangement for ventilation

In this method our aim is to reduce the room temperature for the arrangement of ventilation system and difference in room temperature in north-south faces than east-west faces with room inner temperature.

Table 6: Temperature Measurement chart in North-South &East-West faces of building no.2

Point	North Face(Temp. ^{0}C)		SouthFace(Temp. ^{0}C)		East Face(Temp. ^{0}C)		West Face(Temp. ^{0}C)	
	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
Point -1	28.5	32.9	28.3	32	29.2	33	29.5	32
Point -2	28.8	32.7	28.4	32.6	29.5	33.5	29.7	32.7
Point -3	28.9	33	28.3	32.7	29.9	34.2	29.7	32.9
Point -4	28.9	33.3	28.5	32.7	29.8	34.4	29.8	33.1
Point -5	29	33.5	28.6	32.8	30	34.9	30	33.2

((Vhodra, Rajshahi)

The above data identifies that there is a certain amount of changes in temperature from outer face to inner face especially for north and south faces.So a north-south facing house is recommended.

CONCLUSION

1. Fly ash brick is also eco-friendly as it reduces CO_2 emission too much. Our typical building plan is too small, if we think for a huge amount of project then it will be so cleared about cost and CO_2 emission.

2. In case of cement production as a fuel coal is so important and which emits CO_2 .We have tried to review some processes for the reduction of CO_2 emission in cement production.

3. For modified ventilation we have discussed about the sensor idea .It may be seemed that it will reduce few amount of power but if we think about lot of houses even for a country or even for lots of countries that it will be understood how much amount we are saving.

4. We have reviewed the natural method and Building orientation arrangement for ventilation system which are better for making comfortable temperature in room. These processes also help to reduce the use of electricity which emits CO_2 in its production due to burning of coal, so ultimately we can reduce cost and CO_2 emission.

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ID: SEM 028

WATER LOGGING PROBLEMS AND PROBABLE ANALYTICAL SOLUTIONS: A CASE OF TEKNAF POURASHAVA

Nureen Faiza Anisha[1]* & Shahadat Hossain[2]

[1] *Dept. of Water Resources Engineering, Bangladesh University of Engineering and Technology, Dhaka-1200, Bangladesh.*
[2] *Institute of Water Modelling, New DOHS, Mohakhali, Dhaka-1206, Bangladesh.*
**Corresponding Author, <anishahossain@yahoo.co.uk>*

ABSTRACT

Bangladesh being located on the extensive flood plains of the Ganges and Brahmaputra Rivers, flooding is a natural part of life here. Thus water logging in the urban area is not a new problem. It's a part of life but the frequency of this problem is increasing. The urban area has been experiencing water logging for the last few years even a little rain causes a serious problem for certain areas. Increased urban development is not providing sufficient drainage results in water logging, leaving parts of urban area inundated for several days. This causes naturally large infrastructural problems for the city. Most of the time during the monsoon, the water level of the river remains higher than the terrain inside the urban area. Hence, standard draining by gravity may not always be possible. It has been identified that improvement of the drainage system is one of the highest priority of the urban authority for living environment of its urban population. The urban area 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 urban area 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 Teknaf, a small urban area of southern part of Bangladesh beside the Bay of Bengal. The developed technology can easily be transferred and applied to catchment areas experiencing similar problems.

Keywords: Waterlogging, Urban Area, Drainage Study, Teknaf

1. INTRODUCTION

Bangladesh is a country of heavy rainfall and regular floods. The mean annual rainfall is about 2320mm. Bangladesh is one of the most densely populated countries of world. The cities face some regular problem like traffic congestion, water logging, water pollution, improper waste management, air pollution etc. Water logging is one of the major problems of the urban area. With uncontrollable population growth and unplanned urbanization, the drainage and sewer facility are not developing accordingly which is causing massive water logging around the country in urban areas. This study aims to do a case study of the current urban water logging in Bangladesh, trying to highlight the main causes behind it, the influence of undertaken projects and people's perspective about the effects of water logging in their urban areas.

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 to provide the urban area a free area from water logging/congestion within an acceptable environmental condition. Present drainage system is insufficient or not enough to handle the situation of draining the runoff resulting from heavy rainfall. There are number of places in the urban area where water logging/drainage congestion occurs after heavy rainfall. The development objective of the drainage system works is to provide new drains, cleaning of existing drains, re-excavation of the existing natural canals/khals for increasing their discharge capacity, repair/rehabilitation of existing drains up to their full capacity, provision of new cross drains/ culverts and repair/rehabilitation of existing drainage culverts/cross drains including removal of blockage from the existing drains. The improvement of drainage system of TEKNAF urban area 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 and protection of the water quality at outfall.

2. METHODOLOGY

Drainage system of an urban area is assessed through a sequence of analytical processes and it finally results in a proposed drainage system. The proposed drainage system is planned for gravity drainage. The study includes data collection from secondary sources, analyzing and checking of data, development of hydrological model, reviewing and correlating urban area drainage system with existing regional models, identification of design year and simulation of the model for the design year, determination of design flows from model simulation, calculation of design parameters from design flows etc.

Location and Topography

TEKNAF Urban area is located in TEKNAF Upazila, COX'S BAZAR District under Chittagong Division. The change in elevation of most of the urban area is gradual. The land elevation of the urban area effectively ranges between 0.88 mPWD and 10.33 mPWD. Insignificant fractional percent of land lies outside the above range.It is assessed that only 20% land of the urban area is below 0.88 mPWD while 41%, 66%, 87%, 97%, 99% and 100% of the land are below 2.46 mPWD, 4.03 mPWD, 5.61 mPWD, 7.18 mPWD, 8.75 mPWD and 10.33 mPWD respectively. The use of present urban area's area can be broadly divided into lands for agricultural (10%) and non-agricultural (90%).

Rainfall

Design rainfall storm intensity for the urban area 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 urban area. This implies that the storm event of Dhaka and the urban area is correlated by the conversion factor. The conversion of storms at Dhaka to storms at urban area follows the Urban Drainage Manual, May 1998 of Local Government Engineering Department. The conversion procedure is also briefly given in section 1.3.1.Kutubdia is a rainfall gauging station (R316) with reasonable length of records and is located nearest to the Urban area. It is selected as the reference station for assessment of storm intensity for TEKNAF Urban area. The station is about 115 km distant from the urban area.

River and Khal System

Regional river system adjacent to the urban area is shown in **Figure: 1**.The Teknaf urban area lies on the right bank of Naf River. The Naf River flows north to south beside the Teknaf urban area. The khal system of this area is shown in **Figure 1**. There are two khals named Heccha Khal and Kayokkhali Khal branch. Heccha Khal flows through the urban area and the downstream route to Naf River and finally falls into Bay of Bengal. Kayokkhali Khal carries storm water from major parts of the Urban area and drains into Naf River and finally to Bay of Bengal. There are some low lying lands on the eastern part of the urban area which serves as runoff destination of storm water from some parts of the urban area and finally route to Naf river system.

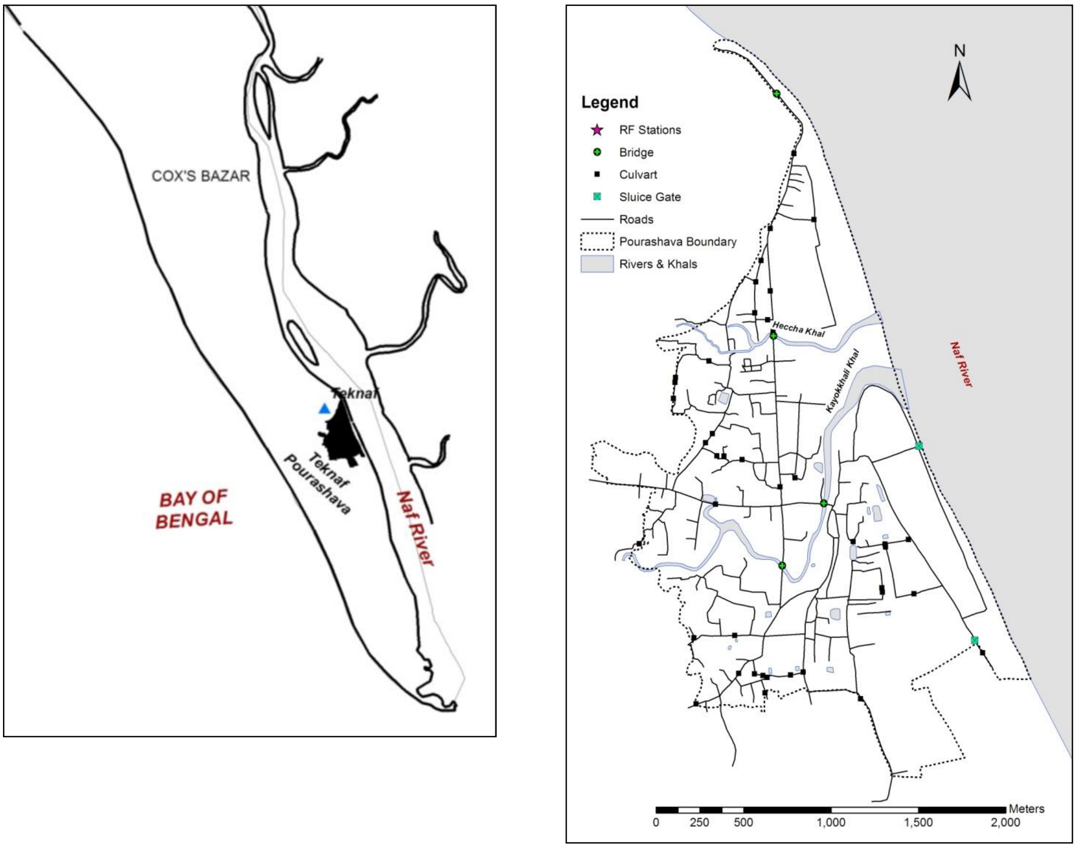

Figure 1: Regional River system adjacent to the Teknaf urban area

Flood

The Urban area lies in the Naf River basin. The water level gauging in close vicinity of the urban area is available at Saflapur (200) on the Matamuhuri River. From field visit, local people's opinion and Satellite base flood map analysis the water level of MatamuhuriRiver is adjusted for TEKNAF Urban area. Thus average year flood level at the Urban area is estimated to 1.36 mPWD.It is assessed that 70% of land of the Urban area is above the average flood level. The rest of the land ranges from moderate to very deep flooding. It is assessed that 6%, 18% and 6% of land is subjected to moderate (30-90 cm flood depth), deep and very deep (90-180 cm flood depth) flooding in reference to average year flood.

Existing Drainage System

There exist few lined and unlined drains within the urban area. These can drain some local areas of the urban area. The capacity and outfalls of existing drainage system is not planned with well defined consideration of drainage areas/zones for the whole urban area. Many of the drains randomly fall into relatively low lying areas. Such arrangement has allowed drainage relief for the very local areas and for the time being only. Most of these outfalls will not be available in course of future development at the location of such outfalls. The lengths of existing lined and unlined drains are about 8.79 km.In absence of planned and adequate drainage system, the Urban area in places suffer from drainage congestion and water logging after heavy rainfall.

Following the field visits and engineering survey, the main concerns for drainage issues of the Urban area 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.

Water Logged Area

Inundation occurs in some localized places of the urban area after heavy rainfall in absence of appropriate drains and routes. Presently mentionable water logging is observed following moderate to heavy rainfall in and in the vicinity of ward -3, 4, 5& 6.The depth and duration of inundation vary from place to place. Such areas are freed from inundation by the process of evaporation and infiltration. The reasons for inundation/water logging are technical, social and institutional. These water logged areas have been considered and brought under proposed drainage network.

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 TEKNAF Urban area 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)

Identification of Outfalls

The outfalls for the present and expanding core area of the Urban area are in the Heccha Khal and Kayokkhali Khal branch. The outfalls of the proposed drains and drainage zones have been identified and exhibited in the following section.

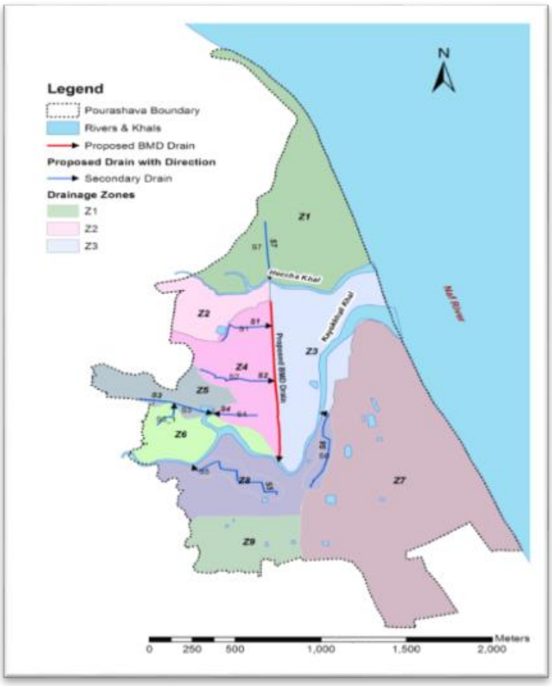

Figure 2: Proposed Drainage system for Teknaf urban area

Proposed Drainage System

About 70% of land of the urban area is above the average flood level. Proposed drainage system has been planned for the core area of the urban area as well as for the extended area in near future in consideration of priority needs.The area of the urban area has been planned for improvement under gravity drainage system. The whole urban area has been divided into 9 zones for drainage improvement plan as shown in **Figure 2**. The proposed drainage system for the Urban area focusing the drainage basins/zones,sub basins/ sub zones, dependency of zones/sub zones, drainage network, outfalls (khals/rivers/beels/destination).Zones 4-7, Zone 10, Zone 14 and Zone 16 are planned with proposed storm drains as they are in the core area of Urban area or will be characterized as core area in near future.

Table 1: List of proposed drainage system in TEKNAF Urban area

Sl No	ID of Drain and Catchment	Length m	Area sqkm	Drain Type	Outfall
1	S1	393.01	0.03	Sec/Rec/ RCC	Proposed BMD Drain
2	S2	482.32	0.06	Sec/Rec/ RCC	Proposed BMD Drain
3	S3	449.99	0.02	Sec/Rec/ RCC	Kayokkhali Khal
4	S3_1	216.36	0.02	Sec/Rec/ RCC	S3 to Kayokkhali Khal
5	S4	261.29	0.04	Sec/Rec/ RCC	Kayokkhali Khal
6	S5	733.33	0.07	Sec/Rec/ RCC	Kayokkhali Khal
7	S6	632.67	0.05	Sec/Rec/ RCC	Kayokkhali Khal
8	S7	504.25	0.04	Sec/Rec/ RCC	Heccha Khal

Table 2: Frequency Analyses for WL at the Outfall Channels of TEKNAF Urban area

Return Period	Water Level (mPWD)
1.1 year	0.11
2 year	1.16
2.33 year	1.34
5 year	2.12
20 year	3.37

An assessment is made by correlating the land elevation of the core area of TEKNAF Urban area to the water level at outfalls which is given in **Figure 3**. It is observed that average land level of core area in the Urban area is higher than the average water level.

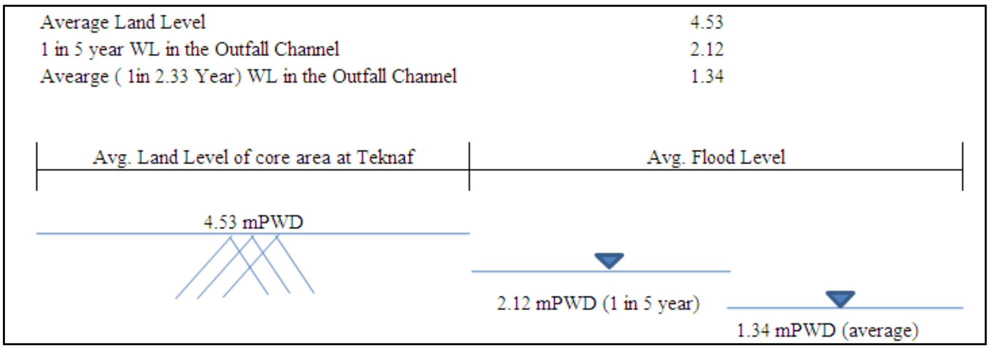

Figure 3: The land elevation of the core area of TEKNAF urban area

Effects of Water Logging in Urban Area:

The logged water becomes polluted with solid waste, silt and contaminants that are washed off from roads. The increase in volume and rate of logged water causes erosion and siltation. It becomes a burden for the inhabitants of that urban area, leading to unhygienic environment and creating adviser social, physical, economical as well as environmental impacts. A survey done through a common questionnaire in four major urban areas of Bangladesh, Dhaka, Mymensingh, Chittagong and Sylhet, in various parts of those urban areas shows a brief perspective of them inhabitants about the problems caused by water logging. The table below is made with a sample of 50 samples in each city.Different Types of Problems Faced due to Water Logging in Urban Areas (Sylhet, Dhaka, Mymensingh and Chittagong) of Bangladesh [Source: Field Survey]

Problems	Percentage
Disruption in Traffic Movement	76
Disruption in Normal Life	94
Damage of roads	82
Damage of houses	69
Damage of household goods	45
Damage of underground service lines	62
Water pollution	68
Water borne diseases	71
Damage of trees and vegetation	49
Increase of insects	38
Increase of construction and maintenance cost	22
Death and damage of fisheries	9

CONCLUSIONS

The highlight features of the urban area system in connection with the issues of its storm drainage are that Teknaf lies on the right bank of Naf River. The Naf River flows north to south beside the urban area.There are two khals named Heccha Khal and Kayokkhali Khal branch.The relatively low lying area of the urban area is flood affected from internal rain feed. The part on the north and south of Urban area is affected by the internal rain fed flood.The area of the Urban area is comparatively high. The land elevation of the urban area effectively ranges between 0.88 mPWD and 10.33 mPWD. The average year flood level in the Urban area is estimated to 1.34 mPWD.It is assessed that 70% of land of the Urban area is above the average flood level. The rest of the land ranges from moderate to very deep flooding. It is assessed that 6%, 18% and 6% of land is subjected to moderate (30-90 cm flood depth), deep and very deep (90-180 cm flood depth) flooding in reference to average year flood.There exist few lined drains within the Urban area.They are much unplanned and lack in systematic drainage network. Some localized places of the urban area suffer from inundation due to internal storm water drainage congestion, and water logging in few places in absence of adequate gravity drainage provision and routes.Flow capacity of the drains is largely reduced as they are choked up with solid wastes. Lack in social awareness is a huge concern for smooth functioning of the drains.

RECOMMENDATIONS

Proposed drainage systems have priority needs secondary drains are recommended at this level of feasibility study. Detailed engineering survey is a pre-requisite for planning of tertiary drains, and is recommended for planning and design at the time of detailed engineering. About 12 nos. of cross drainage works (e.g.; box culverts/ pipe culverts) will be required in connection with the whole proposed drainage network. About 08 nos. of cross drainage works will be required for the priority drainage systems and the rest will be required for the future drainage systems.Raising of low land with earth fill above the flood level is a pre-requisite for the land be brought under gravity drainage. It is recommended that such land is raised to the similar level of high land (not less than 1.34 mPWD) of the Urban area.Protection works at outfalls will have to be provided in consideration of design energy dissipation.Trash racks and silt traps shall have to be provided at regular interval during construction of drains. The existing drains and also those will be constructed shall have to cleaned at regular interval. Maintenance and cleaning of drains shall have to done at least once in every year specially before starting of monsoon season. Zone 7 & 9 of the urban area will drain overland across the urban area boundary to the lying beels and finally route and drain to Naf River. The Urban area authority will have institutional linkages with all relevant line agencies for the continuation of drainage provision of Zones 7 & 9 in view of long term consideration.The Urban area authority will monitor the water level of Outfall Khals & Rivers, record the drainage congestion area of each significant storm, and maintain the existing river and khal so that natural channels / drains are not encroached anyhow. The Urban area authority will have institutional linkages with all relevant line agencies such as BWDB, LGED and in particular with DPHE in connection with operational bottlenecks of the planned system and for maintenance of appropriate section all through its natural drainage routes.

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ID: SEM 029

CHANGES OF REFERENCE EVAPOTRANSPIRATION (ET_0) IN RECENT DECADES OVER BANGLADESH

Mohammad Alfi Hasan[1], A. K. M. Saiful Islam[2], & S.M. Bokhtiar[3]

[1] *Institute of Water and Flood Management, Bangladesh University of Engineering and Technology, Dhaka, 1000, Bangladesh.*
[3] *Bangladesh Agricultural Research Council (BARC), Dhaka, 1200, Bangladesh.*
**Corresponding Author, <mdalfihasan19@gmail.com>*

ABSTRACT

The quantification of water balance is crucial in Bangladesh due to increasing demand of water for agricultural production. One way to estimate the water requirements is the determination of reference evapotranspiration (ET_0). The ET_0 values measured or calculated at different locations or in different seasons are comparable as they refer to the evapotranspiration from the same reference surface. The only factors that affect ET_0 are the climatic parameters. Any change in climatic parameters might lead to changes in ET_0 over a particular area. In this study, an attempt has been made to investigate the changes of ET_0 in the past several decades over Bangladesh under the changing climate. Daily observed data are collected from the Bangladesh Meteorological Department (BMD) for the 1971 to 2010 time period. ET_0 are analyzed using the FAO Penman-Monteith method which is recommended as the sole method for determining ET_0. A software called CROPWAT 8.0, developed by Water Resources Development and Management Service of Food and Agricultural Organization (FAO) has been used to calculate ET_0 in the study. Two historical time-slices each 20 years of length i.e. 1971-1990 as 1980s and 1991-2010 as 2000s are considered to investigate the change of ET_0 over Bangladesh. In the recent decades, the average evapotranspiration in Bangladesh has reduced from January to April. However, from July to December, ET shows slight increase in recent decades. Spatial Analysis has revealed that ET_0 has reduced more in the western part than in the eastern part of the country. The south eastern region of Bangladesh shows a notable decrease of ET_0 particularly in Meherpur District during *Kharif-I* cropping season.

Keywords: Agro-meteorology, Climate Change, Reference evapotranspiration

INTRODUCTION

According to Allen et al. (1998), reference evapotranspiration (ET_0) or sometime referred as reference evapotranspiration (ET_0), can be defined as the rate of evapotranspiration from a hypothetical reference crop with an assumed crop height of 0.12 m, a fixed surface resistance of 70 sm/1 and an albedo of 0.23. This parameter plays an important role in hydrology, watershed management and agriculture sectors. Estimation of ET_0 is essential for the irrigation scheduling, calculation of crop water requirements, design of irrigation and drainage structure as well as for the study relative to climate variability (Liang et al., 2010). As Bangladesh has agriculture based economy, this parameter exerts more importance in water management practice of the country.

ET_0 depends on several climatic parameters like air temperature, wind speed, relative humidity, and shortwave radiation etc. However, it is evident from recent report of IPCC (2007) that earth temperature has been continuously raising in recent decades, causing a change in total climatic system of the globe. As a consequence, all the dependent parameters of ET_0 also showed some differential changes in their variability and magnitudes. This provides a heterogeneous changing pattern of ET_0 all around the world. Such changes, eventually affects a number of vital sectors e.g. agriculture and food security, water resources and ecosystem, health and livelihood etc. both in global and regional scale. Thus, for the climate vulnerable country like Bangladesh, it is essential to understand the change of ET_0 for the proper management practice, effective planning and robust decision making in the agriculture and water sectors. Several studies have already been made regarding the regional changes of ET_0 under climate change (Chattopadhyay and Hulme, 1997; Gong et al., 2006; Goyal, 2004; Moonen et al., 2002; Shenbin et al., 2006; Wang et al., 2007; Xu et al., 2006; Youqi et al., 2008), but none of them are available over this region. Therefore, this study is conducted to identify the significant changes of ET_0 over Bangladesh in both spatial and temporal scale.

Estimation of ET_0 can be done in two ways, one is by meteorological data and another is by hydrologic models. The Penman–Monteith (PM) equation has emerged as the de-facto standard to calculate ET_0 with the help of meteorological data. This method provides values that are very consistent with actual crop water usage data worldwide as it has been demonstrated through many years of evaluations reported in the scientific literature. This method explicitly incorporates both physiological and aerodynamic parameters. Moreover, procedures have been developed for using this method even with limited climatic data. Therefore, this study calculates and compares ET_0 over Bangladesh under prevailing climate change condition. To estimate ET_0, daily meteorological data were collected from 36 stations over Bangladesh from 1971 to 2010. Changes of estimated ET_0 are determined by comparing results of the last 20 years with the previous 20 years.

METHODOLOGY

Bangladesh is tropical country with a high intensive annual rainfall. Other than rainfall and temperature data, meteorological variables like wind speed, solar radiation etc. are not available with continuous integrity in the existing global observed data sets. However, data from 34 stations maintained by the Bangladesh Meteorological department (BMD) are available that allowed to calculate ET_0 estimates over the country. In this study, six daily meteorological variables are collected from BMD including daily (1) minimum air temperature (Tmin, °C), (2) maximum air temperature (Tmax, °C), (3) relative humidity (Rh), (4) wind speed at 2 m (U_2, m/s), (5) bright sunshine hours (N, h/d) and (6) precipitation (P, mm/month). Mean monthly air temperature are calculated by taking the average of the maximum and minimum air temperature. After quality control, six stations are removed from the analysis as they have missing value greater than 20% and failed in the homogeneity test. Twenty eight out of these thirty four stations has long term quality controlled data sets dated back to the independency of country (at 1971). Data from all stations ranging from 1971 to 2010 have been used to compare the mean state of reference evapotranspiration over Bangladesh. Two periods of each 20 years of length (i.e. from 1971 to 1990 as 1980s and from 1991 to 2010 as 2000s) are considered to investigate the change of climate variables.

Daily ET_0 rates has been estimated according to the Penman–Monteith (PM) procedure and summed to monthly values. The Penman–Monteith approach is regarded as the most accurate method under all climates giving estimates that differ less than ±10% from the actual values (Allen et al., 1998). The PM method for calculating daily reference evapotranspiration is:

$$ET_0 = \frac{0.408\Delta(R_n - G) + \gamma\frac{900}{T+273}u_2(e_s - e_a)}{\Delta + \gamma(1+0.34u_2)} \quad (1)$$

Where, ET_0 is the reference evapotranspiration (mm day^{-1}); R_n is the net radiation at the crop surface (MJm^{-2} day^{-1}); G is the soil heat flux density (MJm^{-2}day^{-1}); T is the mean daily air temperature at 2 m height (°C); u_2 is the wind speed at 2 m height (ms^{-1}); e_s is the saturation vapour pressure (kPa); e_a is the actual vapour pressure (kPa), Δ is the slope vapour pressure curve (kPa°C^{-1}) and γ is the psychrometric constant (kPa°C^{-1}).

With the collected climatic variables, the calculation of ET_0 has been done using a software known as 'CROPWAT'. 'CROPWAT' is a software developed by Joss Swennenhuis for the Water Resources

Development and Management Service of FAO and useful tool for determination of crop water requirements and irrigation requirements of a particular crop (Clarke et al., 2001). All calculation procedures of the software are based on the FAO guidelines and it can used to calculate references evapotranspiration or ET_0 using PM method. Using this software ET_0 has been estimated with 28 station data for the time period of 1971 to 2000. After calculation of daily ET_0 rate, monthly and climatic ET_0 has been determined to assess the potential changes of ET_0 in recent years. For the agricultural importance of ET_0 in Bangldesh, the assessment has been made for three cropping seasons, namely *Rabi*, *Kharif-I* and *Kharif-II*. *Kharif-I* season ranges from March to July, *Kharif-II* season ranges from July to October and Rabi season ranges from November to February.

RESULT AND DISCUSSION

To understand the change of climatic mean of ET_0, monthly evapotranspiration has been analyzed for 1980s and 2000s time period as shown in Figure 1. From the January to April, average evapotranspiration in Bangladesh reduces from the past decades (1980s) and highest reduction is observed during January is about 6%. But from July to December, ET_0 shows increasing trend in the recent decades (2000s). As, rainfall during November and December is very low compare to other months of the year, continuing increases of ET_0 during these months results irrigation water deficiency over the country which might hamper the production of cold loving *Rabi* crops like wheat, potato etc.

Spatial distribution of reference ET has made in the study for two so called historical time slices. Comparison has been made for different croping seasons of the country as shown in Figure 2 and Figure 3. In 1980s, ET_0 over the western part of the country varies from 4.5mm to 5.7mm per day, whereas the eastern part of the country experiences lesser rate of ET_0 during *Kharif-I* season. However, reference ET distribution shows less spatial variability ranging from 3.8 to 5 mm per day in 2000s. The south eastern region of Bangladesh shows a notable decrease of reference ET especially in Meherpur District. It can be suggested that reference ET has decreased during *Kharif I* season in recent climate. On the other hand, during *Kharif II* season, reference ET has shown more variability in the coastal areas of the country in 1980s and deceased at a rate of 0.2 to 0.3 mm per day in 2000s. But in the north western parts of the country, reference ET has been increased rapidly than any other parts of the country. On the other hand, during *Rabi* season, reference ET has not changed much around the country. Slight reduction of reference ET occurs in the western parts of the country.

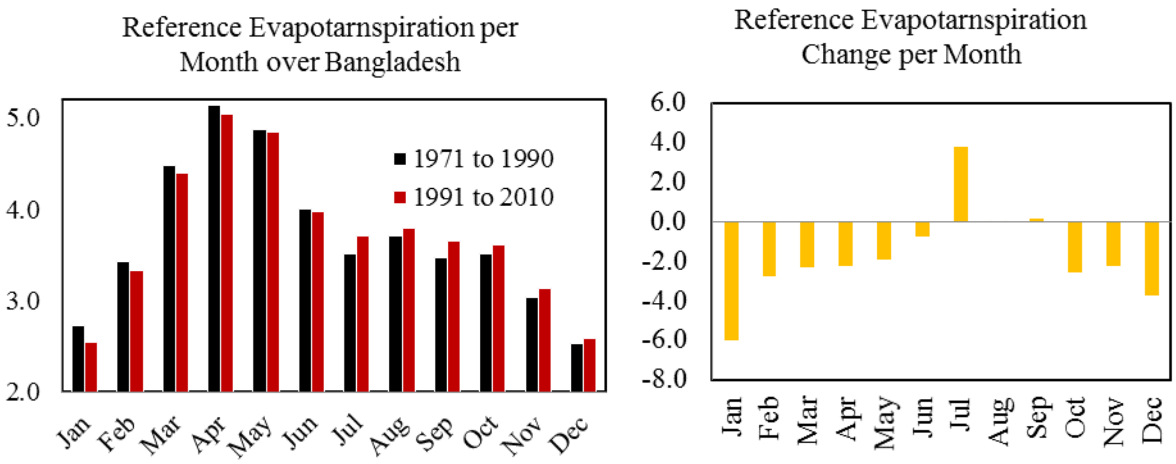

Figure 1: Monthly reference evapotarnspiration (left) and their changes (right) over Bangladesh during two histrocial time slices (1971-1990 & 1991-2010).

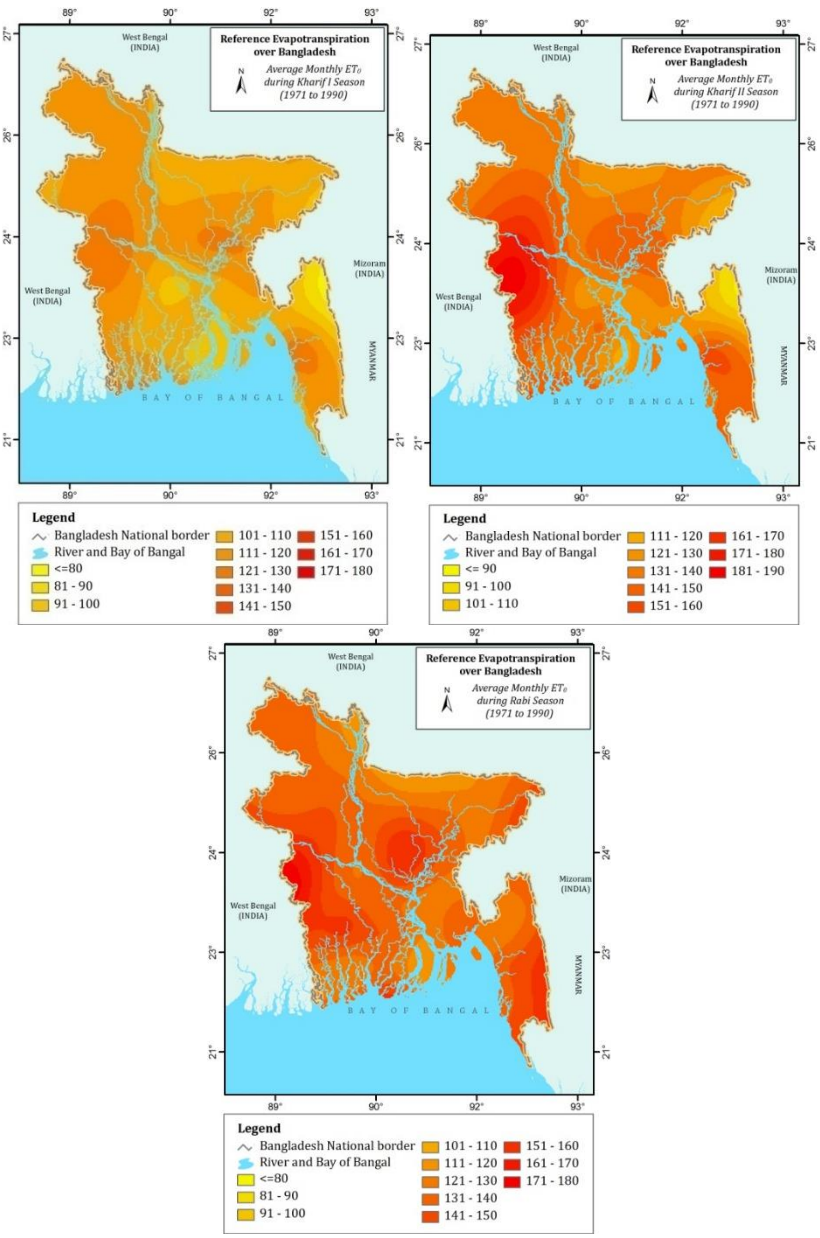

Figure 2: Reference evapotranspiration over Bangladesh at *Kharif I*, *Kharif II* and *Rabi* season (left to right) during 1971 to 1990.

CONCLUSION

Reference evapotranspiration (ET_0) has been estimated over Bangladesh in both monthly and seasonal time scales during two historic periods (1971-1990 and 1991-2010). Study reveals that ET_0 have decreased from January to April and increased from July to December. Decrease of evapotranspiration during November and December might hamper the crop production. Condition will be much aggravated for the cold loving crops as the night time temperature will increase in the future (Hasan et al., 2013).

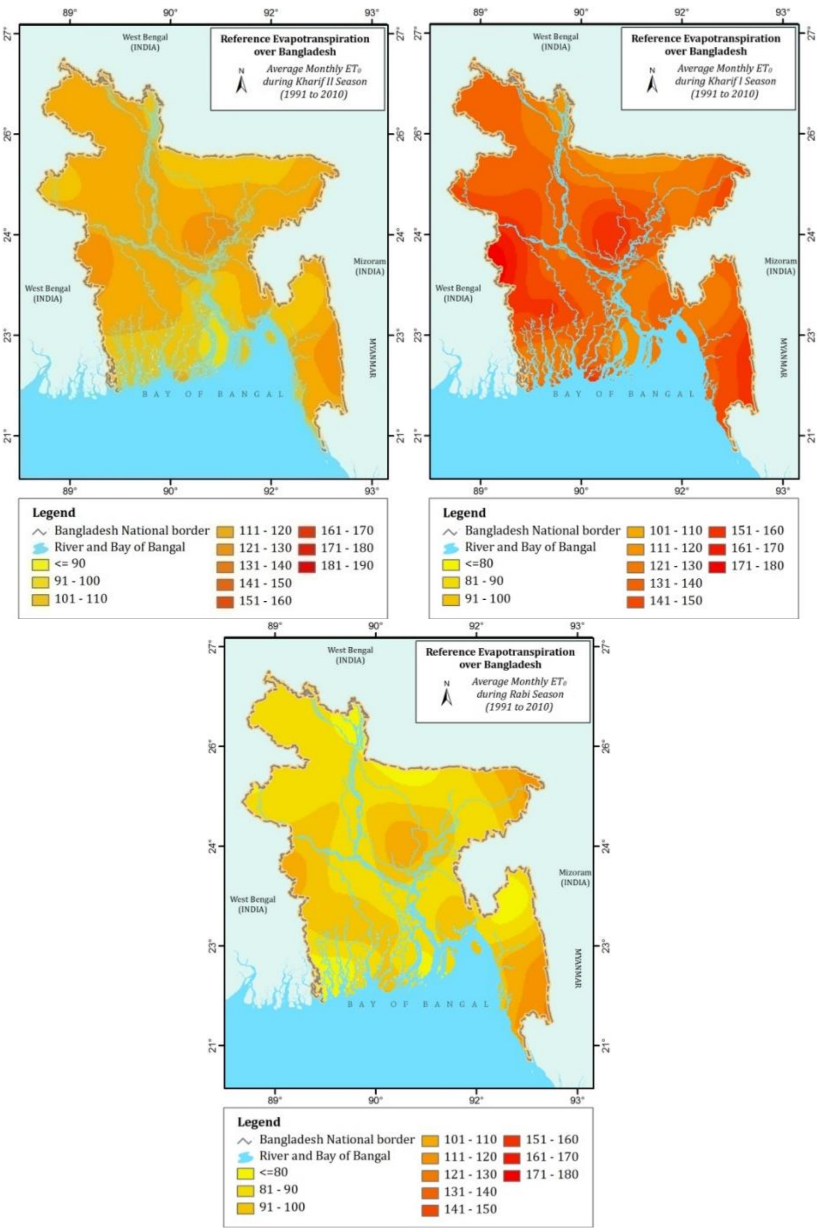

Figure 3: Reference evapotranspiration over Bangladesh during *Kharif I*, *Kharif II* and *Rabi* seasons (left to right) in 2000s.

Kharif-I seasons observes a decreasing trend of mean ET_0 over Bangladesh. During *Khari-II* season, ET_0 increases rapidly in the north western parts of the country. During the *Rabi* season, ET_0 decreases at a uniform rate in all over the country. Seasonal water requirements of any particular crop can be determined by multiplying crop coefficient of that crop. This study reveals spatial and temporal changes of ET_0 over Bangladesh which can help to improve the future water management practices of the country. This information can also be useful for the improvement of agro-climatic zoning of Bangladesh.

ACKNOWLEDGMENTS

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IMPACT OF URBAN PARTNERSHIP FOR POVERTY REDUCTION (UPPR) PROJECT: A CASE STUDY ON WARD NO. 35 OF CHITTAGONG CITY CORPORATION

K. M. Rahman[1*] & N. A. Hashi[1]

[1] *Department of Urban and Regional Planning, Chittagong University of Engineering and Technology, Chittagong, 4349, Bangladesh.*
**Corresponding Author, <kazi_jonas@yahoo.com>*

ABSTRACT

In urban area huge amount of poor people leads unsatisfied life. They are deprived from their basic needs and other facilities. Urban poor is one of the major problem in developing countries. So Urban Partnership for Poverty Reduction (UPPR) Project is run to reduce the urban poor. This paper has examined the impact of Urban Partnership for Poverty Reduction (UPPR) Project which is executed by Local Government Engineering Department (LGED). How this project is run and how much the poor people is benefited from this project is necessary to identify. To examine the impact of this project some project area is surveyed and get comparison between their present and past condition. The data was collected by field survey and the findings show the change of their socio-economic condition and livelihood pattern.

Keywords: Community Development Committee (CDC), Community Acton Plan (CAP), Local Government Institution (LGI), Settlement Improvement Fund (SIF), Socio Economic Fund (SEF), Operation and Maintenance (O&M) Fund.

INTRODUCTION

Background of the Study

Poverty is about not having enough money to meet basic needs including food, clothing and shelter. However, poverty is more, much more than just not having enough money (ESIC, 2008-09). The incidence of Poverty in Bangladesh is one of the highest in the world. Present scenario shows that about one-third of its total population 31.5 percent are living below the poverty line (HIES, 2010). The estimated population of CCC is about 2.7 million (about 562,500 households) among which 301,527 households are poor live in about 5778 poor settlements according to a primary survey conducted in 2010 (CCC, 2011). These poor settlements suffer from lack of services and facilities. Migration from rural areas is one of the driving force for increasing urban population. Government of Bangladesh has taken different attempts to reduce urban poverty along with various development partners. Urban Partnership for Poverty Reduction Project by UNDP is such kind of development partner. This project is executed in 10 city corporations and 14 Pourashava of the country. The aim of UPPR is to improve the livelihoods and living conditions of three million urban poor people especially women and girls by 2015. It is expected that after the implementation of the project local communities will be mobilized, living environment will be improved, income and assets of poor people will be increased and pro-poor urban policies and partnerships will be supported at the national and local levels.

Goals and Objectives

Chittagong is a very dynamic city in terms of commerce, industry, education, tourism, and port facility. It attracts a wide range of poor and low income people from different rural areas which accelerates the growing of poor settlements here. It is necessary to improve the socioeconomic condition and livelihood pattern of these settlements. Urban Partnership for Poverty Reduction Project

(UPPRP) is run to improve the livelihoods and living conditions of three million urban poor people, essentially women and girls by 2015. Evaluation of this project is important as impact of this project will help to take more other steps of urban poverty reduction. Evaluating community planning and implementation of basic infrastructures in poor community is the goal of this study. According to this goal the objectives are to review the working procedure of UPPR project and to identify the improvement and mobilization of the poor community.

METHODOLOGY

Two sources of data are used in this study classified as primary data and secondary data. As primary data a reconnaissance survey has been conducted in the areas with a vision to build up an initial idea about the study areas. Many areas have visited to know the past & present condition of the areas from the people. The socio-economic maps prepared by the community people have used as secondary data. The ward boundary map & related important data from CCC have also used for the report. Finally a report have been written after analyzing and evaluating the condition as well as prospect and potentiality finding.

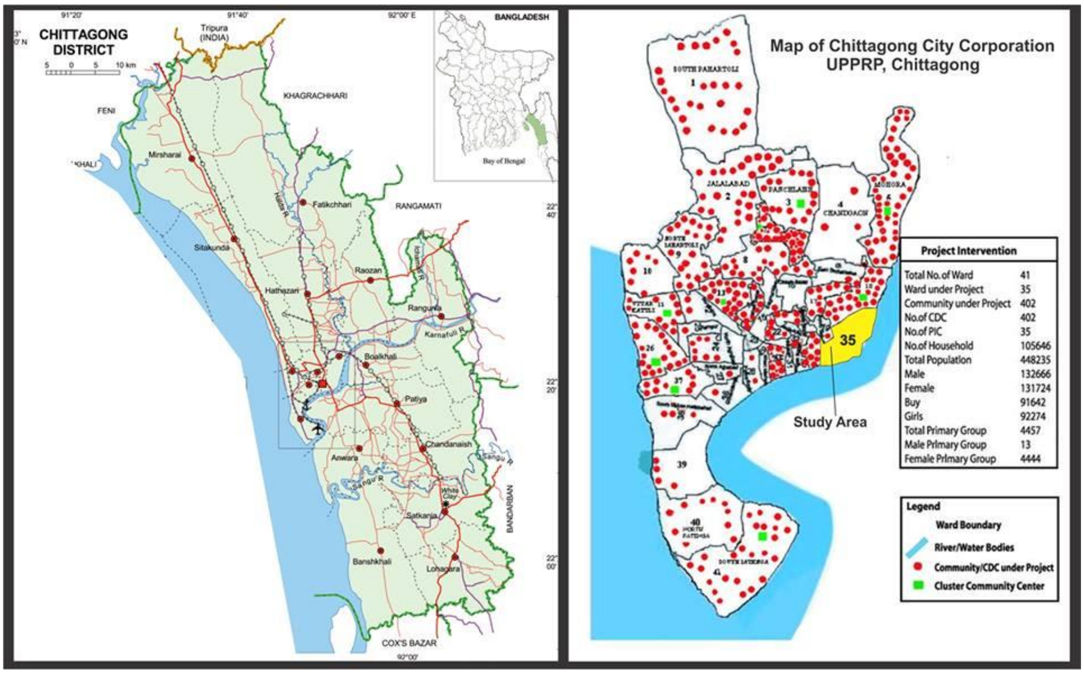

Fig. 1. Location map of the study area

Source: UNDP, 2013

UPPRP WORKING PROCEDURE

Project development Methodology

The UPPR project methodology which is provided very scientifically by the authority shown in fig. 2. This procedure is very effective and it is successfully run in the poor community.

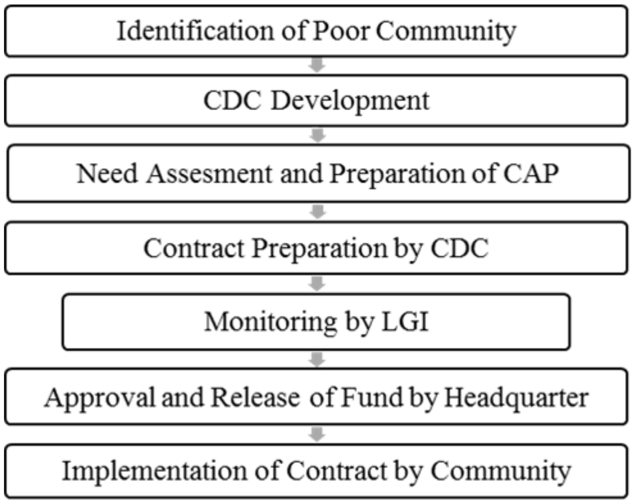

Fig. 2. Project Development Procedure
Source: UNDP, 2013

Community Development

First of all communities with at least 70-80 % poor families are selected through a survey. Maximum one family member is allowed to be the representative of the community. Then, 20-25 representatives from a group which is termed as Primary Group. Some primary group forms a CDC. There is no hard and fast rule about the number of families to form a CDC. But 400 families are generally considered as threshold families for a CDC. Some CDC forms a Cluster CDC. All Cluster CDCs are monitored under a Town Federation Team. The following figure depicts the hierarchy of the community development:

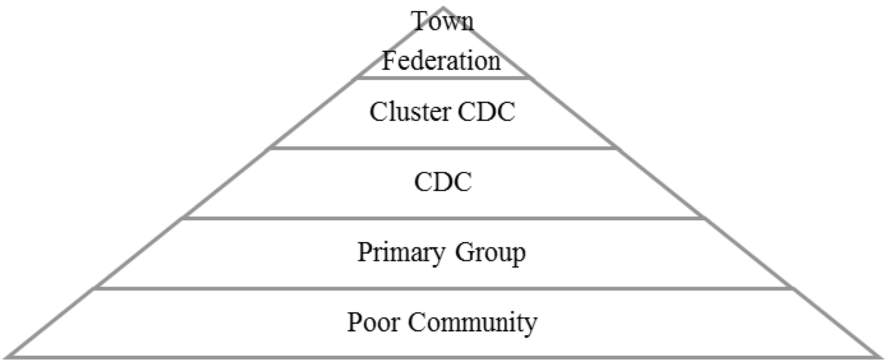

Fig. 3. Hierarchy of the Community Development

Source: UNDP, 2013

Phases of CAP Development

The Community Acton Plan (CAP) workshop proposed for UPPR would follow six phases of work for development of a CAP.

Table 1: The phases of CAP workshop

Phase 1	Identification of Community Assets	What are the community assets? This gives people a positive place to start a plan.
Phase 2	Building A Community Vision	A shared vision for the future to bring positive community change.
Phase 3	Identification and Prioritization of Needs and Development of	What are the ways and means to meet community needs? A strategy for actions needs to take the whole community system into account. People will use the understanding of how they have development and how

	Strategies for Taking Action	their community works to design strategies
Phase 4	Community Action Plan	When all the pieces are pulled together, community will have a plan that can serve to guide the planning processes within their community.
Phase 5	Preparation of Micro Projects	The main result of a Community Action Planning workshop should be the decisions to meet one or more of community's needs through the preparation of micro – projects. A micro-project is simply a specific activity designed to solve a specific problem/need.
Phase 6	Monitoring	Tracking and Reporting Progress – How is it working and what can we learn?

Source: UNDP, 2013

CASE STUDY

Notun Bridge Bastuhara

Notun Bridge Bastuhara CDC has the accreditation no. of 151. There are total 293 families in the settlement and total population is around 1700. The total cost of the work is taka 1,082,556.00. The total number of children is 475 and the number of women is 594 get benefited from this project. Before the formulation of Notun Bridge Bastuhara CDC the condition of this area was beyond description. There were inadequacy of water supply, no provision for hygienic latrine, unemployment problems and also many domestic violence of which women were the victims. But after the implementation of the project the scenario of this community has been changed greatly. The community people have become more self-reliant than before. As the development of CDC is done through the contract Bastuhara CDC has submitted its second contract. There are two main sectors of this project named Settlement Improvement Fund (SIF) and Socio Economic Fund (SEF). For the implementation of this contract their financial support is needed. There are two accounts of CDC which are supported by the community itself. One is Savings which is contributed by all the households of the community and the amount is 50 tk per month and other is Operation and Maintenance (O&M) fund which money is given by the beneficiary group of the community and it is 10 percent of SIF. The total amount of money in these accounts are given in the below.

Table 2: The account and their savings of Notun Bridge Bastuhara

Account name	Total amount (BDT)
Savings	147550
O & M Fund	148000

Source: UPPRP, 2013

The total scenario of the community has been changed after the implementation of the contracts.

Rajakhali Beribadh

The accreditation no. of the settlement Rajakhali Beribadh is 150. There are total 319 families in this settlement which received the benefits from this project. The total population is around 1850 of this community. The total cost of the work is taka 708, 065.00. Total 423 children and 719 women get benefited from this project. Rajakhali Beribadh is also a developed CDC like Bastuhara. The problem of unhygienic latrine, inadequacy of water supply, unemployment problem, illiteracy problem, Women empowerment have been solved to a great extent. The community people are now capable to live an improved life. As Bastuhara Rajakhali Beribadh has bank account to support the implementation of the contract.

Table 3: The account and their savings of Rajakhali Beribadh

Account name	Total amount (BDT)
Savings	125000
O & M Fund	112500

Source: UPPRP, 2013

Before the implementation of CAP, there were a number of problems in this area. Though all the problems are not solved, the scenario has been changed after the implementation of the contracts.

Shanti Colony

The accreditation no. of this settlement is 212. There are total 274 families in this settlement and total population is around 1820. The total cost of the work is taka 550, 296.00. All of the families of this CDC benefitted from this work. Total 466 children and 702 women are the beneficiaries of this work. Shanti Colony is a newly formulated CDC. So its improvement and fund is lower than previous two. Shanti Colony is a poor community near Kornafully river. It is a low lying area without adequate drainage facility. The condition of Shanti Colony is poorest among the three CDC. AS it is a new CDC it will take time to be developed like the two CDC. The problems of this CDC is also similar to other CDC. The procedure of development is also same. There are two accounts of CDC which are supported by the community itself. The total amount of money in these accounts are given in the below.

Table 4: The account and their savings of the community

Account name	Total amount (BDT)
Savings	55300
O & M Fund	50455

Source: UPPRP, 2013

The community people of Shanti colony is still suffering from different problems. From the first contract they got hygienic latrine and tube well which were essential for their day to day life. But there are also many infrastructures needed for proper development of the area.

FINDINGS

Notuna Bridge Bastuhara, Rajakhali Beribadh and Shanti colony these are the poor community in different condition. Two are mostly developed and another one is under implementation. The major findings are:

Table 4: The findings of the study

Notun Bridge Bastuhara	Rajakhali Beribadh	Shanti colony
• There are 60 twin pit latrine used by two or three families which reduces the tendency of open defecation. • There is a water reservoir to mitigate the shortage of drinking water. • The community also has solar light system which is helpful for saving their expenses of electricity. • Bastuhara has mentioned of constructing pacca drainage in their second contract. • There is a primary school for	• In Rajakhali Beribadh there are also 60 latrines prepared by the UPPR project. • Beside tube well the community people also get the facility to use water from the reservoir. • The community people of Rajakhali also have the privilege of solar light. • Rajakhali has pacca drainage system which reduces sufferings during rainy season due to water logging. • There is a primary school for	• As Shanti colony is in the initial stage there are 15 latrines which are used by the community people. • There is a deep set tube well with a reservoir is provided in this area. • In Shanti colony there is also solar light system. • The drainage system of Shanti colony is in a very poor condition. • There is no provision for primary school.

the community children in two shifts.	the community children in two shifts.	

CONCLUSION

Bangladesh is a small country with a huge population. It has limited resources to serve its population. Poverty is a part of day to day life of the people of Bangladesh. UPPR project has come out with such solutions which can reduce poverty. The main theme of this project is changing the condition of the poor people by their own involvement. People of the poor community under this project are very conscious about their savings. They are now realize what life is and how to make it beautiful.

ACKNOWLEDGMENT

We express our profound gratefulness and gratitude to our respective teachers Muhammad Rashidul Hasan, Associate professor; Gulshana Rashid Lopa, Asssiatanat professor; A.T.M. Shahjahan, Lecturer and officials of UPPR project in Chittagong for their constant guidance, encouragement and valuable suggestion to prepare this report. Also we express our gratefulness to our friends Adrita Alam, Masuma Begum, Tandra Das, Rumky Dasgupta and Fatema Azim Chowdhury for their supporting hand to complete the report.

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ID: SEM 040

PROSPECT OF SOLAR ENERGY AS THE SOLUTION OF EXISTING POWER CRISIS ON COASTAL AREA OF BANGLADESH: A STUDY FOCUS ON RANGABALI UPAZILA

Md. Shafiqul Islam[1*], Tania Yeasmin[1], Md. Hamidul Islam[2], Md. Sohel Mir[3], Irteja Hasan[3] & Md. Tanzilur Rahman[3]

[1]*Dept. of Geography and Environment, Dhaka University, Dhaka-1000, Bangladesh.*
[2] *Dept. of Building Engineering & Construction Management, KUET, Khulna-9203, Bangladesh.*
[3] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <islamshafiquldm@gmail.com>*

ABSTRACT

This paper focuses on the prospect of solar energy to meet the power demand on coastal area of Bangladesh. It also investigates into the current scenario of supply and demand of solar power in the south-western coastal Rangabali upazila where more than 80% people are deprived from national grid connection. The entire study was conducted straightly in the field visit, questionnaire survey, Focus Group Discussion (FGD), Bangladesh Bureau of Statistics (BBS) and from relevant articles. The study finds that the people of this upazila rely on solar power for electrification in their houses, market and so on. 63% people are using solar energy in their households to meeting power crisis in this area. Rests of the people are using generator, kerosene oil, and candle for electrification. The application of solar technology for rural electrification is indirectly increasing the income as well as the living standard of the rural poor as lessen poverty, energy shortage and climate change at remote settlements of Bangladesh as like this coastal area. For this remote area, solar energy can be an environment friendly source of power and can play a significant role in reducing existing power crisis as well as harnessing the rural development. It is also cheap and poor people in this upazila can easily afford it. Government and private sector should concentrate to harness the immense potential of solar energy to empower this community by providing them more accessibility of solar power for increasing its users to rural development.

Keywords: Solar energy, power crisis, solution, Rangabali Upazila and Rural development

1. INTRODUCTION

Developing countries like Bangladesh is the major victim of power crisis problem. To lessen this power crisis renewable energy plays a greater role. Solar energy is one of them. Because the amount of energy we get from the sun is enormous. The average bright sunshine period in Bangladesh in the dry season is about 7.6 hours a day, and that in the monsoon season is about 4.7 hours. These are very good statistics when compared to the 8 hours of daylight in Spain which produced 4 GW of energy covering 2.7% of national demand by the end of 2010. Moreover Germany produces 18 GW of energy which is 2% of their national demand with only half the solar radiation received by Bangladesh (Khan S H et al., 2012). But presently only 53% of total population in Bangladesh has access to electricity and per capita generation being 265 kWh is very low compared to some other developing countries (Hamid, 2013). A solar PV system is a significant emerging option to supply electricity with quality light, reliable service, and long-term sustainability. This system not only would provide reliable, clean, and environmentally friendly energy but also could create employment opportunities in the vicinity of its operation (Ahammed F et al., 2011). In the perspective of Bangladesh several NGOs like IDCOL, Grameen Shakti, Rahim Afrooz, BRAC, CCDR foundations are working to progress our electricity sector with renewable energy sources (Ahmed M S et al., 2012). This study was conducted in the coastal Rangabali Upazila of Bangladesh. It is located in the southern part of this country where most of the areas are depriving national grid electricity connection. Bangladesh has 19 coastal districts out of 64 are in the coastal zone covering a total of 147 upazilas of the country (Sarwar, 2005). Maximum remote coastal districts are beyond the service of national grid connection. The main obstacle is high initial costs. Lack of demonstration of the technology, limited awareness, and ambiguity over after-sales service are the other obstacles in the improvement of solar energy based electricity. The objectives of the study as follows; (a) To explore the present scenario of supply and demand of solar energy in the study area (b) to identify the overall change of socio-economic condition by the blessing of solar technology in the study area.

METHODS APPLIED

Study Site

The study was conducted in very remote area which known as a coastal area situated in south part of Bangladesh. The name of the selected location is Chitolbunia and Char Montaz Union under Rangabali upazila of Patuakhali District. Rangabali Upazila was formed on 14 March 2011 comprising part of Galachipa Upazila. Rangabali Upazila area 720.76 sq km, located in between 21°46´ and 22°05´ north latitudes and in between 91°15´ and 90°37´ east longitudes. It is bounded by Amtali and Galachipa upazilas on the north, Bay of Bengal on the south, Galachipa and char fasson upazilas on the east, Amtali upazila on the west (Banglapedia).

Approach: Mixed method and cross-sectional.

Study population: Household members including male and female in different ages.

Data collection tools and techniques: Methods for this study was conducted both primary and secondary way. Quantitative and qualitative survey with the household members, shopkeepers, farmers, students, doctors was conducted through semi-structured questionnaire. Focus Group Discussions (FGD), Key Informants Interview (KII), site observations were also applied for successful completion of this study. Also various data about the study was collected from Bangladesh Bureau of Statistics (BBS) 2011, Banglapedia, Upazila, union office and from various published relevant articles.

Sample size and sampling: A total of 100 peoples including household members, shopkeepers, farmers, students, doctors were interviewed from two unions of this upazila who were randomly selected. Also 10 FGD_S with selected all respondents from various group of people, 2 KII_S and 5 observation session were carried out.

Map Collection: Map was collected from Banglapedia website for indicating the study site.

Collection of Photographs: Lot of photographs was also needed to illustrate the present status of solar energy of the study area. These photographs have been collected directly from selected two unions of this upazila.

Study Period: July to December, 2013.

2. RESULTS AND DISCUSSIONS

2.1 Status of solar energy in the study area

The study area is situated in the southern coastal area of Bangladesh. A little portion area of here is connected with national grid electricity because of poor communication and less concern of authority. As a result people of this area have to depend on solar energy for mitigating their power crisis.

2.1.1 Pattern of past energy use

The people of the study area had used different types of energy for electrification to their houses, shops, pharmacy, poultry farm and so on. Those past energy types included as (a) car battery (b) dry cell battery (c) kerosene (d) grid electricity with a very few percent (e) biogas etc. Past energy use distribution in the study area with percentage is shown in Figure 1.

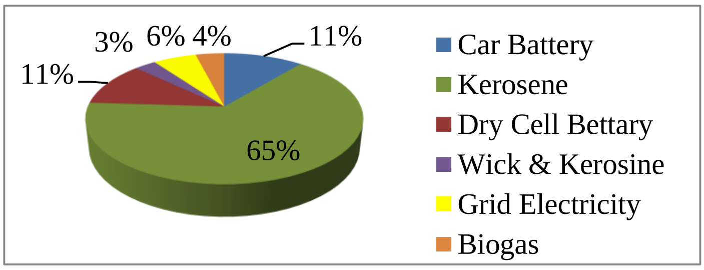

Figure 1: Past energy use distribution

The survey results indicate that maximum people of the study area had used kerosene as their main energy source. 65% people used kerosene in the study area for lighting their house, studying children at night, shops and also for another household work where only 6% people get benefit from the national grid electricity. Besides that, car battery, dry cell battery, wick & kerosene and biogas are used 11%, 11%, 3%, 4% respectively.

Figure 2: Traditional energy system

Above picture (Figure 2) was taken from the study area which indicates the traditional energy use.

2.1.2 Present system of energy use

The present pattern of energy use was found from the study are as (a) solar energy (b) grid electricity (c) car battery (d) biogas (e) kerosene . The present energy use distribution in the study area is depicted in Figure 3.

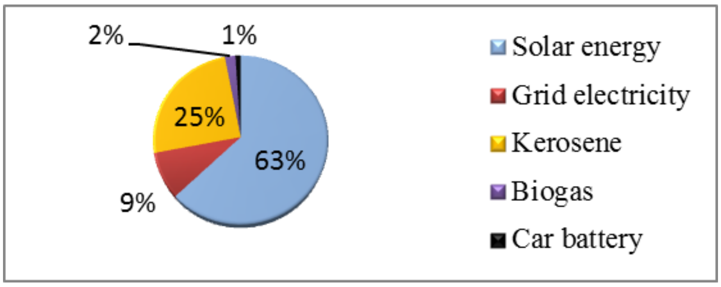

Figure 3: Present energy use distribution

At present people of the study area are using different types of energy which is little bit different than before energy use pattern. Most of the people are using solar as the solution of energy crisis. The study found that 63% people using solar energy in their houses, shops, mosques, poultry farm, pharmacy and so on. The survey results also indicate that 9%, 25%, 2% and 1% people are using grid electricity, kerosene, biogas and car battery correspondingly.

Figure 4: Solar energy use in houses in the study area

The above picture was captured from the study area which shows the solar home system (SHS) using in the houses. Different organizations are working for installation of solar energy in the study area as (a) Risda (b) Grameen Shakti (c) Srijony Bangladesh (d) Sirda (e) IDCOL (f) BRAC (g) Rahimafroj Solar.

2.1.3 Comparison between past and present energy use

It is seen that some dissimilar was found from the study about the energy use system comparing with present and past energy use pattern of the study area. People had been used car battery, dry cell battery, biogas, kerosene for meeting the energy crisis but now people are using solar energy instead of that energy. The previous energy use components are decreasing day by day. Most uses energy was kerosene which is lessening day by day because of rapid development of solar technology. From the previous and present energy use status revealed that the use of kerosene has reduced in 37% and on the other hand 60% increased the use of solar energy. People views on solar energy use are shown in Figure 5.

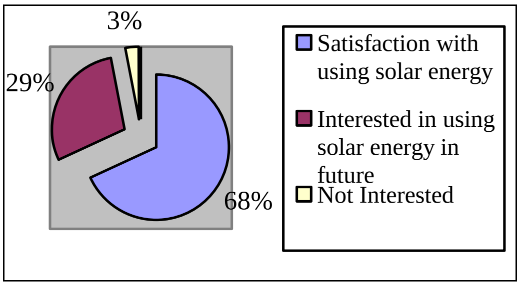

Figure 5: Respondents' opinions about satisfaction with solar energy and tendency toward using it (n=100)

From the above mentioned figure it is clear that 68% people have expressed satisfaction with using solar energy. On the other hand, only 3% people show non-satisfactory opinion about solar energy. One of the important points noted that 29% people interested in using solar energy in future.

2.2 Existing solar energy scenario

Present study shows that solar energy uses in different sectors in the study areas as (a) households (b) educational institutions (c) union parisad (d) community center (e) religious house includes mosque, temple (f) hospital (g) shops (h) pharmacy and so on. Solar energy uses in different sectors with number and percentage in thy study area is presented in Table 1.

Table 1: Solar energy uses in the selected study area

Sectors	**Sub sectors**	**Number**	**Solar energy**	**Solar energy use as**

			uses	percentages
Families		4983	2483	49.83
Education	Primary schools	21	12	57.14
	High schools	4	2	50
	Madrasaha	3	1	33.33
	Collages	1	0	N/A
Union parisad		2	2	100
Community centers		8	6	75
Religious	Mosques	41	26	63.41
	Temples	8	3	37.5
Hospitals		3	2	66.67
Shops		1211	545	45.00
Pharmacy		571	231	40.46

Source: Authors calculation based on field survey, 2013

2.3 Socio-economic impact of solar energy

The solar energy makes a significant impact on rural community in the study area from socio-economic viewpoint.

2.3.1 Lighting Facilities Before and After SHS

The people of the study area are using some sort of kerosene lamps for their household lighting. They use dry cell batteries for torch lights and radios. For watching Television people usually used car batteries. After the solar home system (SHS) installation, the scenario altered. Nobody was using kerosene pressure lamps and car batteries for lighting and entertainment purposes.

2.3.2 Income Generation Activities

Income generation activities are increased after acquiring the solar home systems in the study unions. The people involved in doing business using traditional fuel now switch to solar light that outcomes in more progress of their business than before. Many people established mobile phone service center due to installation of solar home system. Grocery shops, Pharmacies owners who were using kerosene lamps for their business get working hours extended due to solar home system. Women become engaged with income generation activities such as tailoring machine, handicraft works and so on.

Table 2: Income Generation Activity Hours Before and After the SHS

Income Generation Activities	Number of Hours		
	Before SHS	**After SHS**	**Change**

Pharmacy	1	4	+ 3
Grocery shop (Used Kerosene lamp)	1	4	+ 3
Grocery shop (Used Pressure Lamp)	2	4	+ 2
Mobile phone business	0	10	+ 10
Tailoring (House)	2	5	+ 3
Tailoring (Shop)	4	6	+ 2
Sewing	0	1	+ 1

Source: Authors calculation based on field survey, 2013

From the above table it is clear that the hours of various income generation activities are increased after installation of solar home system.

2.3.3 Solar Energy to Empower Women

Women are one of the focal victims of the energy crisis. With the access to solar electricity their live is converted. Women no longer have to suffer unsafe kerosene fumes. They can complete their household duties more easily and in less time by not having to clean kerosene lamps. By using chargeable carry along lights, women can also enjoy more security and augmented mobility. Many have transmuted their homes into income generating centers by opening small business such as poultry farming and handicrafts. Solar energy helps to bringing them dignity in the society.

2.3.4 Impact on Literacy and Education

Solar energy has greater impact on literacy and education by providing additional facility to extended study period of children. Children become much more interested in study after the SHS installation than before SHS. Many schools are using solar light to extend the education services for working students and adults to the evening hours. If these processes continue then the illiteracy rate will be decreased.

2.3.5 Impact on Household Assets

Radio, Television and cassette, mobile phone uses increases after the solar system installation. The telecommunication system was upgraded pointedly due to solar home system. In the study area, people could communicate with their near and dear simply. The respondents could know the news of the country by watching TV and listening to radio.

Table 3: User number percentage about household assets before and after SHS

Household Asset	User number percentage Before SHS		Increased user number by Percentage (%) after SHS	
	Study area		Study area	
	Chitolbunia Union (n=50)	Char Montaz Union (n=50)	Chitolbunia Union (n=50)	Char Montaz (n=50)
Radio	40%	44%	67%	74%
Television	24%	27%	59%	69%
Mobile Phone	42%	48%	77%	81%

Source: Authors calculation based on field survey, 2013

2.4 Linkage between MDGs and Solar Photovoltaic

Solar PV systems can be efficaciously utilized for world poverty reduction. The Millennium development Goals (MDGs) include the lessening of poverty, hunger, disease, illiteracy, environmental degradation, and gender discrimination by solar technology.

3. HINDERS THE GROWTH OF SOLAR ENERGY IN THE STUDY AREA

There are several problems with solar energy in this area as (a) lack of awareness (b) installment and repairing cost high (c) people poor socio-economic status (d) Poor road network (e) site unsuitability (f) low microcredit system etc.

4. RECOMMENDATIONS

Following are the recommendations that might be applied for smooth progress of rural development over solar energy in the study area as (a) Provide training to the user (b) Facilitate microcredit system (c) need for planning integration between rural electrification authorities, ministries and transmission and distribution system operators.

5. CONCLUSION

It is concluded that the study area is lacking behind the supply of power from national grid connection. As a result, solar technology is increasing here day by day. The current scenario of solar energy has been presented with necessary data obtained from field survey. It is found that the present energy use patterns are different than past pattern of energy use. The use of kerosene oil, car battery for lighting houses, shops, poultry farm, pharmacies etc. replaced by solar PV technology. The study results show solar technology enhancing the income generation activities, empowering women as well as bringing them dignity, changing the lifestyle into modern, increasing literacy and education quality, reducing poverty and so on in the study area. Low Economic status and little knowledge about installation and benefit of solar technology to the people are the main obstacles for growth solar energy here. It should be build awareness, facilitate the microcredit for improving solar technology as developing their socio-economic condition in the study area.

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Banglapedia.

ID: SEM 041

A STUDY ON THE STRENGTH CHARACTERISTICS OF STEEL FIBER REINFORCED CONCRETE

Nafisa Tabassum[1*], Jonayed Kaiser Sourav[1], Md. Saiful Islam[1] & Md. Moinul Islam[1]

[1] *Department of Civil Engineering, CUET, Bangladesh.*
**Corresponding Author, <nafisa058@yahoo.com>*

ABSTRACT

Although concrete is a widely used construction material, it has major disadvantages such as low tensile strength, low strength to weight ratio, brittle nature, liable to cracking etc. In recent decades, steel fibers have gained popularity for use in concrete due to its certain special features. Steel fiber reinforced concrete is a composite material consisting of cement, fine aggregate, coarse aggregate, water and a small amount of randomly distributed steel fibers. Inclusion of steel fibers can overcome the brittle failure of plain concrete and principle reason for incorporating fibers into concrete is to improve the properties of concrete such as toughness, compressive strength, tensile strength, flexural strength, shock resistance and fatigue resistance. Steel fibers act as a crack arrestor restricting the development of cracks and thus transforming an inherently brittle matrix (concrete) into a strong composite with superior crack resistance, improved ductility and distinct post cracking behaviour. This paper presents an experimental programme carried out to investigate the strength characteristics of concrete with or without steel fibers. A total of 162 nos. 100 mm cubical concrete specimens were cast from M25 concrete with varying volume fraction (0.5, 1.0, 1.5 & 2.0 %) and two different aspect ratios (29.40 & 44.10). Both compressive as well as split tensile strength test of the specimens were carried out after specific curing periods. Required nos. of plain concrete specimens were also cast from the same concrete mix to compare the strength characteristics of both plain and fiber reinforced concrete and to predict the effect of fiber inclusion. The test results showed significant improvement of strength for fiber reinforced concrete as compared to plain concrete. The inclusion of fiber showed better improvement of tensile strength as compared to compression strength.

Keywords: Durability, Fiber Reinforce Concrete, Compressive Strength, Tensile Strength.

INTRODUCTION

Fiber reinforced concrete (FRC) is cement concrete reinforced with more or less randomly distributed fibers. In FRC, thousands of small fibers are dispersed and distributed randomly in the concrete during mixing, and thus improve concrete properties in all directions (Rana, A., 2013). Fiber is a small piece of reinforcing material possessing certain characteristics properties. They can be circular, triangular or flat in cross-section. The fiber is often described by a convenient parameter called ―aspect ratio which is the ratio of its length to its diameter (Rana, A., 2013).

Concrete in general has a higher brittleness with increase in strength. This is a major drawback since brittleness can cause sudden and catastrophic failure, especially in structures which are subjected to earthquake, blast or suddenly applied loads i.e., impact (Kandasamy and Murugesan, 2011).
This serious disadvantage of concrete can at least partially be overcome by the incorporation of fibers, especially, steel. The incorporation of fiber can cause a change in the failure mode under compressive deformation from brittle to pseudo-ductile, thereby imparting a degree of toughness to concrete (Kandasamy and Murugesan, 2011)

It has been successfully used in construction with its excellent flexural-tensile strength, resistance to spitting, impact resistance and excellent permeability and frost resistance (Rana, A., 2013).

FRC composite properties, such as crack resistance, reinforcement and increase in toughness are dependent on the mechanical properties of the fiber, bonding properties of the fiber and matrix, as well as the quantity and distribution within the matrix of the fibers (Rana, A., 2013).

When the fiber reinforcement is in the form of short discrete fibers, they act effectively as rigid inclusions in the concrete matrix. Physically, they have thus the same order of magnitude as aggregate inclusions; steel fiber reinforcement cannot therefore be regarded as a direct replacement of longitudinal reinforcement in reinforced and prestressed structural members. However, because of the inherent material properties of fiber concrete, the presence of fibers in the body of the concrete or the provision of a tensile skin of fiber concrete can be expected to improve the resistance of conventionally reinforced structural members to cracking, deflection and other serviceability conditions.

EFFECTS OF STEEL FIBERS IN CONCRETE

Fibers are usually used in concrete to control cracking due to both plastic shrinkage and drying shrinkage. They also reduce the permeability of concrete and thus reduce bleeding of water. Some types of fibers produced greater impact, abrasion and shatter resistance in concrete. Generally fibers do not increase the flexural strength of concrete and so cannot replace moment resisting or structural steel reinforcement. Indeed, some fibers actually reduce the strength of concrete (Amit Rana, 2013). The amount of fibers added to the concrete mix is expressed as a percentage of total volume of the composite (concrete and fibers), termed volume fraction (V_f). V_f typically ranges from 0.1 to 3%. Aspect ratio (l/d) is calculated by dividing fiber length (l) by its diameter (d).For fibers with a non circular cross section, an equivalent diameter can be used for the calculation of aspect ratio. If the modulus of elasticity of the fiber is higher than the matrix (concrete or mortar binder), they help to carry the load by increasing the tensile strength of the material. Increase in the aspect ratio of the fiber usually segments the flexural strength and the toughness of the matrix. However, fibers which are too long tend to ‘ball’ in the mix and create workability problems. Some recent research indicated that using fibers in concrete has limited effect on the impact resistance of the materials (Amit Rana, 2013). This finding is very important since traditionally, people think that the ductility increases when concrete is reinforced with fibers. The results also indicated out that the use of micro fibers offers better impact resistance compared with the longer fibers (Rana. A., 2013).

EXPERIMENTAL PROGRAMME:

Portland composite cement (Tiger brand) was used for making concrete specimen.19 mm nominal size stone chips (F.M. =7.5) as coarse aggregate and local sand (F.M. =1.55) as fine aggregate was used in concrete mix. Locally available thin steel wires (0.68mm diameter) were cut into various lengths as per two different aspect ratios (29.40 & 44.10).

The constituent materials of concrete, viz., cement, sand and aggregates were tested as per requirement of Mix design according to ACI 211.1-91.Concrete of M25 grade was designed (Mix ratio: 1:1.69:2.39; w/c = 0.49) for casting the specimen as per plan of the study. A total of 162 nos. 100 mm cubical concrete specimens were cast with varying volume fraction (0.5, 1.0, 1.5 & 2.0 %) and two different aspect ratios (29.40 & 44.10).

The molds were filled with concrete mix and vibration was given to the molds using table vibrator. The top surface of the cube was finished as smooth by trowel. Demoulding was done after 24 hours and the specimens were cured under normal water. After specific curing periods i.e. 7day, 28day and 60 day, the specimens were removed from curing tank for testing.

Two types of strength tests i.e. Compressive and Split tensile strength were performed to access the performance of SFRC. A total of 81 cube specimens were tested for compression strength and another 81 specimens for split tensile strength. The test results were presented in tabular and graphical forms.

RESULTS AND DISCUSSIONS

COMPRESSIVE & TENSILE STRENGTH TEST RESULTS

The 7, 28 and 60 days cube compressive and split tensile strength of plain and SFRC specimens obtained from tests are presented in Table-1.

Table-1: compressive and tensile strength of plain and SFRC at various curing ages

Aspect Ratio	Volume	Compressive Strength (Mpa)			Tensile Strength (Mpa)		
		7 Days	28 Days	60 Days	7 Days	28 Days	60 Days
0	0	22.66	28.36	30.4	2.07	3.11	4.12
29.4	0.5	23.12	29.02	31.8	2.41	3.41	4.25
29.4	1	26.53	32.72	34.6	3.24	4.4	5.2
29.4	1.5	23.8	30.1	32.1	2.67	3.68	4.37
29.4	2	26.03	31.3	33.33	2.52	3.51	4.29
44.1	0.5	23.41	29.84	31.95	2.52	3.58	4.39
44.1	1	26.69	34.1	36.18	3.77	4.88	5.88
44.1	1.5	24.46	31.65	33.7	2.74	3.97	4.62
44.1	2	25.1	32.68	34.36	2.64	3.81	4.47

Plate-1& 2 shows the failure/cracking pattern of specimens under compression and Plate-3 & 4 shows the same for tension specimens. For compressive strength, it is seen that formation of crack is higher in plain concrete specimen (Plate-1) than SFRC specimen (Plate-2). When tested for split tensile strength, the plain concrete specimen broken into two pieces (Plate-3) .The SFRC specimen retains the geometry integrity, indicating improved ductility of SFRC due to the addition of fibers over control concrete (Plate-4).

Plate-13: Compression test of plain concrete cube specimen

Plate-14: Compression test of SFRC cube specimen

Plate-3: Split tensile strength test of plain concrete cube specimen

Plate-4: Split tensile strength test of SFRC cube specimen

Fig 1-4 shows the variation of concrete compressive strength of SFRC specimens with time for various steel fiber contents and aspect ratios. Also, the compressive strength values obtained after different curing ages at various fiber contents and aspect ratios are shown in Table -1. From Fig. 1 and Fig. 2, it is observed that the compressive strength increases from 30.4 MPa with 0% fiber content to a maximum of 36.18 MPa with 1% fiber content and then starts decreasing with an increase in fiber content . Fig. 3 & Fig. 4 demonstrate that at each curing age, i.e. (7, 28 and 60 days) the strength is maximum for 1% fiber content for both aspect ratios.

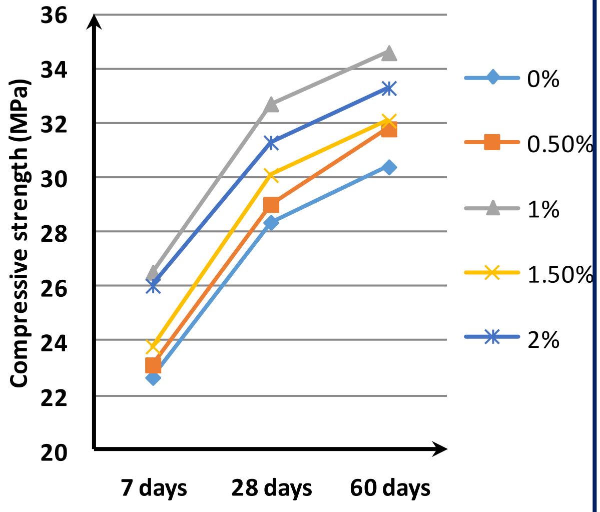

Fig. 1: Variation of compressive strength with time for steel fibers of different volume fractions (Aspect ratios= 29.40)

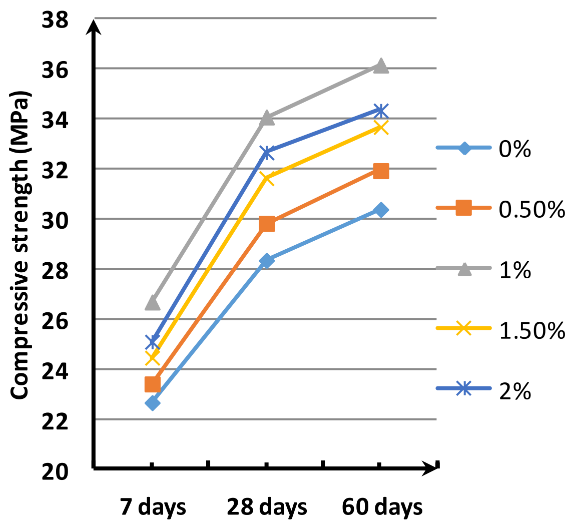

Fig. 2: Variation of compressive strength with time for steel fibers of different volume fractions (Aspect ratio=44.10)

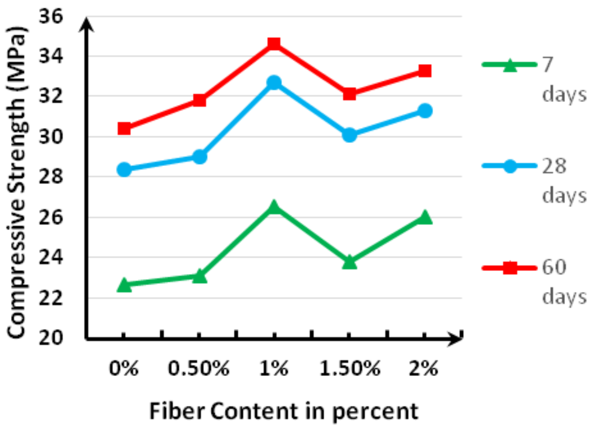

Figure 3: Variation of compressive strength of concrete with different percentages of steel fiber for aspect ratio=29.40

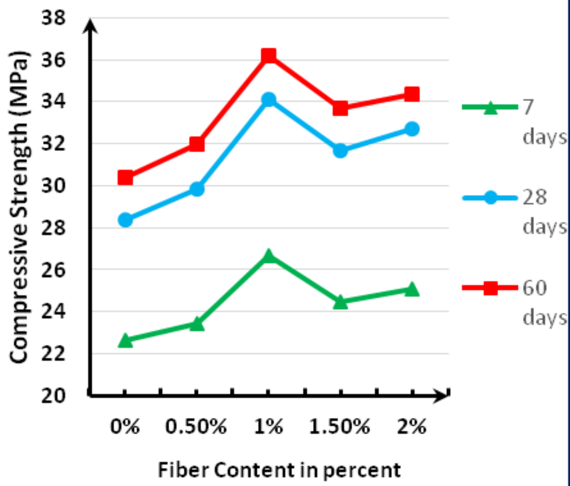

Fig. 4: Variation of compressive strength of concrete with different percentages of steel fibers for aspect ratio=44.10

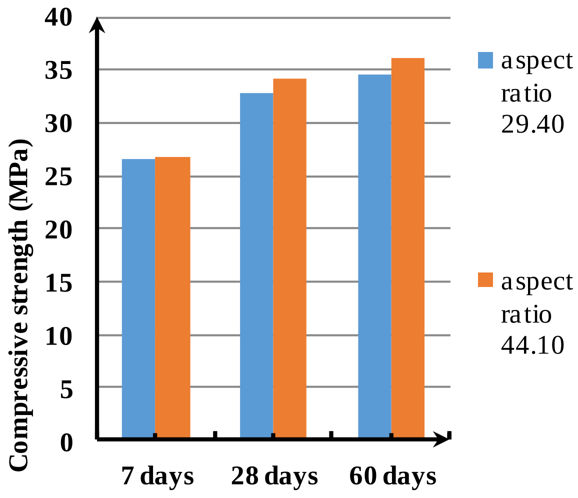

Fig. 5: Variation of compressive strength with time for steel fibers (1%) for different aspect ratios

The compressive strength values obtained for fibers with 29.40 and 44.10 aspect ratios for 1% fiber content are 34.6 MPa and 36.18 MPa respectively. The compressive strength values obtained after 7, 28 and 60 days curing with fibers of two aspect ratios are shown in Table-1 and the variation is presented in Fig. 5. It is observed that at each curing age, the SFRC specimens having higher aspect ratios show higher strength as compared to specimens with lower aspect ratio.

Fig 6-9 shows the variation of concrete split tensile strength of SFRC specimens with time for various steel fiber contents and aspect ratios. From Fig. 6 and Fig. 7, it is observed that the split tensile strength increases from 4.12 MPa with 0% fiber content to a maximum of 5.88 MPa with 1.0% fiber content and then again decreases with an increase in fiber content.

From Fig. 8 & Fig. 9, it is found that at each curing age, i.e. (7, 28 and 60 days) the strength is maximum for 1% fiber content for both aspect ratios.

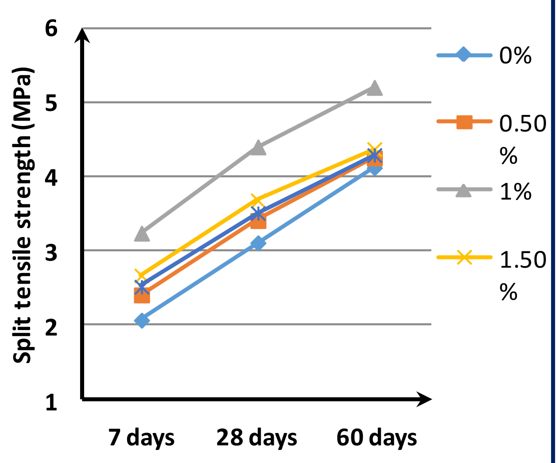

Fig 6: Variation of split tensile strength with time for steel fibers (aspect ratio=29.40) of different volume fractions

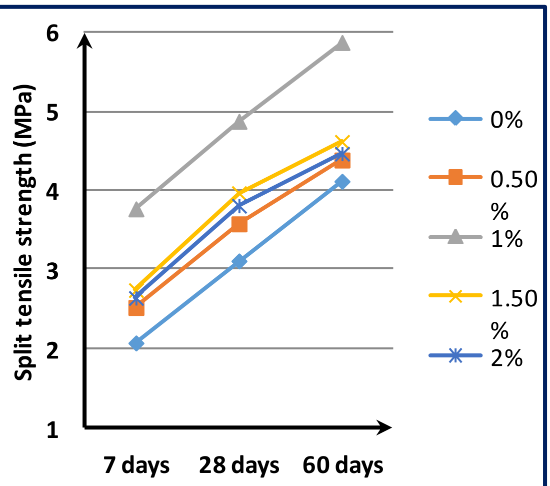

Figure 7: Variation of split tensile strength with time for steel fibers (aspect ratio=44.10) of different volume fractions.

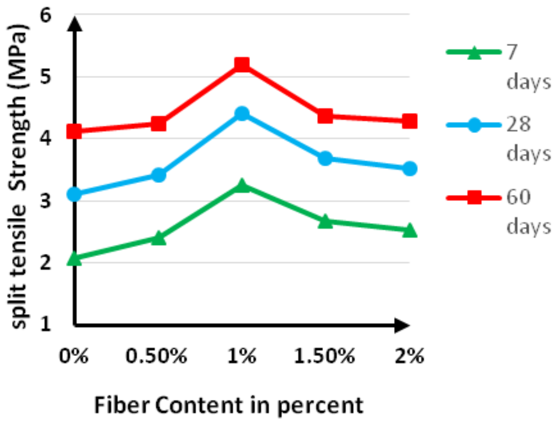

Fig. 8: Variation of split tensile strength of concrete with different percentages of steel fibers for aspect ratio = 29.40

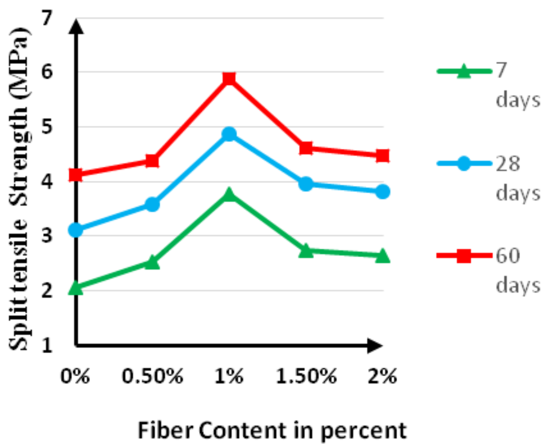

Fig. 9: Variation of split tensile strength of concrete with different percentages of steel fibers for aspect ratio = 44.10

The split tensile strength values obtained for fibers with 29.40 and 44.10 aspect ratios for 1% fiber content are 5.2 MPa and 5.88 MPa respectively. The split tensile strength values after 7, 28 and 60 days curing with fiber type of two aspect ratios are shown in Table-1 and the variation is presented in Fig. 10. It is observed that at each curing age, the SFRC specimens having higher aspect ratios show higher strength as compared to specimens with lower aspect ratio.

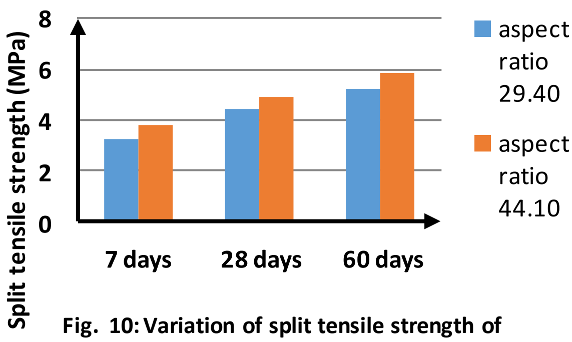

Fig. 10: Variation of split tensile strength of SFRC with time at 1% steel fiber content for different aspect ratios

CONCLUSION

1) Based on the limited numbers of variables studied regarding the strength performance of steel fiber reinforced concrete, the following conclusions can be drawn:

2) Both the compressive & tensile strength of concrete are observed to increase with the inclusion of steel fiber. The improvement depends on the fiber volume fractions and its aspect ratios.

3) The strength of concrete enhances up to 1% of steel fiber and then decreases gradually with the increase of fiber content. Hence 1% steel fiber content can be considered as optimum fiber content.

4) After 28 days curing, the maximum strength improvements are observed as 20% and 57% for compressive and tensile strength respectively.

5) Both compressive and split tensile strength values of SFRC increase with the increase in aspect ratio of fiber. In case of compressive strength, the maximum increase in strength at 1% fiber content was observed as 4.4% when aspect ratio changes from 29.40 to 44.10. In case of split tensile strength, the corresponding value was represented as 12%.

6) The cracking/failure pattern of the SFRC specimens indicated improved ductility over plain concrete. Also inclusion of fibers in concrete are observed to improve its tensile strength more effectively as compared to compressive strength.

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SUITABILITY AND COMPARATIVE ANALYSIS OF LOCAL COASTAL SANDS IN CONCRETE CONSTRUCTION

S. Das[1*], M.A. Mohiuddin[2], K.M. Islam[2], A.M.F. Rashid[2]

[1] *Dept. of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.*

[2] *Lecturer, Dept. of Civil Engineering, Southern University Bangladesh, Chittagong-4000, Bangladesh.*

**Corresponding Author, <sanjoy0701098@gmail.com>*

ABSTRACT

In Bangladesh, most of the construction works prefer using Sylhet sand due to better performance. However, local sands are often used due to ready availability, time and cost minimization. These samples are collected from many local sources and naturally, the sample properties vary in wide range. A comparative analysis of sample properties from the prominent sources may present us the preferable source and suitability according to requirement.

To ascertain the properties of local sands, samples were collected from ten different locations, including a sample from Sylhet. The other sand samples were collected from Dalu at Adhunagar-Lohagara, Shilak at Rangunia, Halda at Fatikchari, Narayanhat, Kharana at Patiya, Shreemai at Patiya, Fashiyakhali at Chakaria, Eidgah at Cox's Bazar and Baraiyarhat at Meersarai. The samples were tested for their size, shape, gradation, compressive strength and salinity. While Sylhet sand shows better properties in almost all aspects, but samples from Fasiyakhali, Kharana, Dalu and Shilak show lesser salinity than Sylhet sand. Based on the tested parameters like compression strength of mortar, salinity level, co-efficient of curvature, uniformity co-efficient and fineness modulus, it is found that the sands from Dalu, Halda, Shilak and Shrimai are of acceptable quality, while the sand from Halda is the best among the local sources considered.

Keywords: Local Coastal sand, Mortar, Comparative Analysis, FM, Gradation, Salinity, Compressive Strength.

INTRODUCTION

Sand is a naturally occurring granular material composed of finely divided rock and mineral particles. The composition of sand is highly variable, depending on the local rock sources and conditions, but the most common constituent of sand in inland continental settings and non-tropical coastal settings is silica (Silicon Dioxide, or SiO_2), usually in the form of quartz**.** Sand, as an engineering material, plays an important part in engineering construction. In concrete work, it is usually termed as fine aggregate. Sand is a form of silica (quartz) and may be of argillaceous, siliceous, or calcareous according to its composition. Natural sands are the weathered and worn out particles of rocks (mainly quartzite) and of various grades or sizes depending upon the intensity of weathering. The sand grains may be of sharp, angular or rounded**.**

Sand is abundantly available all over Bangladesh. Very good variety of river sand is available in the districts of Dhaka, Mymensingh, Sylhet, Savar, Kaliakair, Durgapur (Netrakona), Sunamgonj. Both sea and river sand of good quality are found in coastal districts like Chittagong, Khulna and Noakhali. A variety of coarse sand is abundantly found in the Rivers flowing down from the high hills (part of Arakan Hills) in the districts of Rangamati, Khagrachari, Bandarban, Chittagong and Cox's Bazar. Sand is also available in northern districts of Bangladesh but not of good quality.

Sand is being used as a construction material since long, perhaps dates back to civilization. Usually users use local sand for their work. In coastal area the sands that are being in use may contain many impurities like salinity which is of great importance. In our country Bangladesh, usually there is a common trend of using sand from Sylhet for better quality construction work. But its procuring is costly and time consuming. It incurs high transportation cost and also may be in demand, consequently escalating the price. In this context, the study was targeted to test the suitability of local sands usually found in coastal areas of Chittagong and Cox's Bazar districts and compare various important parameters of these sands to Sylhet sand. In this work the priority will be given to the quality of sands of coastal area by determining their grain size, shape of particles & nature of gradation of the same. Particularly, the compressive strength for sand from different locations/areas of coastal region will be accounted along with the salinity level and fineness modulus of those samples of sands.

SALINITY IN SAND

Sand salinity is the salt content in the sand. Salt is a natural element. Salination can be caused by natural processes such as mineral weathering [**3**]. Over long periods of time, as soil minerals weathered and release salts, these salts are flushed or leached out of the soil. It addition to mineral weathering, salts are also deposited via dust and precipitation. Salts generally found include NaCl (Table salt), $CaCl_2$, $CaSO_4$ (Gypsum), $MgSO_4$ (Magnesium Sulphate), K_2SO_4 (Potassium Sulphate), KCl (Potassium Chloride) and $NaSO_4$ (Sodium Sulphate). The ions responsible for salinity are: Na^+, K^+, Ca^{2+}, Mg^{2+} and Cl^-. The salts are chlorides or sulphates of sodium, potassium, calcium and magnesium that affect construction work [**4, 5**].

MATERIALS

To find out the properties of local sands, samples were collected from ten different locations, including a sample from Sylhet. The other sand samples were collected from Dalu at Adhunagar-Lohagara, Shilak at Rangunia, Halda at Fatikchari, Narayanhat, Kharana at Patiya, Shreemai at Patiya, Fashiyakhali at Chakaria, Eidgah at Cox's Bazar and Baraiyarhat at Meersarai. Ordinary Portland cement was used here.

EXPERIMENTAL PROGRAM

To complete the intended work first of all locations of sands to be collected for testing have been selected with great care. Then sufficient amount of sands were collected from those locations. After drying out the samples of sands in the oven for 24 hours the sand samples were sieved using standard sieves recommended by ASTM guidelines. After the completion of sieving three sand-mortar cubes (2″*2″* 2″) with mixing ratio 1:2.75 have been prepared for each of the sand sample type. Accordingly curing of those sand mortar cubes have been done for the intended time periods (3 days, 10 days & 30 days) for each sand sample type. To find out the compressive strength, the cement mortar blocks were crushed after 3 days, 10 days & 30 days and the salinity level in those sand-mortar cubes have been tested according to standard provision.

RESULTS & DISCUSSION

The experimental program was intended to test the suitability of local sands usually used in coastal areas of Chittagong and Cox's Bazar districts and compare some characteristic parameters of these sands with most favourite Sylhet sand. The results are described below with their graphical interpretation.

Source of Sample	**Fasiyakhali**	**Narayanhat**	**Kharana**	**Eidgah**	**Baraiyarhat**	**Dalu**	**Sylhet**	**Shrimai**	**Halda**	**Shilak**
Fineness Modulus	**1.47**	**1.49**	**1.03**	**1.5**	**0.98**	**1.33**	**2.48**	**1.56**	**1.29**	**1.46**
Uniformity Coefficient, Cu	**1.975**	**2.096**	**1.575**	**1.92**	**1.85**	**1.76**	**3.305**	**2.13**	**1.71**	**1.965**
Co-efficient of	**0.913**	**0.896**	**0.961**	**0.91**	**1.057**	**0.97**	**1.006**	**0.891**	**0.964**	**0.902**

Gradation, Cc										
Nature of Gradation	**Poorly Graded**	**Poorly Graded**	**Poorly Graded**	**Poorly Graded**	**Poorly Graded**	**Poorly Graded**	**Well Graded**	**Poorly Graded**	**Poorly Graded**	**Poorly Graded**
Compressive Strength(MPa)	**12.7875**	**14.725**	**11.275**	**13.95**	**14.275**	**17.825**	**24.025**	**15.5**	**16.6625**	**14.725**
Salinity (% as NaCl)	**0.00461**	**0.00527**	**0.00461**	**0.00527**	**0.00593**	**0.00396**	**0.00527**	**0.00593**	**0.00527**	**0.00461**

Table-01: Comparison of Characteristic Parameters of Sand Samples from various sources

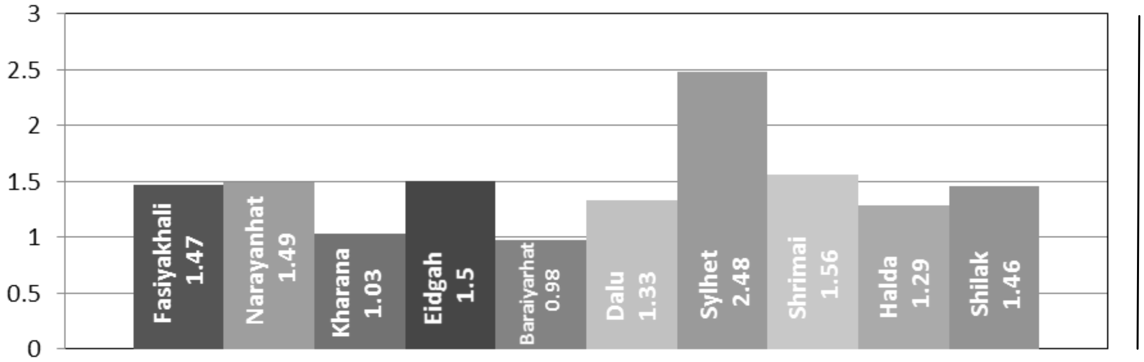

Fig-01: Fineness Modulus of sands from Different Locations

The figure evinces that fineness modulus reached a value of 2.46 for Sylhet sand which is more than 1.5 times the maximum value obtained for other local coastal sands. Among ten different samples other than Sylhet, six of them attained nearly same value of FM. Only Kharana and Baraiyarhat have FM values close to unity.

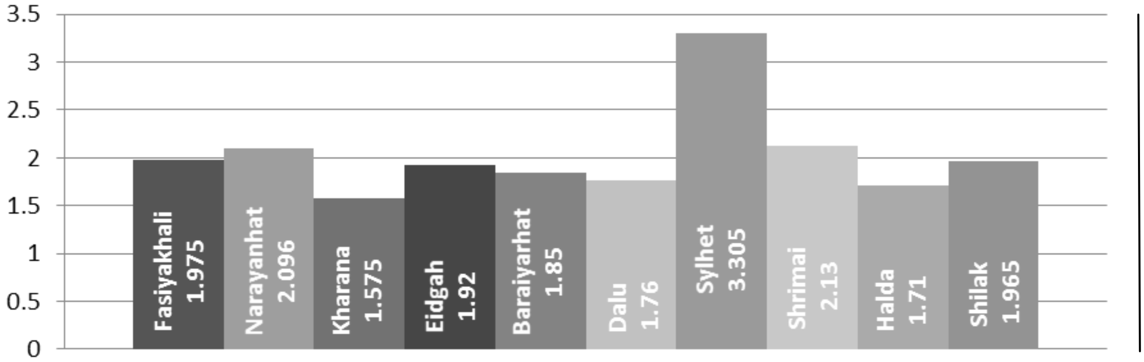

Fig-02: Uniformity coefficient (C_u) of sands from Different Locations

Uniformity coefficient (C_u) curve obtained for various sands depicts the waning of the Cu value for the local coastal sands starting from C_u=2.13 for sand of Shrimai to C_u=1.576 for sand of Kharana. Again the maximum uniformity coefficient is C_u= 3.305 for Sylhet sand.

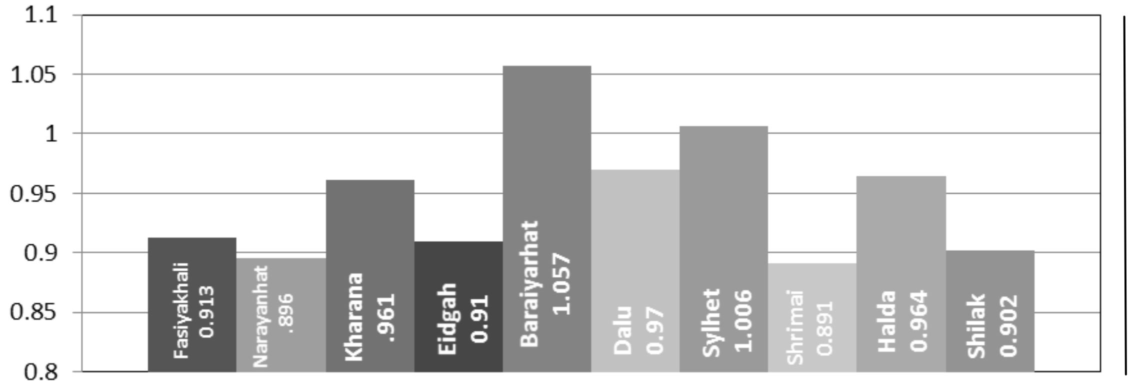

Fig.-03: Co-efficient of Gradation (Cc) of sands from Different Locations

While Sylhet sand has shown rational result than other local sands in case of FM and Cu value, sand of Baraiyarhat attains better Coefficient of Gradation (C_c) value than the Sylhet sand. Though rest of the sand samples shown uniformly declining pattern with a lowest value of C_c=0.891 for sand of Shrimai.

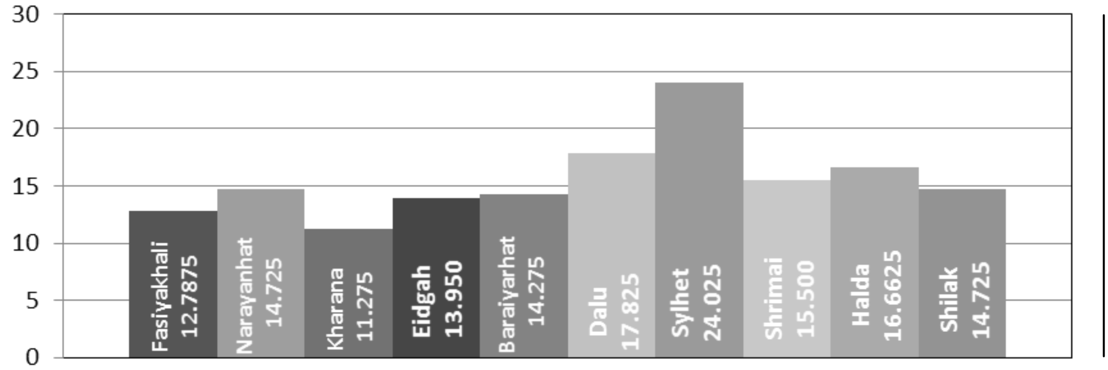

Fig.-04: Compressive strength (MPa) of Mortar from sand's of different Locations

When it comes to the compressive strength of several sands, the sand of Dalu has a mortar compressive strength of 17.825 MPa, which is second largest in the series and Sylhet sand has the maximum compressive strength of 24.025 MPa. It is clear that lowest compressive strength (11.237 MPa) attained by the sand of Baraiyarhat is only 50% of the Sylhet sand.

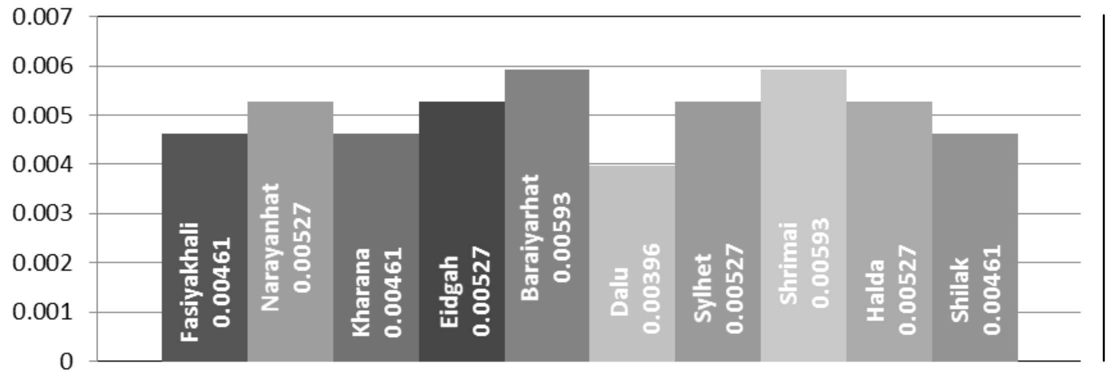

Fig.-05: Salinity (as % of NaCl) of sands from Different Locations

The bar chart in this case is showing the Salinity level in different sands. Sylhet sand though previously depicted very good results for other representative parameters, contained substantial amount of salinity (0.00527) as shown in bar/ column charts though two other sands have salinity greater than this amount.

CONCLUSION

From this project work, we arrived at the following conclusions:

1) From the analysis, it was found that the cube compressive strength of local coastal sands vary from 11.275MPa to 17.825MPa while it was 24.025MPa for Sylhet sand.

2) The Fineness Modulus (FM) of local coastal sand was obtained 1.03 to1.56, while it is 2.48 in the case of Sylhet sand.

3) The Uniformity Co-efficient (C_u) vary from 1.71 to 2.13 for local coastal sand while it is 3.305 for Sylhet sand.

4) The Co-efficient of Gradation (C_c) varies from 0.891 to 0.964 for local sand while it is 1.006 for Sylhet sand.

5) The salinity as % of NaCl ranges from 0.00396 to 0.00593 for local coastal sands while it is 0.00527 for Sylhet sand.

6) Considering the parameters like compression strength, salinity level, co-efficient of gradation, uniformity co-efficient and fineness modulus, it is found that the sands from Dalu, Halda, Shilak, and Shrimai are of acceptable quality available in coastal districts.

7) We were able to find out the better quality local coastal sand that can be favourably used in concrete constructions.

8) The comparative analysis report can be very helpful if further work is done on the same field.

9) It is found that the local sands are easily available in bulk quantity, less costly and can be easily transported to any construction site.

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ECO-FRIENDLY SUSTAINABLE CONCRETE USING RECYCLED MATERIALS: A REVIEW

S. Rana[1*]

[1] *Dept. of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong, Bangladesh,*
**Corresponding Author, <sahel.cuet@gmail.com>*

ABSTRACT

Concept of a sustainable development in the field of engineering offers several possibilities for the utilization of the recycled solid waste materials. A sustainable structure is that for which the total environmental impact during its entire life cycle is minimum. Sustainable design and construction of structures have a small impact on the environment. Concrete, one of the most widely used construction materials, is a sustainable material because it has a very low inherent energy requirement. It is produced with very little wastes, can be made with recycled materials, and is completely recyclable. Use of green materials embodies low energy costs. Their use must have high durability and low maintenance leading to sustainable construction materials. High performance cements and concrete can reduce the amount of cementitious materials and total volume of concrete required. Reuse of post-consumer wastes and industrial by-products in concrete is necessary to produce greener concrete. Use of slag, rice- husk ash, palm oil fuel ash, natural pozzolans, silica fume, and other similar pozzolanic materials can reduce the use of manufactured Portland cement clinker; and, at the same time, produce concrete that is more durable. Greener concrete also improves air quality, minimizes solid wastes, and leads to sustainable structures.

Keywords: sustainable concrete, waste materials, CO_2 emission, recycling, cement replacement.

INTRODUCTION

It is commonly known that the increase of population, the urbanization and industrialization directly affect the increase in consumption of all kinds of materials and energy sources and, therefore, the increase of the amount of solid waste. Generation of solid waste represents one of the most significant problems of our civilization. Statistics show that over 5 billion ton of non-hazardous by-product materials are produced each year in USA. These by-products are from agricultural sources, domestic sources, industrial sources, and materials processing sources. If disposal of a material is an expense, one naturally thinks of ways to turn it into something useful, which would be even better for us. The possible solution of this problem is the philosophy of sustainable development. Sustainable development implies such a developing path which will ensure the use of natural resources and will create assets in a manner to ensure meeting the needs of the present generations without compromising the future generations. Sustainable development is one of the few ever present issues, because it is of great importance for a modern society.

Concrete is one of the most widely used construction materials in the world. In fact, concrete executes outstanding responsibilities for the construction of modern infrastructures, industrialization and urbanization for the growing population. Besides, it is relevant to mention that the concrete industry today is the largest consumer of natural resources- water, sand, gravel, and crushed rock. The production of Portland cement, an essential constituent of concrete, leads to the release of significant amount of CO_2, a greenhouse gas. One ton of Portland cement clinker production creates one ton of CO_2 and other greenhouse gases (GHGs). Not surprisingly, about 7% world's CO_2 is released to atmosphere due to OPC manufacturing alone; in consequence, global warming is rising day by day. The burning of OPC clinker (around 14000^{o}C) is not only liable for CO_2 emissions but also it is costly in terms of fossil fuel usage. For these reasons, sustainable concrete is one of the prime topics in concrete industry all over the world and its main objectives are: reduction of the amount of polluting gases and carbon dioxide (CO_2) emitted during the manufacture of concrete; more efficient use of waste materials; development of a low-energy, long-lasting flexible buildings and structures; exploiting the thermal mass of concrete in a structure to reduce energy demand. Thus, all of these problems, particularly in concrete sector, could be solved or minimized simultaneously by proper utilization of waste materials (slag, RHA, POFA, FA, ash from timber, silica fume, etc.) as an ingredient of cement or constituent of concrete that are suggested by several researchers. The most successful examples are use of coal fly ash to make high-quality, durable concrete and recycling old, demolished concrete as aggregate for new concrete. Since the 1990s, other by-products have been successfully used in concrete. These materials include: used foundry sand and cupola slag from metal-casting industries; post-consumer glass; wood ash from pulp mills, sawmills, and wood-product manufacturing industries; sludge from primary clarifiers at pulp and paper mills; and de-inking solids from paper-recycling companies.

SUSTAINABILITY

It is well known that sustainable development, one of the most important issues in the world at present days, involves to build our communities in such a way that we can all live comfortably without consuming all of our resources. Sustainability requires that engineers consider a building's lifecycle cost extended over the useful lifetime. This includes the construction, maintenance, demolition, and recycling (Moriconi, 2003; Coppola et al. 2004). To build in a sustainable manner and conduct scheduled & appropriate building maintenance are the keys that represent the "new construction ideology" of this millennium. In particular, to build in a sustainable manner means to focus attention on physical, environmental, and technological resources, problems related to human health, energy conservation of new and existing buildings, and control of construction technologies and methods (Coppola et al., 2004).

A sustainable concrete structure is one that is constructed so that the total societal impact during its entire life cycle, including during its use, is minimum. Design for sustainability means accounting in the design for the full consequences, short-term and long-term, of the societal impact. Therefore, durability is the key issue (Moriconi, 2003). New generation of admixtures/additives are needed to improve durability.

PRODUCTION AND CONSUMPTION OF WASTES

Nowadays, due to the increasing population, the demands and consumptions of goods from every sector (e.g. industrial, agricultural) are increasing all over the world. As a results, huge quantity of various types of wastes are also been generated from those sectors as a by-products. Production & consumption of some wastes among the different types of waste are stated as below.

- Slag is a by- product from still industries. Every year about 100 million tons of slag is produced but only 35 million tons are consumed (Nehdi, 2001).
- Fly ash (FA) is the waste generated from coal operated power plant. About 900 million tons FA is produced but no significant amount is consumed (Malhotra, 2006).
- Silica fume is mainly generated from silicon industries. About 2 million tons of SF is produced, no significant amount is consumed (Malhotra, 2006).

These waste generation trends are increasing gradually due to the increasing demand in various industrial and agricultural sectors. Owing the technical advantages of RHA, unfortunately, only a little fraction of these RHA are being used for different purpose such as heat producing in the rice processing mills as alternative of fuel but most of them are dumped as garbage. Although, a few quantities of rice husks are being used as animal's and fish food. There is no exact statistics of consumption of these wastes, only a small portion of these wastes is consumed for different purposes. Although these waste bears valuable and technical merits, but these are simply dumped into ponds, lagoons or as disposed as landfills. However, all of these wastes contain high percent of silica;

consequently, more suggestions have been come out from various researchers to use these as supplement of cement of as ingredient of sustainable concrete.

NECESSITY OF SUSTAINABLE CONCRETE CONSTRUCTION

Concrete is a strong & durable building material with low environmental impact. It is the cornerstone for building construction and infrastructure that can put future generations on the road towards a sustainable future. The concrete industry has to serve the two pressing needs of human society; protection of the environment and meeting the infrastructural requirement for increasing industrialization and urbanization of the world. As we know, cement and concrete industries is the third energy consumer after aluminium and steel (Nehdi, 2001). Besides, huge amount of natural stones, sand, and water is being consuming by the concrete construction. It could be noted that about 850 to 900 kcal/kg (in the dry process), and 1300-1600 kcal/kg (in the wet process) heat energy is required in cement production (Climate Policy Assessment for India, 2004). However, the global average electricity consumption is approximately 111 kWh per ton of cement production (IEA, 2007; IEA, 2009). The embodied energy requirement is nearly 817,600 BTU per ton of concrete production, from which maximum (94%) is comes from cement manufacturing (Wilson, 2011). As a result, natural resources and energies (fuel or electrical) are depleting gradually.

CEMENT REPLACEMENT: POSSIBLE WAYS OF SUSTAINABLE CONCRETE

The preservation of the natural resources and energies is necessary for us as well as for the future generation. Therefore, it is the optimum time to find an alternative way of concrete production or to search another sustainable binder that can be used as supplement of cement for the sustainable concrete production. More amounts of wastes consumption in cement and concrete manufacturing could be another solution. Because, consumption of wastes in cement and concrete production could be reduced the energy demand as well as reduced the CO_2 emission rate. Besides, 300 million tons of CO_2 could be reduced by replacing only 18.5% of the cement with slag or fly ash per year in the world (Bremner, 2011). It is obvious that large-scale cement replacement (60- 70%) in concrete with industrial by-products will be advantageous from the standpoint of cost economy, energy efficiency, durability, and overall ecological profile of concrete (Malhotra 2004). Furthermore, all the concerns in concrete construction have the responsibilities to preserve our natural resources and prevent the CO_2 emission by utilizing more wastes in cement and concrete manufacturing to achieve the sustainable concrete as well as sustainable development. In addition, awareness regarding sustainable construction and sustainable development could spread out among the designer, developer, proprietor and the general public in the society.

BENEFITS OF WASTE MATERIALS AS CEMENT REPLACEMENT

Waste materials when processed properly could be used as valuable engineering materials and could also be satisfy the design requirements. From several researches, rice husk ash (RHA), palm oil fuel ash (POFA), fly ash (FA), slag, etc. can be the replacement of cement to some percentages in concrete production. Pozzolans from industrial and agricultural by- products such as FA and RHA are receiving more attention now since their uses generally improve the properties of the blended cement concrete, and reduce the cost and negative environmental effects (Chindaprasirt & Rukzon, 2008). The 28-day compressive strengths of the saw dust ash (timber ash)/OPC concretes at 5%, 10% and 15% of levels of replacement of cement are about 93%, 78% and 68% of the control mix, respectively (Elinwa & Mahmood, 2002). Use of POFA, RHA, FA & Slag cement has been illustrated below.

PALM OIL FUEL ASH (POFA)

Ground POFA is a good pozzolanic material and can be used to increase both the compressive strength and the sulfate resistance of mortar (Tangchirapat et al., 2009a). Ground POFA with high fineness can be used as a cement replacement to produce high- strength concrete with a compressive strength as high as 70 MPa at 90 days when used to replace Type I Portland cement at 20% by weight of binder (Tangchirapat et al., 2009b). So, for producing high-strength concrete, POFA can be used as a pozzolanic material; it improves the durability and reduces cost due to less use of cement. It will also be beneficial for the environment with respect to reducing the waste disposal volume of landfills.

POFA contains the silica oxide that can react with calcium hydroxide [$Ca(OH)_2$] generated from the hydration process; and the pozzolanic reactions produce more secondary calcium silicate hydrate (C-S-H) gel compound as well as reducing the amount of calcium hydroxide. Thus, for the concrete production, POFA contributes to make stronger, denser and more durable concrete.

RICE HUSK ASH (RHA)

RHA can be used as pozzolans to replace part of Portland cement in making mortar with relatively high strength and good resistance to chloride penetration (Chindaprasirt et al., 2008). RHA has been used in lime pozzolana mixes and could be a suitable partly replacement for Portland cement (Sata et al., 2007).

FLY ASH (FA)

Fly ash (FA) can be used as pozzolans to replace part of Portland cement in making mortar with relatively high strength and good resistance to chloride penetration (Chindaprasirt et al., 2008). Replacements rates of 15% for fly ash cement improved long term concrete properties without much sacrifice in early age properties (Hale et al., 2008).

SLAG

Slag is commonly used in concrete because it improves durability and reduces porosity; improve the interface with the aggregate; lower cement requirement; save energy; and good performance as well as better engineering properties (Mahmood et al., 2009). Hale (Hale et al., 2008) found that replacements rates of 25% for slag cement improved long term concrete properties without much sacrifice in early age properties.

Beside these some other advantages are,

→ With high fineness, POFA can be used as a cement replacement to produce high-strength concrete; it also reduces the water permeability of concrete (Tangchirapat et al., 2009a), produces good resistance against sulfate attack (Jaturapitakkul et al., 2007; Tangchirapat et al., 2007).

→ RHA can be incorporated either as an admixture or as cement replacement material (Oner & Akyuz, 2007). It improves compressive strength (Safiuddin et al., 2010; Saraswathy & Ha-Won, 2007), flexural strengths of concrete (Coutinho, 2003); and split tensile strength of concrete (Habeeb & Fayyadh, 2009; Sakr, 2006; Sensale, 2006), shows better bond strength compare to OPC concrete (Saraswathy & Ha-Won, 2007; Sakr, 2006).

CONCLUSIONS

Fundamental laws of nature say that we cannot create or destroy matter; we can only affect how it is organized, transformed, and used. In fact, to survive in the global community, production would be continued from various sectors, consequently, waste must be generated from those sectors as well. Generally, large volumes of by-product materials are disposed in landfills. Because of stricter environmental regulations, disposal cost is escalating. Thus, searching a sustainable as well as less energy demanded production technology from different sectors, including cement and concrete construction, could be investigated. Recycling not only helps in reducing disposal costs, but also helps to conserve natural resources, providing technical and economic benefits. This is sustainability. Sustainable design must use an alternative approach to traditional design that incorporates these changes in the designer's mind-set.

Post-consumer wastes and industrial by-products can be and should be used in concrete to make "greener" concrete. Recycling minimizes solid waste disposal, improves air quality, minimizes solid wastes, and leads to sustainable cement and concrete industry. The goal of sustainable construction and sustainable development could be achieved by consuming wastes in cement and as an ingredient of concrete as prescribed by several researchers, because, incorporation of wastes in cement and concrete not only reduce the CO_2 emission but also improve the strength and durability properties of concrete; production cost of concrete would also been reduced. Furthermore, proper utilization and consumption of waste is valid, logical and significant issue to reach the sustainable world for the future inhabitants.

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POSSIBILITIES OF INCREASING SETTING TIME USING RICE HUSK ASH (RHA) CEMENT AS BINDING MATERIAL

Md. Shahrior Alam[1]* & Syed Ishtiaq Ahmad[1]

[1] *Dept. of Civil Engineering, Bangladesh University of Engineering and Technology, Bangladesh.*
**Corresponding Author, < shahrioralam@yahoo.com>*

ABSTRACT

There are numerous Portland cement replacement materials that can be used to reduce the amount of Portland cement in concrete; rice husk (RH) ash is one of them. The country like Bangladesh is now producing about 33.0 million tons of rice and almost 6.5 million tons are RH.

RH removed from paddy during rice refining creates disposal problem due to its less commercial value. The use of byproducts of rice is an environment-friendly method of disposal of large quantities of materials that would otherwise pollute land, water and air. This paper will show the use of waste materials like RHA will be effective in cement and the reduction of environmental impact of a construction building .The paper also will show the benefits of using RHA cement in setting time. This cement can be very effective city like Dhaka where jam problem loss the strength of ready mix concrete as ordinary Portland cement has 95 minutes initial setting time but RHA cement has up to(20 % replacing with RH) 215 minutes. Here the total reduction of carbon emission is calculated for using RHA cement and discuss the importance of green cement in the world.

Keywords: Setting Time, Carbon Dioxide (CO_2), Rice Husk (RH), Rice Husk Ash (RHA) Cement, Green Cement.

INTRODUCTION

The primary components of concrete are gravel/stone, sand, water, and Portland cement. Although the Portland cement constitutes only 10 % to 15 % of the total volume, it is the critical material system that, upon reacting with water, binds the other components together. There are numerous Portland cement replacement materials that can be used to reduce the amount of Portland cement in concrete; rice husk (RH) ash is one of them. Rice husks are shells produced during the de-husking of paddy rice. One thousand kilogram of paddy rice can produce about two hundred kilogram of husk, which on combustion produces about forty kilogram of ash. The country like Bangladesh is now producing about 33.0 million tons of rice and almost 6.5 million tons are RH. RH removed from paddy during rice refining creates disposal problem due to its less commercial value. Also, handling and transportation of RH is problematic due to its low density. Rice husk has a very low nutritional value and as they take long time to decompose are not appropriate for composting or manure. Rice husk ash (RHA) is a great environment threat causing damage to land and surrounding area where it is dumped.The use of byproducts of rice is an environment-friendly method of disposal of large quantities of materials that would otherwise pollute land, water and air. Ready mix concrete is important in present condition of construction where short time required and no space for construction. One of the major problems is travelling time for the truck and we know the initial setting time nearly fixed for cement .so need to use retarder which is expense for construction and may also degrade the quality of concrete. So retarding quality of cement may change result. There are several studies on RHA .However none of these deals with

RHA cement which increase the setting time. Therefore, there is a need to study the effect of RHA after it is incorporated in cement.

METHODOLOGY

- Preparing rice husk ash has two parts one is Combustion and other is grinding. Firstly, to produce the best pozzolanas, the RH is carefully burnt. The second step in processing is grinding the RHA to a fine powder, Los Angeles machine was used.

- Testing Was done with the help of ASTM C 187 – 98 "Standard Test Method for Normal Consistency of Hydraulic Cement" and ASTM C 305 "Practice for Mechanical Mixing of Hydraulic Cement Pastes and Mortars of Plastic Consistency"

RESULTS

Table1 shows the initial and final setting times of the cement and different percentages of rice husk (RHA). The initial and final setting times increases with increase in rice husk ash content. An increase in setting time was noticeable from 129 minutes (at 5% RHA) to 215minutes (at 20% RHA) .Similarly, the final setting time also increases as the percentages of RHA increases thereby retarding the hydration process. The increase in setting time of paste having rice husk ash showed low level of hydration for rice husk ash concrete which result from reaction between cement and water, which liberate calcium hydroxide Ca(OH)2).
The percentages of cement replacement level by RHA against standard consistency graph are shown in Fig 2. It was observed that the water demand for standard consistency linearly increases with an increase of cement replacement level by RHA. The specific surface area of RHA is higher than the cement and the ashes are hygroscopic in nature, so needs more water.

Table 1 Setting time using rice husk ash (RHA) cement

Cement (%)	RHA (%)	Initial Setting Time (Mins)	Final Setting Time (Mins)
100	0	95.00	165
95	5	129.00	200
90	10	164.00	235
85	15	178.00	250
80	20	215	285

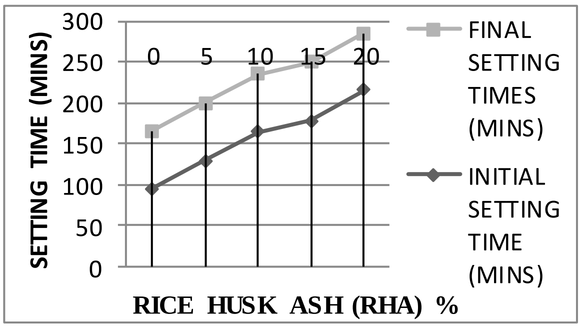

Fig.1: Increasing Setting Time using Rice Husk Ash (RHA) Cement

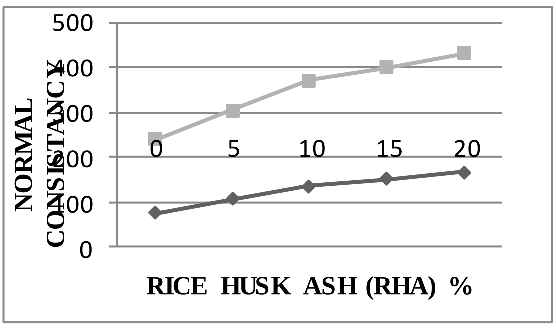

Fig.2: Normal consistency verses % 0f RHA

DISCUSSIONS

A sustainable industrial growth will influence the cement and concrete industry in many respects as the construction industry has environmental impact due to high consumption of energy and other resources. So the important issue is the use of environmental friendly concrete or Green concrete to enable world-wide infrastructure-growth without increase in CO_2 emission. Environmental issues associated with the CO_2 emissions from the production of Portland cement, energy and resource conservation considerations and high cost of Portland cement plants demand that supplementary cementing materials should be used in increasing quantities to replace Portland cement in concrete.

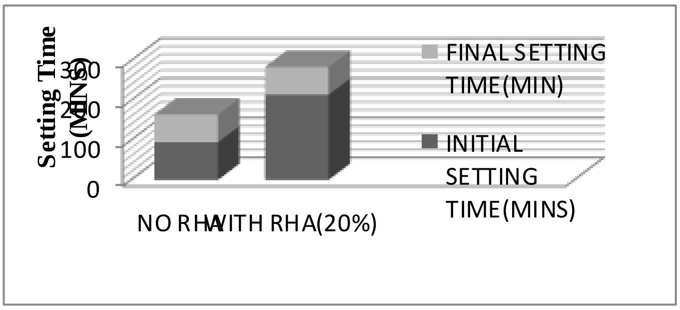

Fig.3: Comparison with RHA cement and normal cement

Another, probably even more important issue, is the use of more environmental friendly structural designs incorporating more environmental-friendly maintenance or repair strategies which requires less use of resources, reduce CO_2-emissions at all phases during the entire service life of a concrete structure. So we need high tech to lower the environmental impact of concrete production. Here we mix 20 %of RHA that will reduce 20 % of carbon emission in cement production and save energy. It means one ton production of cement produces 0.8 ton of CO_2 but in OPC cement which is actually one ton.

Ready-mix concrete is so effective in the site condition of Dhaka where concrete quality is so poor that designer must give extra thought on it. So ready-mix concrete is the solution finding the RHA cement which has more retarder value can be useful on that.

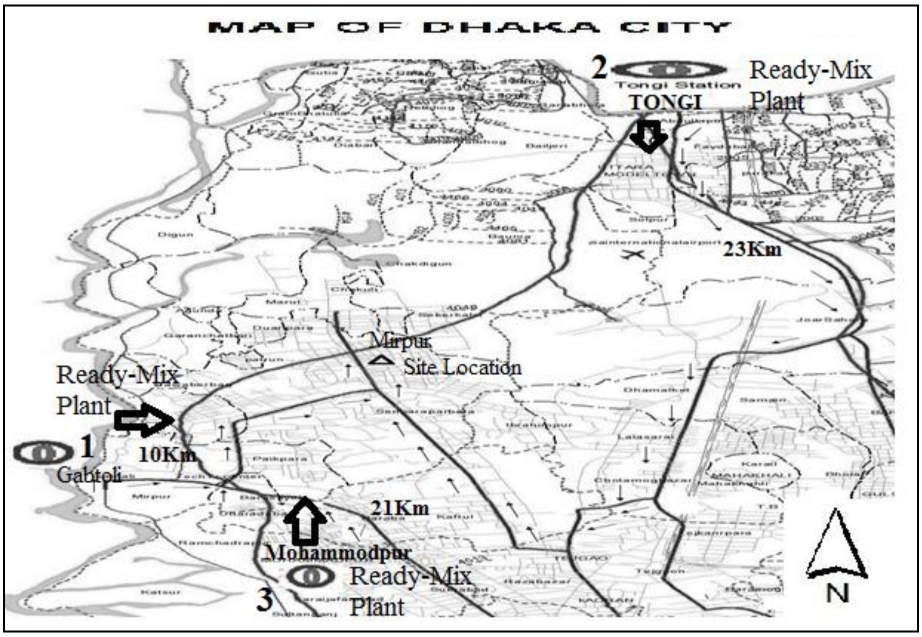

Fig. 4: TRUCK ROUTE OF DHAKA CITY

Fig.4 shows that anywhere from the specified site the minimum distance would be 10 km. It will take an hour to reach to the site and one hour is required for pumping, placing, compacting, vibrating and finishing. So it is important to use retarder admixture but any admixture will increase the value concrete automatically. But if we can use RHA cement the need of using retarder admixture will fulfil automatically. There is a need to study whether it increases the concrete strength or not.

CONCLUSION

Concrete structures are widely used in Bangladesh and all most all over the world. There are numerous Portland cement replacement materials that can be used to reduce the amount of Portland cement in concrete, such as fly ash, slag, and silica fume, RHA. These supplementary industrial by-product (IBP) materials are among the most feasible means of reducing the embodied energy and associated greenhouse gas (GHG) emissions. The development and use of alternative and indigenous construction materials is a growing concern among materials engineers in developing countries. The employment of RHA in cement and concrete has gained considerable importance because of the requirements of environmental safety and more durable construction in the future. RHA blended increase the initial and also final setting time of cement pastes. Rice husk ash prolongs or lengthens the setting time of the cement paste and therefore it is a retarder. There are advantages of prolonging the setting time of cement paste most especially for massive concrete constructions like concrete dams, concrete spur dikes, concrete floodwalls, and other similar structures that contain large volume of concrete. It also prolongs the workability of fresh concrete and provides better blending of concrete especially for successive pours. These are some of the substantial contributions of mineral admixtures in massive concrete structures. To ensure its being economical and environmental friendly we need to use locally available materials. Using RHA as cement replacing materials (CRM) will increase the retarder value and give extra time for setting and construction which hopefully is helped Bangladesh construction arena.

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SUSTAINABLE DEVELOPMENT FOR THE VULNERABLE COMMUNITY OF CHITTAGONG: PROBLEMS & POSSIBILITIES

S. Paul[1*], M. Parvez[1] & T. Islam[1]

[1] *Dept. of Architecture, Chittagong University of Eng & Tech, Chittagong, Bangladesh,*

**Corresponding Author<shajib_arch@yahoo.com>*

ABSTRACT

Environmental disasters in Bangladesh have a negative impact on every aspect of living and poses serious threats to any development project undertaken for sustainable living. As sustainability calls for equitable distribution of opportunities between present and future generations through preservation and conservation of resources, the protection of the life of vulnerable communities should be our first priority. Landslides and Earthquakes are regular geo-environmental hazards in Bangladesh, especially in Chittagong, the south eastern part of the country. Landslide alone has caused the death to more than 300 people in Chittagong division since 2000. One single event in 2007 caused the death of 127 people, affecting 1.5 million people of the region (ref. daily New Age, April 01, 2012 & Landslide Tragedy of Bangladesh by G.M.Sarwar). Landslide has killed more than 40 people in Chittagong in the last 2 years. More than 70,000 people are currently living in vulnerable condition at high risk of landslide and earthquake (ref. irinnews.org). The study and survey was conducted at the Amin jute mill slum, Moti-jharna, Akbar Shah of Chittagong, Bangladesh from April 03, 2014 to May 7, 2014. This paper deals with the methodological framework consistent with the scope of development of the condition of the vulnerable community. The study illustrates an aspect of current national policy as well as erratic behaviour of the nature induced by illegal hill cutting and climate change. Through the survey and analysis this paper thrives to find out some strategic guideline, Landslide mitigation technology and prototype of sustainable, eco-friendly low cost house module for the vulnerable community in the context of Chittagong, Bangladesh.

Keywords: Landslide, Earthquake, Sustainability

Bangladesh is one of the most disaster-prone countries in the world and it has suffered a number of negative consequences associated with various natural and human-induced disasters in its history [1]. Being a disaster prone country has cost Bangladesh a lot in the past and now, it has become one of the biggest hindrances in the path of sustainable development for the country. Sustainable development is a very important aspect of a country, which can only be ensured by reducing disaster risk and the safety of the vulnerable community. Regarding this issue, the UN general secretary stated that development cannot be sustainable if the disaster risk reduction approach is not fully integrated into development planning [2]. Similar opinion was also given by the World Bank Development Committee stating that, natural disasters can be a serious impediment to poverty reduction and thus, may affect poor and vulnerable people the most [3]. Chittagong, the south eastern part of Bangladesh, has been experiencing some regular geo-environmental hazards almost every year where millions of people are living in vulnerable condition under the threat of landslide and earthquake.

Chittagong is the business capital of Bangladesh having a population of six millions at the bank of Karnafulli river with a geographical coverage of 22º14'N - 22º24'30" N latitude and 91º46" E - 91º53' E longitude [4]. About half of the area of this city is plain land and the rest half is consisted of hillock and small hills. At the edge of these hills, the vulnerable communities can be found living illegally in the overpopulated slums on the government properties. These people are at great risk and since 2000 more than 300 people from these communities died by different landslide hazards. In present condition, this community is at high risk of earthquake too. A total of 135 fatalities were recorded in one single event on 11 June, 2007. [5] It is estimated that, more than 70,000 people are living in this area in a vulnerable condition having a high risk of death due to landslide and earthquake [6].

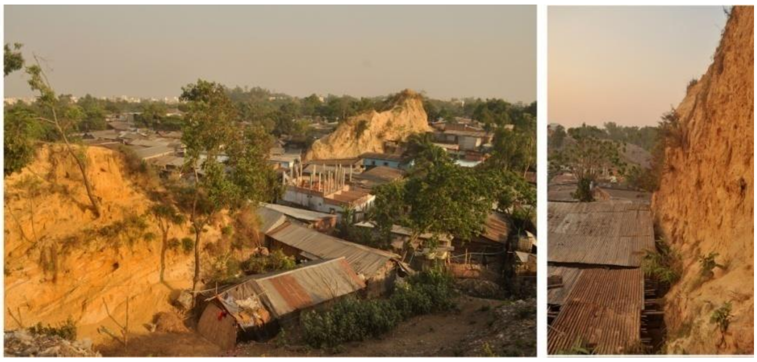
The objective of this paper is to delineate the sustainable development framework for the vulnerable communities by analyzing the results of historical survey. Here sustainable development refers to the safety of the community by suitable landslide mitigation technology and sustainable low cost dwelling construction in the context of Chittagong, Bangladesh.

METHODOLOGY

The study and survey of this paper was conducted at the Amin jute mill slum (Bayzid thana), Moti-jharna and Batalihill (Lalkhan Bazar), Akbar Shah (Khulshi) of Chittagong, Bangladesh from April 03, 2014 to May 7, 2014 by the level - 4 students of department of Architecture, Chittagong University of Engineering & Technology. The survey was conducted as a part of studio project to prepare a report on this vulnerable community. The photographs and drawings of the present condition are collected from the report. The prototype for Disaster resistant building & construction which is described in this paper is experimented in Civil Engineering laboratory. The secondary data that used in the study was collected from various journals, book, seminar and conference papers which are mentioned in the reference chapter.

SURVEY FINDINGS & DISCUSSION

The survey conducted in the mentioned three highly populated areas showed the level of vulnerability of the endangered community. The following pictures will show their living condition and the pattern

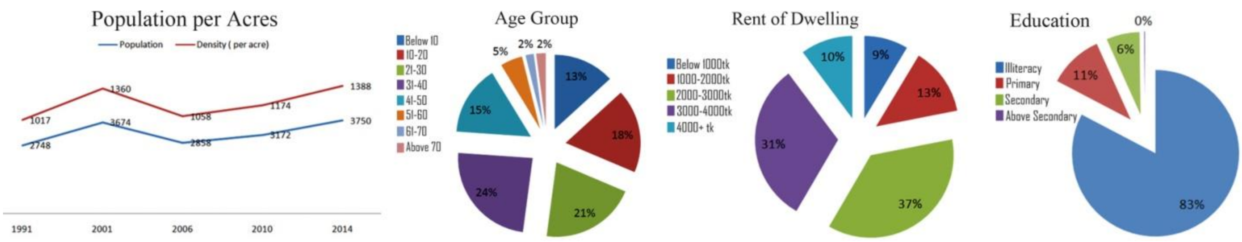


[Fig1. – Earth view of the Slum area] [Fig.2 & 3 – Living condition in Amin Jute Mill slum area]

Fig.1 – Section through the survey site, Amin Jute Mill Slum (Bayzid Thana), Chittagong.

of the dwelling of this vulnerable community. More than 80% of their shelter is made by bamboo structure and tin roofing. Lack of proper foundation and structural support, their shelters fail to sustain in disasters like heavy rain fall, landslide and earthquake. Several tragic events and the amount of life loss show the level of risk under which the community of more than 30,000 people are living here in every slum. The three sites which are mentioned abode showed the same pattern of living condition and level of vulnerability here in Chittagong. Almost every year people die here due to landslide and other natural disaster, which can the prevented by Scientific process and with technical knowledge.

[Fig – Demography Data Charts of the population of survey area] [7]



PROCESS OF DISASTER RISK REDUCTION

Disaster risk reduction makes development sustainable. Safety of the vulnerable community can be ensured by two steps:

1. Protection of the community from the effect of Landslide by having effective Landslide mitigation technique.
2. Disaster resistant low cost low dwelling construction.

LANDSLIDE MITIGATION TECHNIQUES:

There are many methods to control landslide. Such as stacked masonry, rock filled gabions, timber or concrete crib wall, braced wall, pier supported reinforced concrete wall, lofted block wall etc. But a different mitigation technique can be applied which is more effective and economic. It's called Votive grass establishment on slope. This grass is capable to hold the soil stronger than trees. It will take two to eight months to grow up. These grasses not only help to hold soil but also control moisture.

Methods to Improve Vetiver Grass to control surface erosion.

1. Geofabrics
2. Eco Mortar
3. Sand (soil) Bags

All these methods are used to control surface erosion, improve moisture and nutrient retention:

a) Geofabrics:

It is made of Open weave Jute. It's also can be made from organic materials such as jute, coconut fiber or synthetic materials. This fabric will cover the whole slope. Then vetiver grass will be planted to hold the soil. [8]

b) Eco Mortar:

It is a weak shotcrete, (a mixture of cement, soil and fibre).Then to stabilize slopes cover the whole slope with vetiver plants. It's an excellent establishment for erosion control, moisture and fertilizer retention. [8]

c) Sand Bags:

Bags can be filled with sand, soil and fertilizer or a mixture of sand and soil where local soil is poor or rocky. Then vetiver is planted into the bags with soil and fertilizers.

Before After Before After Before After

[Fig.1 – Geo fabrics]

[Fig.2 – Eco Mortar]

[Fig.3 – Sand Bags]

DISASTER RESISTANT LOW COST DWELLING CONSTRUCTION:

The survey showed that, after facing any natural disaster, for example sever rainfall or landslide, the people from the vulnerable community migrate to other sites to take shelter during the catastrophe. But, after a certain period of time when the calamity is under control, they move back to their old place which is still under sever risk. Rehabilitation process does not work for them because of the cheap habitation facilities, work conveniences and other communal issues they get living in these risky zones.

So, rather than forcing these vulnerable community to migrate into another area, one way to solve this problem is turning those risky zone into safe habitations. To protect the vulnerable community in their very own living zone, the dwelling unit should be transform into a disaster resilient shelter, which can protect their inhabitants from a moderate scale of disasters. As the people of the community are basically low income people, the affordability for the material and construction of the dwelling unit is a major issue. Based on the climatic condition and nature of disasters, according to disaster resilient housing research [9], the house plan, design and executions should be carried out considering the following:

a) Strong Foundation: to protect the structure from landslide, moisture and resist lateral force by giving proper stability.

b) Raised Floor: to protect the inhabitants from water overflow due to heavy rain fall and landslide.

c) Raised Roof: to allow cross ventilation and keep the room cool.

d) Operable Walls: to provide shade and to expand the usable communal space in front of room.

e) Go Green: Using of tress and other plants to provide natural shade and barrier against rain and wind.

f) Proper roofing system: Sloped roof is preferable rather than flat roof and low pitched roof are to be avoided.

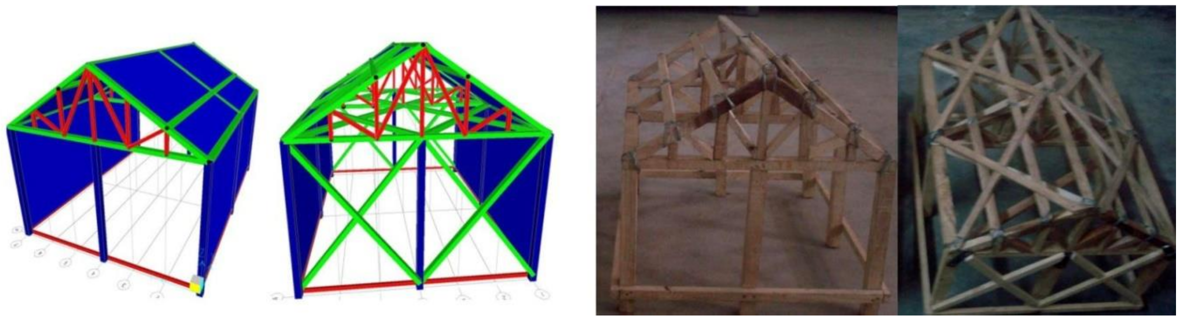
g) Rigidity: The structure must be rigid in all its parts. Cross-bracing helps to avoid tilting and shifting.

j) Continuity: Continuity is very important in disaster resiliency. In the chain of strength from roof to ground, every part must be well connected to each other.

k) Orientation: The shorter side of the dwelling unit should be along with the main wind direction. This will make the structure more resistant to the wind force.

l) Footing of Structure: Even for a low cost house, brick pad footing is very essential for proper foundation. Concrete footing is more effective.

An experimental research was conducted in department of Civil Engineering, CUET on the structural system of low cost housing.[10] The research was conducted for an attempt to increase the load carrying capacity of traditional houses of Bangladesh using tie and bracing. The results of the experiment clearly showed that bracing in traditional houses build by bamboo and other local materials increases the load bearing capacity by 50% which is recommended in the journal paper for cyclone resistant low cost houses, which can equally be used for land slide resistant housing technique where traditional low cost houses were failed to resist the lateral force of wind and mad, the cross brassing system will increase the strength of structure by 50%, which will allow the house to stand against the forces of landslide. The result of experiment is shown below and the source is described in reference section.



[fig.6 – Experimental Model for Bracing System for Disaster Resistant Traditional Housing]

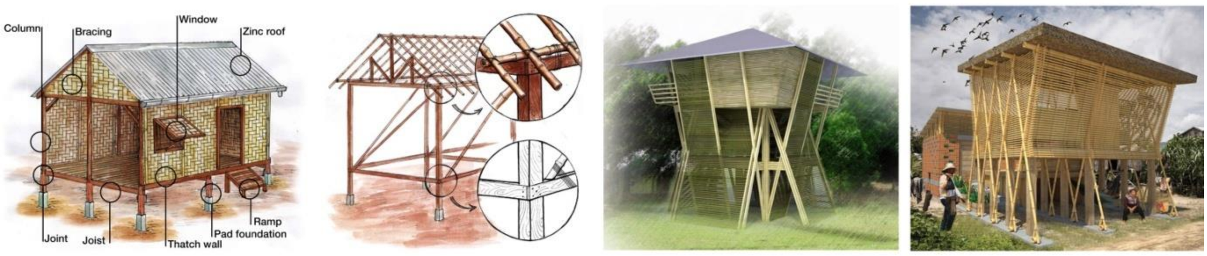
Type of model	Load Carried	Remarks

Without bracing	3000N	Capacity increased by 50%
With Bracing	4500N	

Table – Experimental Result Comparison [10]

RESULTS & RECOMMANDATION

The Survey and Analysis clearly showed that the development of the vulnerable community is depending on the Structural safety of their living condition. Again, that condition cannot be changed overnight by high tech built form or any short of technology. The safety of these vulnerable communities can be ensured by sustainable technical measures which include the sustainable landslide mitigation technique and sustainable low cost dwelling design which will be suitable for their socio economic condition. Technical housing system with bracing and proper landslide mitigation technique which is recommended here will develop the condition of this endangered community of more than 2 million in Chittagong. Some design examples of low cost disaster resistant shelter is shown below, where the bracing system is used.



[Fig. Example of low cost disaster resistant dwelling design using cross bracing technique]

CONCLUSION

Proper Disaster Management, Landslide Mitigation Technique and Sustainable Disaster Resistant low cost dwelling design can ensure a better and safe future for the vulnerable community of Chittagong.

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METHODOLOGY FOR ASSESSING CORROSION OF REINFORCED CONCRETE IN MARINE ENVIRONMENT

Md. Ibrahim Mostazid[1*], Md. Refatul Karim Chy[1], Mohammed Morshedul Hoque[1], Md. Moinul Islam[1] & Md. Saiful Islam[1]

[1] *Department of Civil Engineering, CUET, Chittagong-4349 Bangladesh.*
**Corresponding Author, <mostazidibrahim@gmail.com>*

ABSTRACT

In recent years, the corrosion of steel reinforcement has become a major problem in the construction industry especially when exposed to aggressive environment including marine environment. The progress of corrosion cannot be visually assessed until a crack or a delamination appears. The expansion of the corrosion products (iron oxides) of carbon steel reinforcement may induce mechanical stress that can cause the formation of cracks and disrupt the concrete structure. The corrosion process can be tracked using several electrochemical techniques of which the half-cell potential measurement technique is the most common and is a non-destructive test. Half cell potential techniques has been chosen for assessing the probability of rebar corrosion in structural concrete exposed to a marine environment. A laboratory investigation of reinforced concrete samples exposed to saline water of three different concentration 1T, 3T and 6T is considered. Reinforced concrete samples were exposed for 365 days in mentioned exposure condition. The experimental results indicate that the half cell methodology can be successfully used to assess the probability of corrosion in concrete structures. Results also indicate that for equal period of exposure, nominal corrosion current density (i_{corr}) values were reported to be higher for rebar in concrete exposed to high concentrated salt solution than those for lower concentration.

Keywords: Reinforced Concrete, Corrosion, Marine Environment, Half Cell Potential.

INTRODUCTION

Concrete is the most widely used material in construction due to its attractive physical and mechanical properties and its economic advantages over other materials. In the past decades, it has become well known that the corrosion of reinforcement is the most harmful damage that occurs in reinforced concrete structures. Corrosion of steels in concrete is a global problem for concrete structures and it is a natural process in which a metallic element is transformed into a more stable compound, usually an oxide. Reinforced-concrete structures in the marine environment often deteriorate in the early stages of their service life and the main cause is corrosion of the reinforcing steel and the concrete, which may interact adversely with each other. Damages on concrete induced from environment are due to the migration across the net of pores in concrete, of aggressive agents such as chloride (Cl^-), sulfate (SO_4^{2-}) and others ions, as well as oxygen (O_2) and carbon dioxide (CO_2) dissolved in pore solution. When these deleterious substances reach critical concentrations at the surface of steel reinforcement, the corrosion process starts. The study described in this paper covers the assessment of rebar corrosion in the reinforced concrete structure exposed to artificial aggressive marine environment for 1 year.

When steels corrode, there are usually signs of deterioration on the concrete surface such as rusting, cracking and spalling etc. However, once these signs of corrosion appear, it may be too late to prevent the advance of deterioration by repair works. Half-cell potential measurements are simple, inexpensive and virtually non-destructive techniques to assess the corrosion risk of steels in concrete. These measurements can be used to assess the corrosion risk of steels even if there are no signs of corrosion/distress on the concrete surface, which is a significant advantage of this technique for inspecting existing concrete structures. This study highlights/analyses the changes in concrete corrosion and steel-concrete interface in samples exposed to saline water of three different concentration 1T, 3T and 6T, continuous immersed, alternate wet in saline-water and dry at atmosphere and continuous weathering at atmospheric condition under one year laboratory observation. This paper reports a practical application of half-cell potential measurements for assessment of rebar corrosion in reinforced concrete samples in above mentioned exposure condition. Some samples show no signs of deterioration on the concrete surface. However, the negative potential shows high chloride content and localized corrosion.

MATERIALS AND METHODS

Sample preparation and exposition

A set of cubical reinforced concrete specimens of size 4×4×4 in were prepared using ordinary Portland cement (Type-I), sand and coarse aggregates. Water/cement (w/c) ratio was 0.46. Samples were cured for 28 days in plain water under submerged condition. 33 samples were used for each exposure conditions and two reinforcing bars were embedded in each sample.

Samples were exposed to three different types of environments:

The first 33 samples were immersed on 1T saline water which contains exact amount and proportion of different salts present in normal sea water. **(Fig.1)**

The second 33 samples were immersed on 3T saline water which contains salt amount enhanced to 3 times of that of 1T saline water.

The remainders were immersed on 6T saline water which contains salt amount enhanced to 6 times of that of 1T saline water.

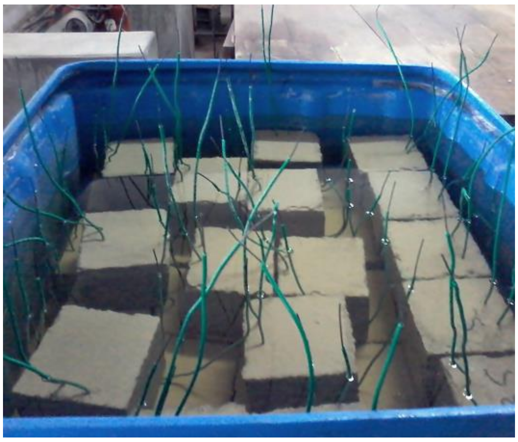

Fig.1: Reinforced Concrete Samples Under Curing

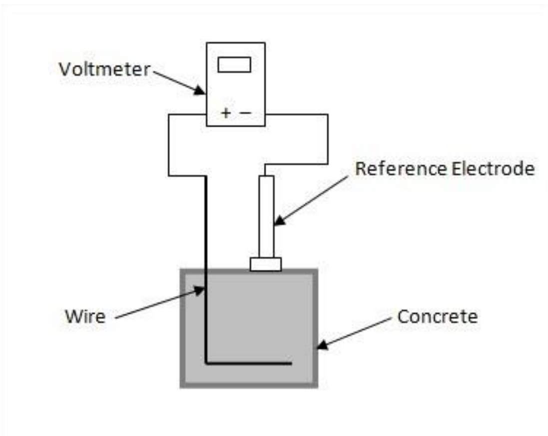

Fig.2: Half Cell Test configuration (for showed concrete samples)

2. *Half-Cell Potential Measurement Technique*

The simplest way to assess the severity of steel corrosion is to measure the corrosion potential, since it is qualitatively associated with the steel corrosion rate. The survey procedure is firstly to locate the steel and determine the bar spacing using a cover meter. **Fig.2** illustrates the basics for such a measurement, also called half-cell potential measurement. The reference electrode is connected to the negative end of the voltmeter and the steel reinforcement to the positive. In these samples the electric wires are laid partly in concrete and partly of it remains outside. This instrument measures the potential and the electrical resistance between the reinforcement and the surface to evaluate the corrosion activity as well as the actual condition of the cover layer during testing. The dependence between potential and corrosion probability are shown in **Table1.**

Table 1: Dependence Between Potential and Corrosion Probability

Potential E_{corr}	Probability of corrosion
$E_{corr} > -350$mV	Greater than 90% probability that reinforcing steel corrosion is occurring in that area at the time of measurement
$-350\text{mV} \leq E_{corr} \leq -200$mV	Corrosion activity of the reinforcing steel in that area is uncertain. % Change of Steel Corrosively Active is 50%.
$E_{corr} < -200$ mV	90% probability that no reinforcing steel corrosion is occurring in that area at the time of measurement (10% risk of corrosion).

The experiment was conducted in order to clarify the fluctuation of the half-cell potential due to various factors: the temperature, and the pre-wetting time. To do this, the half-cell potential was measured in two different weather, summer, and winter. The temperature was approximately 33°C in summer and 23°C in winter. There are three different types of reference electrode generally used for potential measurement namely: a silver/silver chloride electrode, a lead electrode and a copper/copper sulphate electrode. For this study a silver/silver chloride electrode was used. The measurement of potentials for samples was taken under continuous immersion condition of 1T, 3T, 6T concentrated saline water and hence the moisture content of concrete was sufficient.

3. Visual inspection

Before half cell potential measurement the samples were taken out from water tank primarily for visual inspection. By visual inspection, any sign of corrosion, distress in the form of cracks/surface erosion/change in color etc. were observed. Based on this visual inspection, the corrosion state of specimens/rebar was assessed.

RESULT AND DISCUSSION

Half-cell potential, and corrosion state of steels

The laboratory investigation of RC specimens showed no signs of deterioration or corrosion. However, the potential mapping showed alarming results for the reinforcement, as contrary to the visual inspection which showed no signs of corrosion. The measurement was conducted with the silver/silver chloride electrode. All the measured values of half cell potential are shown in **Table 2**. The measured values of the half-cell potential in summer were more negative than those in winter. The most negative potential observed as: - 420 mV for 1T solution, - 490 mV for 3T solution and - 590 mV for 6T solution in summer. The measured values are observed to be decreased in winter. Again the lowest negative potential was obtained as: - 190 mV for 1T solution, - 280 mV for 3T solution and - 305 mV for 6T solution for winter. **Fig.3** shows the potentials vs time graph using average value of the measured potential data. These facts imply that the half-cell potential fluctuated due to the change of corrosive environment of the exposed concrete depending on the temperature. The corrosive environment became more critical as the temperature increased.

Table 2: Measured Potential Data

Month (Starts from January)	Sample in 1T SW (-ve) mV	Sample in 3T SW (-ve) mV	Sample in 6T SW (-ve) mV
1	190	230	270
2	200	285	310
3	305	290	315
4	300	310	390
5	310	330	405
6	350	390	490
7	420	490	590
8	390	450	510
9	310	420	450
10	280	340	390
11	245	310	310
12	240	290	315

The half-cell potential of steel embedded in concrete specimens in laboratory tests was periodically measured and related to the visual observation of concrete cracking. It was observed that, when the half-cell potential values were more negative than - 450 mV to the saturated calomel electrode, 60 percent of the reinforced concrete blocks were seen cracked for the corrosion of the steel. At values between - 525 mV and - 420 mV, the steel was corroded and always enough to cause concrete cracking. In the cracked concrete, the maximum half-cell potential of the steel was measured to be - 590 mV. Changes in corrosion of steel rebar on samples exposed at the three different exposure conditions vs exposure time is presented in **Fig.3 which** provide information about corrosion probability.

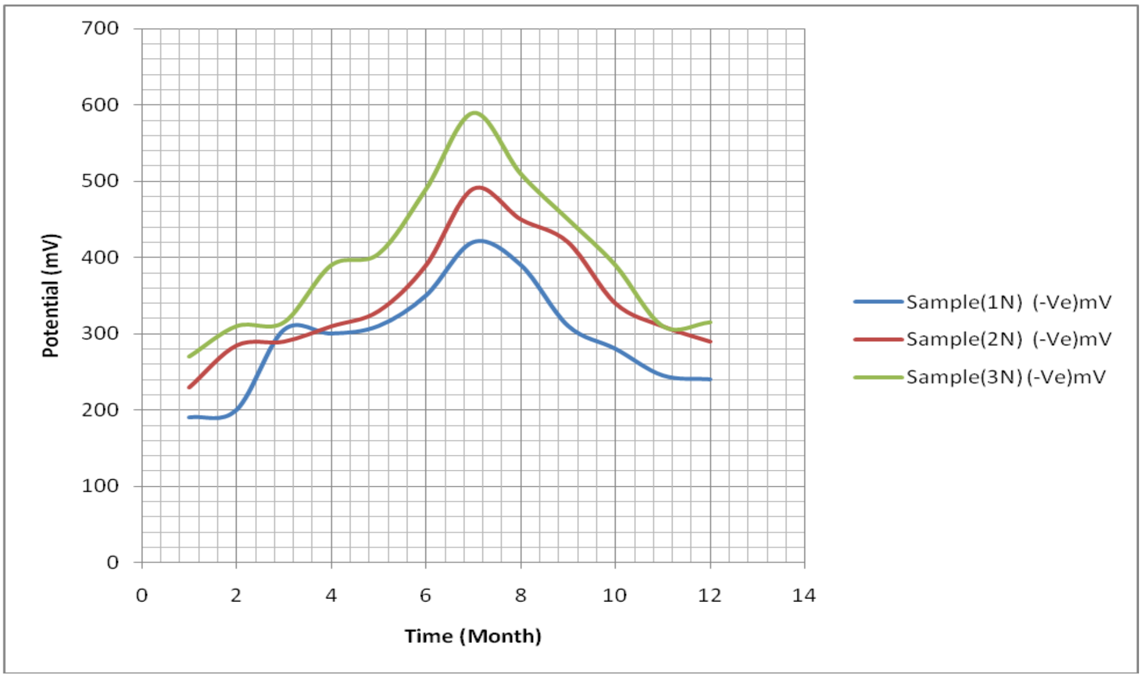

Fig.3: Half Cell Potential vs Time.

The specimens were opened and the reinforcing steel was examined for signs of corrosion. Very little corrosion activity was found in portions of uncontaminated concrete that were free of cracks. In the current research work, all measured values were more positive than - 350 mV, which is the numerical boundary of a greater than - 90 % probability of corrosion in ASTM C 879. From **Fig.3**, it is observed that the rate of corrosion increases with temperature i,e, during summer the rate of corrosion increases and decreases in winter. The half cell potential in summer was shifted by approximately 40 mV to more negative values than in winter. These facts imply that the half-cell potential fluctuated due to the change of corrosive environment of concrete depending on the temperature.

CONCLUSION

Based on the present study, the following conclusions can be drawn regarding the application of half-cell potential measurement to assess the corrosion risk of rebar in concrete in marine exposure condition:

(1) Corrosion rate values are dependent on the environmental conditions and the area on which the current is applied.

(2) Temperature and humidity conditions have a significant influence on the measured corrosion rate values. It is seen that the corrosion rate increases with increasing temperature and it is higher in summer and the rates decreases during winter.

(3) All the structures in the maritime conditions should be monitored during their service life, to avoid high repair costs.

(4) It is essential to combine the on-site corrosion rate measurements with other NDT methods so as to reliably predict the concrete integrity and state of the reinforcement in concrete.

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USE OF FLY ASH IN MAKING GREEN DURABLE CONCRETE FOR COASTAL STRUCTURES

M. Islam[1*] & S. Islam[1]

[1] Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.
**Corresponding Author, <moinul91@yahoo.com>*

ABSTRACT

Good quality concrete with its inherent impermeable characteristics provides ample resistance to the deterioration of structural concrete in marine environment. The use of fly ash as partial replacement of cement in structural concrete has become the common practice due to its beneficial effects. This paper presents an investigation report on the performance of fly ash concrete exposed to artificially make simulated tidal zone marine condition. A total of 600 OPC and fly ash concrete specimens of 100 mm cubical size and 900 cylindrical specimen of size 150 mm diameter and 175 mm high were cast and precured for 28 days in plain water before exposure to different seawater environments. Three different grades of concrete M38, M33 and M28, each with five different cement replacement levels (0, 20, 30, 40 and 60%) with fly ash were used for the experimental program. The concrete test specimens were exposed to sea water of concentrations 1T and 3T (times the actual concentration) to investigate their performance under submerged (SUB) and alternate wetting-drying (AWD). The specimens were exposed to different curing condition over the periods of 1, 3 and 6 months and were taken out periodically for compressive and water permeability test. The study reveals that fly ash concrete, made by replacing cement with fly ash, can improve the strength and durability characteristics of concrete in marine environment. Among all the fly ash concrete studied, fly ash concrete of cement fly ash mix 70:30 can be considered most effective for marine application from strength and durability point of view.

Keywords: Fly ash, Durability, Sea water, Compressive strength, Water permeability.

INTRODUCTION

Durability of concrete is its ability to resist weathering action, chemical attack, abrasion or any other process of deterioration when exposed to aggressive environment. Also durability of concrete is defined as the capability of concrete by itself of keeping the original properties for a certain period (Collepardi, 2000). Durable concrete may be defined as concrete that keeps its original quality, form, and serviceability when it is exposed to its environment (Savas, 1999). Reinforced concrete is a versatile, economical and widely used construction material. It can be moulded to a variety of shapes and finishes. Usually it is durable and strong, performing well throughout its service life. However, it does not perform adequately as a result of poor design, poor construction, inadequate materials selection, exposed to a more severe environment than anticipated or a combination of these factors. In most countries, concrete structures make up a very large and important part of the national infrastructure and both the condition and performance of these structures are important for the productivity of the society (Grigg 1988). Since there is a growing amount of deteriorating concrete structures, however, not only the productivity of the society is affected, but it also has a great impact on resources, environment and human safety. The operation, maintenance and repair of concrete structures are consuming much energy and resources and are producing a heavy environmental burden and large quantities of waste. Thus, the poor durability and premature service life of many concrete structures do not represent technical and economical problems. This is poor utilization of natural resources, and hence, also an environmental and ecological problem (Gjorv 2000).

Compressive strength of concrete is commonly considered as its most important property from various points of view. But in several practical situations, durability is, in fact, more important. The compressive strength usually gives an overall picture of the concrete quality as it is directly related to the structure of the cement paste. The porosity of the hardened cement paste is an indicator of the strength of the paste although the porosity-strength inter relationship for concrete is not, however, simple. Again the permeability of concrete is not a simple function of its porosity but depends on the size, distribution and continuity of the pores which in terms is related with w/c ratio as well as the curing age and conditions. The concrete structures in a marine environment are attacked by various salts ion present in seawater. These detrimental salt ions gradually diffuse into the concrete and after reacting with the hydrated cement paste form expansive/leachable compounds which may lead to the formation of cracks, pore etc. Thus the factors affecting the durability of concrete must be considered before design and placement of structural concrete in such environment. The most important characteristic of concrete that is believed to be affecting its durability is permeability of concrete. There is an approximate inverse relationship between concrete permeability and compressive strength as well as durability. Permeability of concrete can be determined by measuring the rate of fluid (oxygen, water, and chloride ions) penetration into concrete to reach a certain level, for example, level of steel bars because most of the types of deterioration are influenced by fluid ingress (or movement) in concrete. Marine environment is not just over the sea, but it could be deemed to be extending over the coast and the neighbourhood of tidal cracks, backwaters and estuaries. Broadly, it covers the area where concrete becomes wet with sea water and wherever the wind will carry salt water spray which may be as far as few km inland.

The use of blended cement containing supplementary cementitious materials as a replacement for a percentage of Portland cement is more effective than normal Portland cement in reducing the rate of chloride diffusion when properly cured. The four primary types of SCMs utilized are slag, fly ash, silica fume, and metakaolin. Fly ash has been shown to drastically improve chloride ingress resistance by Basheer, et al. (2002), Thomas and Matthews (2004). Thomas, et al. (1999) investigated the use silica fume and fly ash ternary blend cements and found that the ternary blends provided superior chloride ingress resistance to binary blends. The fly ash provided a long-term decrease in the diffusion coefficient, while silica fume increased early age resistance. Basheer, et al. (2002) investigated the use of ternary blends containing fly ash or slag with metakaolin or silica fume. The use of ternary blends resulted in lower diffusion coefficients than binary blends. Fly ash is a byproduct of burning pulverized coal at electric power generating plants. It's a fine-grained material consisting of spherical, glassy particles comprised of silicate glass containing silica, alumina, iron and calcium with diameters ranging from 1 m up to 100 m and a surface area from 300 m^2/kg to 500 m^2/kg. Due to its chemical composition, fly ash exhibits both pozzolanic and hydraulic activities. These properties allow it to be added to Portland cement as a mineral additive at the time of batching, or it can be interground with the cement clinker during the production of the cement. Fly ash is typically used to replace 15% to 25% by mass of cementing material, depending on required properties and workability. In the hardened state, the addition of fly ash greatly reduces the permeability of concrete which provides great resistance to chloride ion ingress (Thomas et al. 1999). The reduction is primarily due to the fineness of fly ash compared to cement. The smaller particles aid in particle packing and result in a more compact concrete mix which reduces pore sizes in the cement paste which reduces the space available for chlorides to penetrate into the concrete. Fly ash is one of the most commonly added extra ingredients in concrete and is classified as a pozzolan, which is a compound that reacts with the lime found in concrete to form a hard paste that holds the aggregate together (Paradise et al. 2003). Fly ash is a synthetic pozzolan, which is created from the by-products of the combustion of ground or powdered coal removed from electric power generating plant exhaust gases. Fly ash is primarily silicate glass containing silica, alumina, iron, calcium and other minor ingredients including magnesium, sulfur, sodium, potassium, and carbon. Some of the benefits of including fly ash admixtures in concrete include improved workability, reduced segregation, bleeding, heat evolution and permeability, inhibiting alkali-aggregate reaction and enhanced sulfate resistance. The fly ash particles are solid spheres that are typically finer than cement. On average, these fly ash spherical particles are less than $0.8x10^{-3}$ inches (20 µm) in diameter (Pham and Newtson 2001). When fly ash is included in concrete mixtures, the density of the concrete is increased due to the voids in the concrete matrix being filled, consequently lowering the potential for corrosion damage as chloride permeability is reduced.

Bangladesh has a long coastline along its southern border. Structural concrete in such location are always under the adverse effect of marine environment. Hence prior to construction of concrete structures at such locations steps should be taken to mitigate the risk of chloride and sulfate attack. Relevant literature reveals that addition of fly ash as a partial replacement of cement in making structural concrete reduces the permeability of concrete, which in turns may resist the penetration of harmful salt ions within the concrete structure. Studies on the use of fly ash concrete in aggressive environments show that the percentage of cement replacement with fly ash and their relative proportion for making concrete in such environment is very important and still debatable as well. In 2006, two units of 125 MW coal based power plant has started generation in Barapukuria, Bangladesh. Currently one million ton of coal is being produced per annum from Barapukuria coal mine of which 65% is being supplied to the 250 MW thermal power plants and other 35% is being used in brick field and other domestic industries. Very limited studies are reported to carry out to investigate the permeability/transport properties of Boropukuria fly ash concrete as made by partial replacement of cement. This experimental program is carried out with a view to study the effects of inclusion of different quantities of Boropukuria fly ash with cement on concrete permeability as well as strength. The study/information regarding the strength and permeability characteristics of concrete containing Boropukuria fly ash would be beneficial for the utilization of these by product materials in concrete construction work, especially from durability point of view.

EXPERIMENTAL PROGRAMS

The experimental program was planned to study the effect of replacement of cement with supplementary cementing material fly ash on the strength and permeability characteristics of hardened concrete. The details of the program including different materials, environments and the various test conducted is summarized below.

Materials used:

ASTM Type-I Portland cement was used as binding material. Fly ash collected from Boropukuria thermal power plant, Bangladesh was used as supplementary cementitious material to produce green concrete. Chemical compositions of cement and fly ash are given in **Table 1.** The coarse aggregate was crushed stone having a maximum nominal size of 12.5 mm with fineness modulus 6.58 and specific gravity 2.7. Locally available natural sand passing through 4.75 mm and retained on 0.075 mm sieve with fineness modulus 2.58 and specific gravity 2.61 was used as fine aggregate.

Variable Details:

In this experimental program, fly ash has been used as blended admixture. Fly ash was added to the concrete mix as partial replacement of OPC.

(a) Artificial seawater of two different concentration 1T and 3T was used as curing water. Plain water was also used for comparison. Artificial seawater is prepared in laboratory by mixing tap water with exact amount of different chemical compounds as specified in **Table 2.** The simulated seawater thus obtained is defined as 1T (One Times) seawater. Similarly 3T simulated seawater is obtained by mixing 3 times chemical compounds respectively as that of 1T solution.

(b) Three different grades of concrete namely M28, M33 and M38 were used in the program. Four different replacement level of cement by fly ash i.e. cement fly ash mix ratio (80:20, 70:30, 60:40, 40:60) were used as binding material. Plain concrete specimens i.e. Cement fly ash mix ratio of 100:0 were also cast as reference concrete. Fly ash concrete means the concrete made by using cement and fly ash as cementitious material with sand, stone chips and water. Relevant information of different concrete mixes is given in **Table 3.**

(c) Two different exposure states, submerged (SUB) and AWD wetting-drying (AWD), were used to simulate submerged and splash/tidal zone condition.

(d) Three different exposure periods of 1, 3 and 6 months were used for this investigation.

Specimen preparation and curing:

A total of 600 no's of cylindrical specimen of size 150 mm diameter and 175 mm high and other 900 no's of cubical specimens of size 100 mm from three different grades of concrete and five different cement fly ash mix ratio were cast as per need for permeability and compressive strength test. The specimens were demoulded after 24 hours of casting and cured in plain water at 27±2°C for 28 days. The specimens were then placed in seawater of different concentration (1T, 3T) as well as plain water for different exposure periods (1, 3, 6 months). Some of the specimens were subjected to AWD cycles (12 hours wetting followed by 12 hours drying) to simulate the tidal marine zone condition. The concrete test specimens were designated keeping concrete grade and replacement level as variable. Thus M28FA40 concrete means grade of concrete is M28 and cement fly ash mix ratio is 60:40.

TEST CONDUCTED

Strength

The concrete specimens were tested for compressive strength at the ages of 1, 3 and 6 months in accordance with the BS EN 12390-3:2009. At each case, the reported strength is taken as the average of three tests results.

Water permeability test

Water permeability test was performed in accordance with the EN 12390-8 at the age of 1, 3 and 6 months. The average of two test results was taken for each type of concrete specimen.

RESULTS AND DISCUSSIONS

Compressive Strength

The compressive strength of OPC and fly ash concrete specimens exposed to different environments has been shown in **Fig.1** to **Fig.6.** Compressive strengths corresponding to '0' month curing age represent the 28 days plain water cured strength. In case of plain water curing, the specimens for compressive strength were kept in SUB condition only and for seawater curing, both SUB and AWD state of exposure conditions were used. In case of plain water curing, OPC concrete shows higher strength at initial ages than that for fly ash concrete. But for relatively longer curing periods, the differences between the results are seen to be decreased. In case of plain water curing, for OPC concrete, compressive strength for M33FA0 concrete after 1 month curing is 36.5 MPa whereas this value for M33FA20, M33FA30, M33FA40, M33FA60 concrete are 34.8, 31.9, 30.1, 18.8 MPa respectively. But after 6 months of curing, compressive strength values for M33FA0 concrete is 41.8 MPa, whereas the same value for M33FA20, M33FA30, M33FA40, M33FA60 concrete are 46.9, 47.9, 45.6, 29.5 MPa respectively. This is due to slow hydration rate of fly ash and for this gain in strength at early age is comparatively lower although after longer curing period, fly ash concrete attains higher strength as that of OPC concrete. Test results also show that compressive strength of both OPC and fly ash concrete is reduced when it is exposed to seawater as compared to plain water curing. In submerged condition and after 6 months of exposure period, compressive strength of M33FA0, M33FA20, M33FA30, M33FA40, M33FA60 concrete are 41.8, 46.9, 47.9, 45.6, 29.5 MPa respectively in plain water; whereas the similar values are 37.4, 42.9, 43.8, 41.4, 26.9 MPa and 36.9, 42.4, 43.3, 41.2, 26.6 MPa respectively for the same concrete exposed to seawater of 1T and 3T concentration. Thus it is also clear that compressive strength is reduced with the increase in seawater concentrations although the nature of variation is not proportional. On the other hand, compressive strength test results of concrete cured in seawater and under submerged condition show relatively higher values than those for alternate wetting-drying condition. After 6 months of exposure periods, compressive strengths of M28FA0, M28FA20, M28FA30, M28FA40, M28FA60 concrete are 32.1, 37.5, 39.3, 38.1, 22.8 MPa and 30.3, 35.4, 37.2, 36.2, 21.5 MPa respectively when exposed to seawater of 1T and 3T concentration in submerged condition, whereas the corresponding values under AWD condition are 29.2, 34.2, 35.9, 34.8, 20.8 MPa and 28.5, 33.4, 35.3, 33.9, 20.3 MPa respectively. This is due to the fact that during wetting cycle, seawater enters into the pore spaces of the concrete and during drying cycles the penetrated salt ions become dried up and react with the cement hydrates that result in the formation of expansive compound and leads to the strength deterioration. Also during drying cycle, moisture inside the concrete may come out form the specimens and as a result, normal hydration process is disturbed.

Rate of strength deterioration for different types of concrete is observed to vary with the grade of concrete and is lower for the higher grade concrete. Also overall observation reveals that fly ash concrete shows relatively higher rate of strength gaining as compared to OPC concrete. Overall gaining of compressive strength was 85.5% for OPC concrete, 98.6% for 80:20 fly ash concrete, 103% for 70:30 fly ash concrete, 98.6% for 60:40 fly ash concrete and 62.8% for 40:60 fly ash concrete. Fly ash concrete of cement fly ash mix ratio 70:30 shows higher resistance to seawater penetration as compared to OPC concrete as well as highest gaining in compressive strength. This is due to the higher degree of fineness of fly ash, which after hydration blocks the pores inside the concrete thereby reducing its permeability. As a result, entrance of seawater in concrete is restricted and the amount of salt ion penetration is thereby reduced. Thus, the rate of deterioration is seen to be decreased and ultimately leads to higher concrete compressive strength.

Water Permeability

Permeability characteristics of three different grades of concrete M38, M33 and M28 exposed to plain water (PW) as well as corrosive marine environment (SW) are graphically presented in **Fig.7 to Fig.12.** The specimens were exposed to plain water for submerged condition whereas in sea water both submerged as well as alternate wetting drying state was used. In all the figures, the values shown at '0' month curing age, represent the 28 day permeability of plain water cured concrete. These values for concretes M38, M33 and M28 are 3.42×10^{-12}, 5.82×10^{-12} m/s and 7.84×10^{-12} m/s. Specimens exposed to sea water show a gradual increase in the permeability at later stages but no definite trend is noticed with the increase in salt concentration of the curing environments whereas specimens exposed to plain water show a gradual decrease in permeability. Permeability values for M38FA0 concrete cured in plain water were 3.01×10^{-12}, 2.33×10^{-12}, 1.96×10^{-12} m/s after 1 month, 3 months and 6 months of curing period, whereas the same value for sea water curing of 3T concentration in submerged state were 2.86×10^{-12}, 2.02×10^{-12}, 2.13×10^{-12} m/s for same curing period. After 6 months exposure, the permeability of concrete subjected to curing in sea water lies in the range of 1.73×10^{-12} to 9.97×10^{-13} m/sec, whereas for the specimens cured in plain water environments ranges from 1.34×10^{-12} to 6.43×10^{-12} m/sec only. As expected, lower permeability values are found to associate with relatively higher grade concrete i.e. the concretes having lower water-cement ratio. A careful study of the curves indicate that in corrosive environment, the permeability of all grades of concrete rapidly decreases upto a period of 3 months and then become starts to increase with the exposure periods. The concrete permeability decreases very rapidly initially and the rate depends on the concrete grade, the higher reduction being associated with the higher grade of concrete.

The results of the permeability characteristics of different grades of concrete in the corrosive environment, however, show a different trend with exposure periods. Fly ash concrete shows relatively higher value of permeability coefficient compared to OPC concrete for early age of curing. But at later age of curing reverse trend was observed. Coefficient of permeability value for M28FA20, M28FA30, M28FA40, and M28FA60 concrete are 7%, 14%, 21% and 31% higher as compared to M28FA0 concrete for 1 month of curing period in artificial sea water of 1T concentration; whereas the same value is -2%, 3%, 12% and 28% higher for M28FA20, M28FA30, M28FA40 and M28FA60 concrete compared to M28FA0 concrete for 6 months of curing in same curing environment. Fly ash concrete shows relatively lower permeability as compared to OPC concrete for longer period of curing. The curves for the permeability values of concrete specimens in artificial sea water shows higher increasing rate compared to the decreasing rate of the curves for concrete specimens in non-corrosive environment. A possible explanation could be that, with the passage of time, the products formed due to the chloride and sulphate attack i.e. $CaC1_2$, brucite etc. leach out after a certain period. These periods are required for the saturation of concrete specimens to get a steady state flow of water. As a result during this period the permeability of concrete specimens were decreased. After leaching out of the products, micro pores to be reunited or new paths to be formed resulting in fresh pore connectivity. As a result, the permeabi1ity of concrete increased, which exceeded the initial permeability of concrete. Among all the concrete studied, fly ash concrete of cement replacement level 30 shows the best result. After 6 months of curing in sea water, the coefficient of permeability values for M38FA0, M38FA20, M38FA30, M38FA40, M38FA60 concrete are 103%, 98%, 102%, 105%, 182% and 119%, 131%, 111%, 118%, 191% of 28 days plain water cured OPC concrete of similar grade for sea water of 1T and 3T concentration respectively. The initial decrease in permeability may be due to filling up of the voids in concrete by the salts of crystallization or the reaction products resulting due to chemical action of sea water with the hydrated cement paste. The subsequent increase in the permeability at later ages may be due to the dissolution of the products formed during the sulphate attack. Under the applied hydrostatic pressure required in the permeability test, the capillary micro pores get reunited showing higher permeability of concrete. The leaching of some calcium compounds in sea water also causes an increase in the permeability of concrete.

CONCLUSIONS

Based on the results of the investigation conducted on different fly ash concrete made with various level of cement replacement as mentioned and cured for various environment and curing period up to 180 days, the following conclusions can be drawn:

(1) At the initial age of curing, strength gaining rate for fly ash concrete specimens is relatively lower as compared to corresponding OPC concrete.
(2) Compressive strength of concretes specimens is observed to be decreased with time when exposed to sea water and also strength deterioration rate is increase with the increase salt concentration of the environment.
(3) 30% mixing of fly ash exhibited the best results with respect to compressive strength. Fly ash concrete with 30% cement replacement shows around 20% higher compressive strength than OPC concrete after 6 months of curing.
(4) Addition of the fly ash reduced water permeability of concrete upto 50% cement replacements. Fly ash concrete with 30% cement replacement showed relatively higher reduction of coefficient of permeability as compared to OPC concrete.
(5) Higher grade concrete showed higher strength gaining and lower coefficient of permeability value as compared to lower grades of concrete. Concrete specimens exposed to AWD state in sea water of different concentration exhibited lower strength gaining and higher value of coefficient of permeability than that of SUB state.
(6) Use of fly ash as partial replacement of cement in any construction work markedly reduces the cost of cement which otherwise been dumped making environmental hazard. Fly ash concrete is reported as environmentally friendly.

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Table 1: Chemical Composition (%) of Ordinary Portland Cement and Fly Ash

Constituents	**Composition**	**OPC**	**FA**
Calcium Oxide	CaO	65.18	0.65
Silicon Di-Oxide	SiO_2	20.80	51.49
Aluminum Oxide	Al_2O_3	5.22	31.60
Ferric Oxide	Fe_2O_3	3.15	2.80
Magnesium Oxide	MgO	1.16	0.28
Sulfur Tri-Oxide	SO_3	2.19	0.19
Sodium Oxide	Na_2O	--	0.18
Loss on Ignition	--	1.70	4.2
Insoluble Residue	--	0.6	--

Table 2: Specified salt contents of artificial seawater used in experimental program

Salt	**Chemical formula**	**Amount (gm)**	**Remarks**
Sodium chloride	$NaCl$	27.2	These amounts of salts were dissolved in plain water to prepare 1000 gm of seawater of 1N
Magnesium chloride	$MgCl_2$	3.8	
Magnesium sulfate	$MgSO_4$	1.7	
Calcium sulfate	$CaSO_4$	1.2	
Potassium sulfate	K_2SO_4	0.9	

Table 3: Mix proportions and properties of fresh concrete

Mixture constituent & properties	**Grade of Concrete**		
	M28	**M33**	**M38**
Cement (kg/m^3)	435	480	500
Water (kg/m^3)	218	224	218
Sand (kg/m^3)	545	530	520
Stone Chips (kg/m^3)	1150	1130	1120
water/cement Ratio	0.50	0.47	0.44
Slump (mm)	68	63	60
Air content %	1.3	1.2	1.1

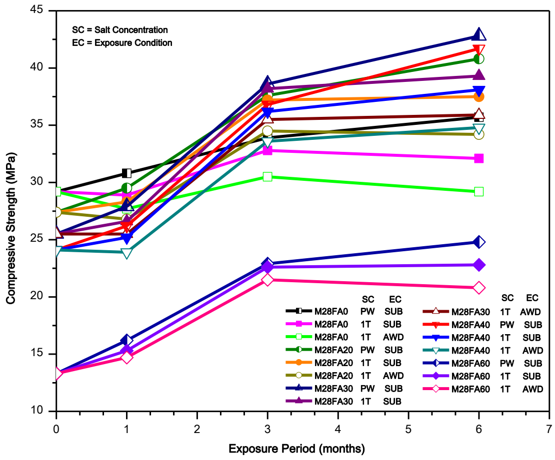

Fig.1: Compressive Strength - Exposure Period Relation for M28 Grade Concrete Exposed to SW of Concentration 1T

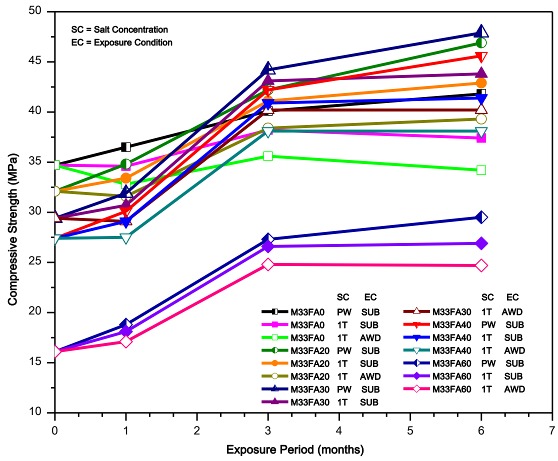

Fig.2: Compressive Strength - Exposure Period Relation for M33 Grade Concrete Exposed to SW of Concentration 1T

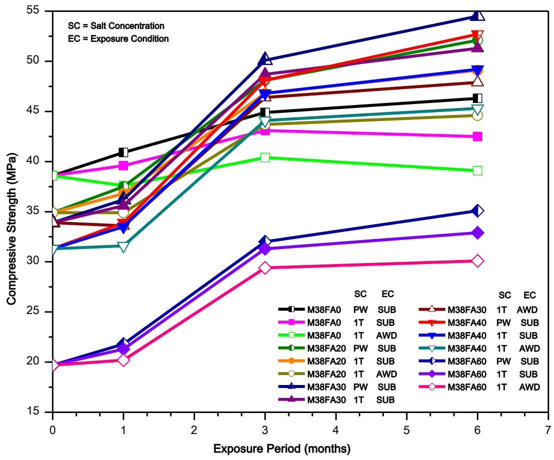

Fig.3: Compressive Strength - Exposure Period Relation for M38 Grade Concrete Exposed to SW of Concentration 1T

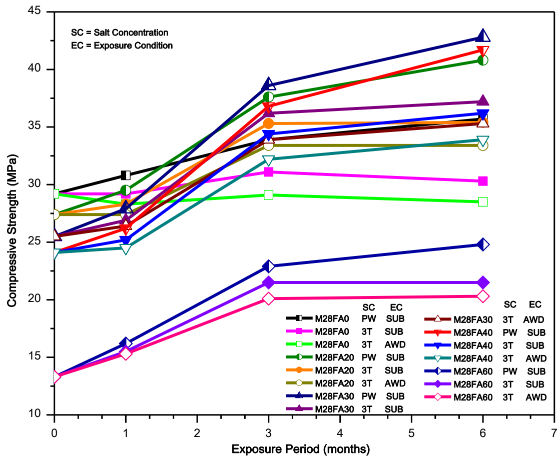

Fig.4: Compressive Strength - Exposure Period Relation for M28 Grade Concrete Exposed to SW of Concentration 3T

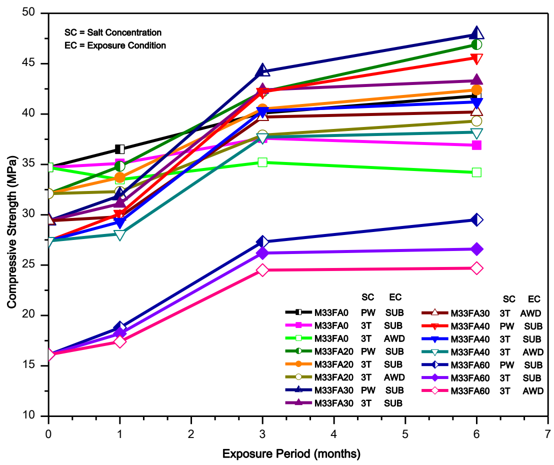

Fig.5: Compressive Strength - Exposure Period Relation for M33 Grade Concrete Exposed to SW of Concentration 3T

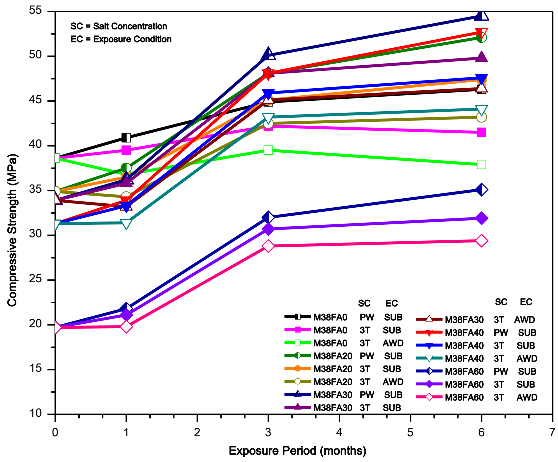

Fig.6: Compressive Strength - Exposure Period Relation for M38 Grade Concrete Exposed to SW of Concentration 3T

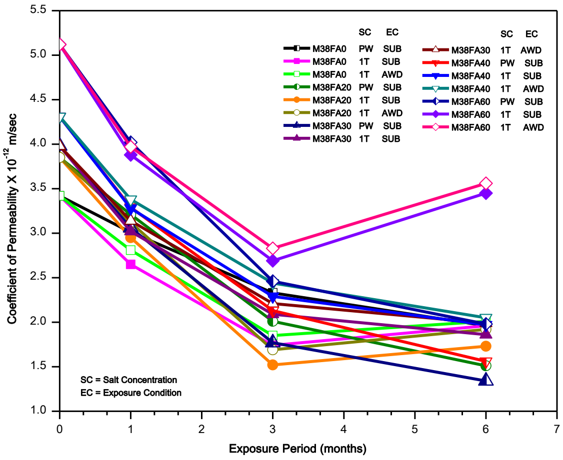

Fig.9: Permeability - Exposure Period Relation for M38 Grade Concrete Exposed to SW of Concentration 1T

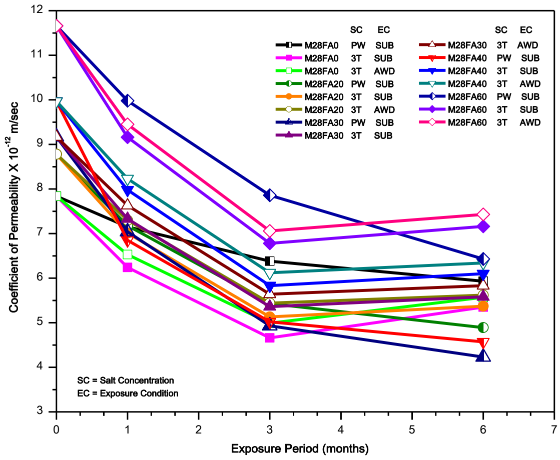

Fig.10: Permeability - Exposure Period Relation for M28 Grade Concrete Exposed to SW of Concentration 3T

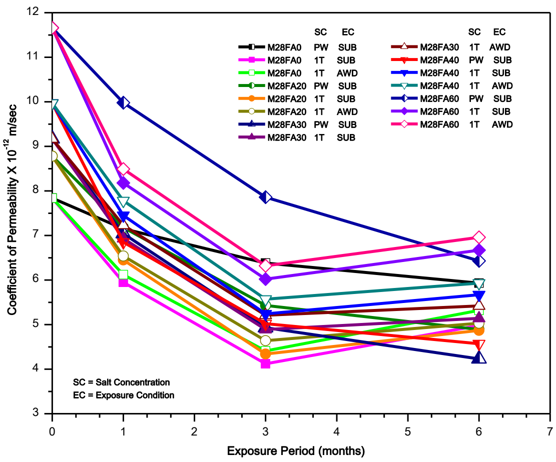

Fig.7: Permeability - Exposure Period Relation for M28 Grade Concrete Exposed to SW of Concentration 1T

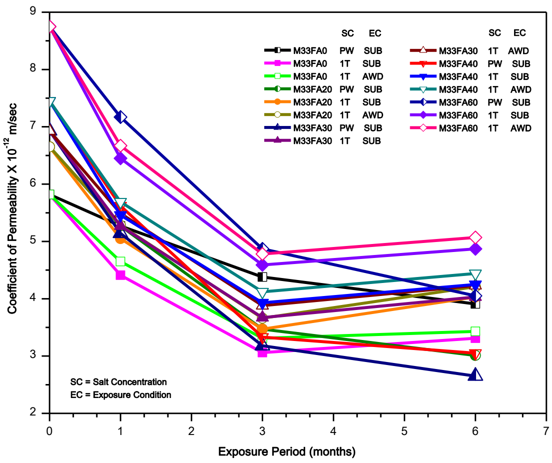

Fig.8: Permeability - Exposure Period Relation for M33 Grade Concrete Exposed to SW of Concentration 1T

ID: SEM 053

LANDSLIDE SUSCEPTIBILITY MAPPING USING ANALYTICAL HIERARCHY PROCESS IN CHITTAGONG CITY CORPORATION AREA

S. Das[1*], D. R. Raja[1]

[1] *Department of Urban and Regional Planning, Chittagong University of Engineering and Technology, Chittagong, Bangladesh.*
**Corresponding Author, <souravurp09@yahoo.com>*

ABSTRACT

Landslide phenomena is the most burning issue in Chittagong City Corporation (CCC) area which causes great problems to the life and properties. This situation is decreasing day by day and becoming one of the main problems of city life. On 11 June 2007, a massive landslide happened in Chittagong city area. As a result, a number of foothill settlements and slums were demolished; more than 90 people died and huge resource destruction took place. It is therefore essential to produce the future landslide susceptibility maps for CCC area so that appropriate mitigation strategies can be developed to help combat the impacts of climate change. To prepare community susceptibility map of landslide hazard, Geographic information system (GIS) and Remote sensing (RS) based Analytical Hierarchy Process (AHP) model are used in this research. The major findings can be described as 27% of total CCC area which is susceptible to landslide hazard where 6.5 sq.km areas are very high susceptible. The findings of the research can be used to prioritize risk mitigation investments, measures to strengthen the emergency preparedness and response mechanisms for reducing the losses and damages due to future landslide events.

Keywords: Analytical Hierarchy Process (AHP), Geographic information system (GIS), Remote sensing (RS), Susceptibility

INTRODUCTION

Due to its geographical location, Chittagong city suffers from numerous natural disasters like landslide, water logging, cyclone, flood etc. But at present landslides are the most burning issues in respect of Chittagong city area. Chittagong hills are degrading by different anthropogenic stress such as, hill cutting for construction, sand and clay mining purpose, increasing settlement in foothills, deforestation which are very much responsible for landslide occurrence. The city, under the jurisdiction of City Corporation (Fig1), has a population of about 2.5 million and is constantly growing with an area approximately 155 square kilometers (CDA, 2014).The CCC has experienced several devastated landslide incidences that brought vast damage to properties and natural environment, and some loss of human life showing in Table 1.The landslides in Chittagong are classified as ‘Earth Slides’ since those consist of 80% sand and finer particles.It has been stated that the rainfall intensity and duration play very important role in producing these shallow landslides in Chittagong because of climate change (Khan et al.,2012).It is therefore essential to produce the future landslide susceptibility maps for CCC area so that appropriate mitigation strategies can be developed to help combat the impacts of climate change. The increase of computer-based tools has been found

Table 1: Summary of the crucial landslide incidences in the last 10 years

Date	Location	Consequence
3 August 2005	Nizam Road Housing Society	2 people killed and 12 injured
11 June 2007	MatiJharna Colony of Lalkhan Bazar	128 people killed and 100 injured
10 September 2007	Nabi Nagar in Chittagong	2 people killed
18 August 2008	Matijharnain Chittagong	11 people killed and 25 injured
26 June 2012	Lebubagan Area &Foys Lake	90 people killed and 150 injured
01 July 2011	Batali hill, Tigerpass intersection	15 people killed and 150 injured
28 July 2013	Lalkhan Bazar	2 people killed

Source: ADPC, 2014 & Field survey, 2014

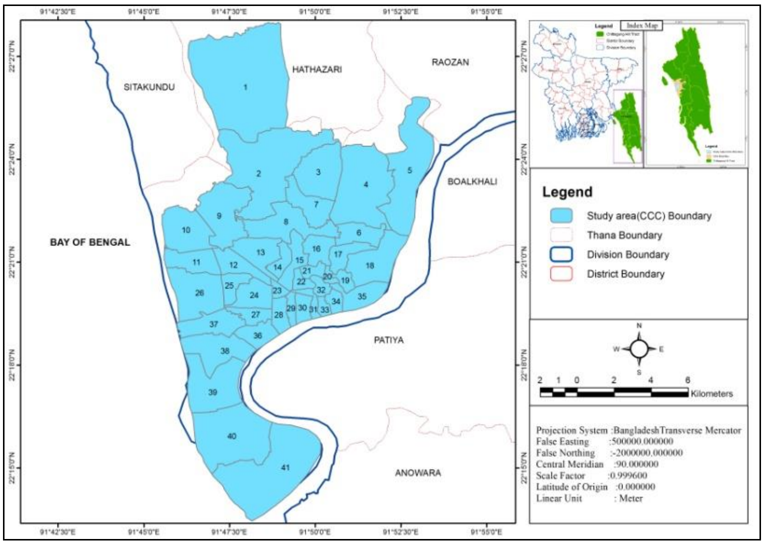

Fig 1: Location map of the study area (CCC)

to be useful in the hazard mapping of landslides. To prepare community susceptibility map of landslide hazard, Geographic information system (GIS) and Remote sensing (RS) based Analytical Hierarchy Process (AHP) model are used in this research.

DATA Processing

In this study ten instability factors are considered which are elevation, slope aspect, slope angle, land cover, NDVI, distance to road, distance to water body, drainage density, geology and geomorphology. The primary data are collected from field survey to prepare the landslide hazard inventory. A total of 20 landslide locations are identified in the study area through field survey. Land cover data are generated from Landsat 8 collected from United States Geological survey (USGS) websiteand 5 major classes are taken such as sandy land, vegetation, water bodies, built-up area, paddy fields &shrubs. The reference pixels are compared with the base map (2010) collected from the Chittagong Development Authority (CDA). The overall accuracy is found as 85.25%. The NDVI map of CCC is also prepared from Landsat 8 using flowing equation.

$$NDVI= ((IR-R)/ (IR+R)) \quad \dots \dots \quad (1)$$

Where, IR=DN values from the infrared band (Band 5)&R=DN values from the red band (Band 4).

The index outputs values between -1.0 to 1.0 (pixel values 0–255) mostly represent greenness. Elevation, slope and aspect can be generated from the raster DEM 30-meter contour interval obtained from USGS website. The output slope angle raster is calculated as percent slope angle andthe aspect represents the down slope direction of the maximum rate of change in value from each cell to its neighbors. Final results are reported in terms of the 8 basic compass directions on the output map. The elevation map can also be prepared from the DEM layer where the relative height of the layer is considered. The distance image from road network layer (300 m) is prepared using 'Euclidean technique which gives the distance from each cell in the raster to the closest source. The distance to water body is also calculated at 200-meter intervals. Drainage density has also extracted directly from stream line of 100m intervals. Geology and Geomorphology data are collected from Geological Survey of Bangladesh (GSB). For precipitation factor, the average daily precipitation of the whole CCC area is more of less same and there is only one weather station installed in the study area so triggering effect of the precipitation factor is assumed to be uniform throughout the study area (BMD, 2014).

LANDSLIDE SUSCEPTIBILITY-MAPPING METHODS

The AHP method is a multi-factor decision-making process which involves building a hierarchy of decision elements (factors) and then making comparisons between possible pairs in a matrix to give a weightfor each element and also a consistency ratio(CR) (Malczewski, 1999).Factor weights for each criterion are determined by a pairwise comparison matrix (Saaty, 1977; Saaty& Vargas, 2001). In the construction of a pairwise comparison matrix, each factor is rated against every other factor by assigning a relative dominant value between 1 and 9 to the intersecting cell. When the factor on the vertical axis is more important than the factor on the horizontal axis, this value varies between 1 and 9. Conversely, the value varies between the reciprocals 1/2 and 1/9. The diagonal boxes of a pair-wise comparison matrix always take ascertain value of 1.The boxes in the upper and lower halves are symmetrical with one another and the corresponding values are, therefore, reciprocal with each other. In this study, AHP considers weighting and rating system developed by collecting questionnaires from expert opinions and secondary data sources.The class weightage and the factor weightage are multiplied each other to produce a combined weightage map of landslide susceptibility as follows:

$$SI=\sum_{i=1}^{n}(Wi\times Ri)\ldots\ldots \quad (2)$$

Where, SI is the required susceptibility index of the given pixel and R_i and W_i are class weight (or rating value) & factor weight for factor irespectively.

The weightage maps are classified into five (5) classes using Natural breaks (Jenks) classification method characterized by Very High, High, Medium, Low and Very Low Susceptibility. Finally, validity of the map was examined using 20 known landslide locations within the area obtained from the field surveys and from official records of the responsible authorities.

RESULTS

Landslide susceptibility mapping consists of the derived factor weights and class weights, and a calculated CR, as seen in (Table 4). In this research, the resulting CR for all the cases is found less than 0.10 (Table 4 and Table 5). From Table 6, it is found that the LSI had a minimum value of 0.053, and a maximum value of 0.457, with an average value of 0.162 and a standard deviation of 0.061.The LSI represents the relative susceptibility of a landslide occurrence.These LSI values are then divided into five classes based the natural breaks range, which represent five different zones in the landslide susceptibility Map showing in Fig 2.Only 11% of the total areas are classified as being in the VHS

Table 4: Pairwise comparison matrix, CR and weights of the sub-criteria of the data layers

Factors	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**Eigen value**	**Factors**	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**Eigen value**
Land cover							**Distance to road (m)**						
(1)Water body	1					0.03	(1)0-300	1					0.54
(2)Vegetation	7	1				0.29	(2) 300-600	1/3	1				0.23
(3)Urban Area	5	1/3	1			0.13	(3) 600-900	1/6	1/3	1			0.09
(4)Paddy Field	2	1/6	1/3	1		0.05	(4) 900-1200	1/8	1/5	1/2	1		0.07
(5)Bare Soil	9	2	5	7	1	0.47	(5) 1200-1500	1/9	1/4	1/2	1/3	1	0.04
CR : 0.05							CR: 0.06						
Factors	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**Eigen value**	**Factors**	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**Eigen value**
Slope angle (°)							**Distance to water body(m)**						
(1) 0 - 2	1					0.05	(1) 0-200	1					0.50
(2) 2-10	2	1				0.08	(2) 200-400	1/3	1				0.23
(3) 10-15	4	2	1			0.14	(3) 400-600	1/5	1/2	1			0.12
(4) 15-30	7	6	5	1		0.51	(4) 600-800	1/6	1/4	1/2	1		0.07
(5) 30-37	4	3	2	1/3	1	0.21	(5) 800-1000	1/7	1/5	1/3	1/2	1	0.04
CR :0.04							CR : 0.04						
Slope aspect							**Drainage density (m)**						
(1) South, South-west	1					0.54	(1) 0 - 100	1					0.05
(2)West, north-west	1/2	1				0.29	(2)100-200	2	1				0.08
(3)North, south-east	1/6	1/3	1			0.10	(3)200-300	3	2	1			0.14
(4)East, north-east	1/8	1/5	1/2	1		0.05	(4)300-400	5	3	2	1		0.25
CR : 0.008							(5)400-500	7	5	4	2	1	0.46
Elevation (m)							CR : 0.013						
(1) 0-12	1					0.04	**Geomorphology**						
(2) 12-22	2	1				0.07	(1)Fluvio tidal	1					0.10
(3) 22-40	4	2	1			0.14	(2)Hilly landforms	6	1				0.62
(4) 40-60	8	6	4	1		0.50	(3) Others	1/2	1/8	1			0.06
(5) 60-88	5	4	2	1/3	1	0.23	(4)Tidal landforms	2	1/4	4	1		0.20
CR : 0.03							CR : 0.03						
NDVI							**Geology**						

(1)- 0.134773- 0	1					0.04	(1)Fluvio tidal deposit	1					0.06
(2)0- 0.105841	9	1				0.47	(2)Hilly deposit	5	1				0.28

Susceptibility classes	Susceptibility index values	Susceptible area (km)	% of Area	Number of landslide points	Frequency ratio (FR)

(3) 0.105841 - 0.203147	6	1/2	1			0.29	(3) Others	1/2	1/7	1			0.04

Landslide susceptibility index

Data layers	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	Eigen values
Land Cover (1)	1										0.0629
Slope (2)	4	1									0.1995
Aspect (3)	1/2	1/5	1								0.0361
Elevation (4)	5	2	6	1							0.2575
NDVI (5)	2	1/3	3	1/4	1						0.0787
Distance to road (6)	1/5	1/8	1/3	1/9	1/4	1					0.0179
Distance to waterbody (7)	1/3	1/7	1/2	1/8	1/3	2	1				0.0252
Drainage density (8)	1/2	1/5	2	1/6	1/2	3	2	1			0.0445
Geomorphology (9)	2	1/2	4	1/2	2	7	6	3	1		0.1233
Geology (10)	3	1/2	5	1/2	2	7	6	4	2	1	0.1545

CR : 0.03

(4) 0.203147 - 0.302287	2	1/7	1/5	1		0.06	4)Slope & valley deposit	6	2	9	1		0.44
(5) 0.302287 - 0.491584	3	1/4	1/3	2	1	0.12	(5)Tidal deposit	3	1/2	4	1/3	1	0.16
CR : 0.02							CR : 0.02						

Table 5: Pair wise comparison matrix, factor weights and CR of the data layers

(4%) or HS (7%) landslide susceptibility zones but they have accommodated about 80% of the landslide reference points. Otherareas are located in the MS (18%), LS (39%), and VLS (33%) susceptibility zones and only 4 landslide incidences (out of 20) are being observed in the MS zones.

Table 6: Allocation of the reference landslide points within the defined landslide Susceptibility class and the associated frequency ratio (FR) of each class

Very low susceptibility (VLS)	0.053 - 0.130	57.11	33	0 (0%)	0.000
Low susceptibility (LS)	0.130 - 0.189	67.58	39	0 (0%)	0.000
Moderate susceptibility (MS)	0.189 - 0.233	30.29	18	4 (20%)	1.142
High susceptibility (HS)	0.233 - 0.316	11.45	7	7 (35%)	5.289
Very high susceptibility (VHS)	0.316 - 0.454	6.58	4	9 (45%)	11.832

The frequency ratio (FR) values are computed from ratio of the percentage landslide occurrences and the percentage area coverage (for each individual class to the whole study area). The possible values begin from 0 onwards where relatively high ones (much greater than 1) indicate high chance of having landslides while low values (close to 0) indicate lower chance of having landslide over the area.The FR values of 11.832 for the VHS zone and 5.289 for the HS zone indicate the higher chance of having landslide activities in these areas when compared to those of the MS (1.142) and LS (0).

The very high susceptible landslide locations in CCC area are identified as *Lebubagan area, kusumbag residential area, Batali hill area, Motijharna Area, Foy's Lake Area*, *Khulshi Area, Nasirabad Area, Goalpara Slum, kanandharaabasikprokolpo etc.* Among those locations, *Motijorana*area and *Batali Hill* are considered as the most susceptible locations in CCC.This area is also heavily populated and occupied by lower income groups. Most of the inhabitants are poor factory workers. Any large scale landslide can cause massive destruction to slums and cause death of many people.

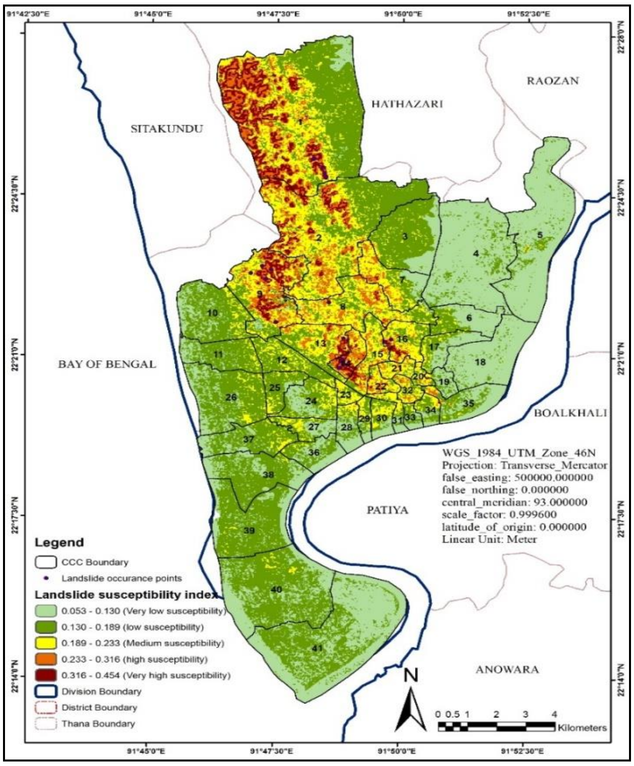

Fig 2**:** Landslide susceptibility map of CCC area

CONCLUSION

Hill cutting and heavy rainfall are prime factors for landslides in Chittagong that causes death to hundreds people with a great property loss. The present study is an attempt to see the efficacy of Analytic Hierarchy Process (AHP) and Geographic Information System (GIS) for landslide susceptibility mapping of the CCC area. In CCC, very high susceptible areas for landslide hazard are 6.5 sq. km and high susceptible areas are 11.45 sq. km which represents 4% and 7% of total CCC areas respectively. In these circumstances it would be rational to identify susceptible locations due to landslide hazard into city area which can provide supportive actions for preparing disaster management plan. Finally on the basis of the study it can be concluded that if the government and other concerned authorities take necessary steps, susceptibility of landslide hazards can be reduced to an extent tolerable to the city people.

ACKNOWLEDGMENTS

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TRENDS IN CLIMATIC VARIABLES IN NORTH-WEST REGION

A. Islam[1] & H. Zobeyer[1*]

[1] *Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <zobeyer@ualberta.ca>*

ABSTRACT

Climate change has turned into a global case of perturbation and the impact of these changes has been a matter of concerned discussion and research. Estimation, detection and prediction of trends and associated physical and statistical significance are salient features of climate research. In this study the trends of temperature, relative humidity, sunshine hour, wind speed and solar radiation have been analyzed for Rangpur and Rajshahi stations in the north-west region. Both parametric and non-parametric methods are used for testing the statistical significance of trends in different agro-climatic variables at these stations. The results of the analysis reveal that maximum temperature has decreasing trends of 0.1^{0}c and 0.3^{0}c at Rajshahi and Rangpur station. Minimum temperature has increasing trends of 0.4^{0}c and 0.01 at Rangpur and Rajshahi station. Relative humidity has increasing trends in most 10-day periods. The average increasing trends are 0.1% and 2.23% at Rangpur and Rajshahi station respectively. Sunshine hour has decreasing trends of 0.4 and 0.3 hours at Rangpur and Rajshahi stations. Average decreasing trends of wind speed at Rangpur and Rajshahi station are 18.6 km/Day and 32 km/Day. Solar radiation has a decreasing trend of 41.2 cal/cm^2.day per decade at Rangpur station.

Keywords: Trend, IPCC, Climate Change, climatic variables

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INTRODUCTION

Despite technological advances and improved crop varieties and irrigation system, weather and climate are still key factors in agricultural productivity. The fourth assessment report of intergovernmental panel on climate change (IPCC, 2007) shows a median increase of 3.3^{0}C in annual mean temperature throughout the Asia by the end of the 21st century. The intensity of weather events and changes in the amount and pattern of precipitation tends to increase due to rise in global temperature. SAARC Metrological Research Centre (SMRC, 2003) showed increasing trend of mean maximum and minimum temperature in some seasons and decreasing trend in some other seasons.

Divya and Mehotra (1995) observed an increase in mean annual temperature 0.4 °C in India for post-monsoon and winter season during the period 1901-1982. IWFM (2008) has analyzed the trends in measured sunshine duration at eight stations in Bangladesh for different seasons during 1961 to 2007. Persson (1998) analyzed fifteen years (1983-1997) data of 12 solar radiation stations network, showing a clear increasing trend in global radiation of 7.2%/decade within the BALTEX area of Sweden. Ahmed et al. (2007) reported a significant increasing trend of annual relative humidity by a rate of 0.13 (%) per year from 1923 to 2005 at Amman Airport Meteorological (AAM) station of Jordan. A decline in mean annual and winter wind speeds was observed by Tuller (2004) at four stations on the west coast of Canada. IWFM and CEGIS (2008) also reported that the precipitation would increase 1.56%, 13.19%, 10.49% and 2.51 % in the North-West, North-East, North-Central and South-West hydrological regions of Bangladesh.

Climate change has many considerable effects on cropping systems. Climate change will increase the temperature which will bring changes in farming pattern and affect crop yield. The extent of this work covers the analysis of trends in various climatic variables such as rainfall, temperature, relative humidity, sunshine hour, solar radiation and wind speed.

DATA COLLECTION AND SITE SELECTION

In this study, two stations in Rajshahi and Rangpur are selected and data of these meteorological stations like maximum and minimum temperature, relative humidity, sunshine hour, wind speed solar radiation, rainfall are collected from Bangladesh Meteorological Department (BMD) for 1961-2011. The North-West region was chosen as most of the agricultural return of our country comes from this region. The locations of the stations are shown in the Fig. 1.

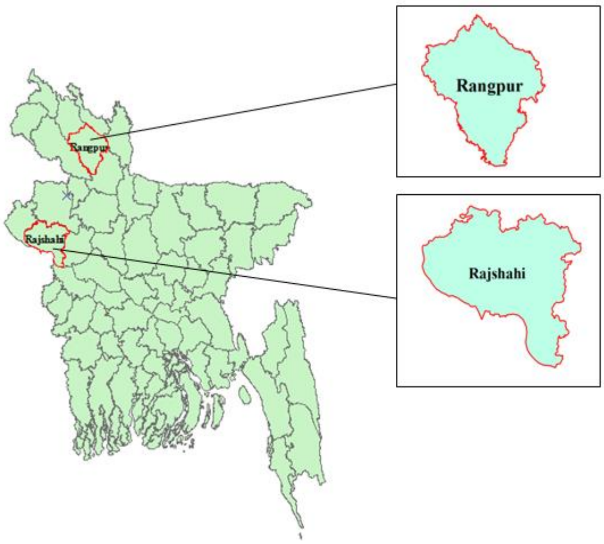

Fig. 1: Study Area

METHODOLOGY

An analysis of trend of climatic parameters is performed in SPSS through two methods- one is the parametric method and the other is the non-parametric method. Linear regression is the most basic and commonly used parametric method. Here a scatter plot of the dependent variable(Y) and the independent variable(X) is first made. A least square linear regression line is then superimposed to the plot. The fitted regression line is represented in Eq. (3).

$$Y=a+bx \qquad (1)$$

Where a and b are intercept and slope of the line means the trend of the given variable. In parametric method Pearson's correlation coefficient measures the correlation between two continuous variables. The following Eq. (2). is used to calculate the Pearson r correlation.

$$r = \qquad (2)$$

Where,
N= number of observation
$\sum x$ = sum of values under x variable
$\sum y$ = sum of values under y variable
$\sum x^2$= sum of squared values of x variable
$\sum y^2$= sum of squared values of y variable
$\sum xy$ = sum of product of x and y
Non parametric Mann-Kendall test (Helsel&Hirsch, 1992) has been conducted for significance test of trends of climatic variables. According to Eq. (3), the Kendall Tau_b coefficient is defined as:

$$\tau_b = \qquad (3)$$

nc= number of concordant pairs
nd= number of discordant pairs
X_O = number of pairs tied only on X variable
Y_0 = number of pairs tied only on Y variable

RESULTS AND DISCUSSION

It is seen from the Table 1 that maximum temperature has decreasing trends at Rajshahi and Rangpur station and they are, 0.1^0c and 0.3^0c per decade respectively. The average increasing trends of minimum temperature during the dry season are 0.01^0c, 0.4^0c per decade for Rajshahi and Rangpur. Relative humidity has increasing trends at both the stations. Sunshine hour has decreasing trends of 0.4 and 0.3 hours at Rangpur and Rajshahi station.The average decreasing trends of wind speed at Rangpur and Rajshahi are18.6 km/Day and 32 km/Day per decade respectively. Rangpur station showed a decreasing trend of solar radiation which is 41.2 cal/cm^2.day.

The values of Pearson's correlation coefficient as shown in Table 2 for maximum temperature lies between ± 0.30 and ± 0.49 for first 10-day of Nov and second and third 10-day of Jan which means a medium correlation. The other r values lies below $\pm$ .29, so it indicates small correlation exists between the two variables. According to Kendall's tau_b value the table shows that eight 10-day periods have increasing trends and the remaining thirteen 10-day periods have decreasing trends. The 1st 10-day period of Nov has significantly increasing trends at 5% significant level (significant level being less than or equal to 0.05) which means there are 95% probability that such trends are due to some genuine reasons and the rest of the seven have non-significantly increasing trends. The 2nd and 3rd 10-day periods of Jan have significantly decreasing trend at 5% significant level and the remaining eleven 10-day periods have non-significantly decreasing trends. For the remaining parameters of both the stations required for the statistical significance test for dry season can be found in the M.Sc. thesis of Islam (2014).

Table 1. Trend of climatic variables at Rajshahi and Rangpur stations.

Month	10day	Tmax		Tmin		Humidity		Sunshine Hour		Wind Speed		Radiation
		Rangpur	Rajshahi	Rangpur	Rajshahi	Rangpur	Rajshahi	Rangpur	Rajshahi	Rangpur	Rajshahi	Rangpur
Nov	1	0.012	0.023	0.025	0.007	-0.098	0.141	-0.007	-0.005	-2.21	-2.029	-2.617

	2	-0.004	0.013	0.029	-0.011	-0.046	0.175	-0.064	-0.039	-1.31	-1.198	-4.65
	3	0.006	0.006	0.039	-0.032	-0.098	0.096	-0.036	-0.004	-2.345	-1.724	-3.594
Dec	1	-0.002	0.012	0.053	0.005	-0.087	0.179	-0.093	-0.059	-1.353	-1.943	-4.483
	2	-0.009	-0.003	0.047	0.006	-0.065	0.168	-0.147	-0.093	-1.505	-1.146	-3.897
	3	-0.02	-0.019	0.048	-0.016	-0.056	0.176	-0.034	-0.055	-1.259	-1.263	-3.024
Jan	1	-0.048	-0.035	0.039	-0.043	0.021	0.181	-0.023	-0.025	-0.835	-1.656	-3.122
	2	-0.064	-0.044	0.027	-0.045	0.066	0.274	-0.109	-0.087	-2.061	-1.343	-4.566
	3	-0.042	-0.034	0.024	-0.033	0.021	0.242	-0.074	-0.065	-1.714	-1.38	-4.153
Feb	1	-0.006	0.012	0.041	0.006	-0.015	0.267	-0.048	-0.015	-2.325	-1.492	-3.227
	2	-0.023	-0.011	0.055	0.007	-0.035	0.285	-0.019	0.015	-2.223	-2.517	-3.31
	3	-0.039	-0.007	0.064	0.006	-0.016	0.275	0.041	0.016	-0.841	-2.502	-3.698
Mar	1	-0.011	-0.009	0.047	-0.006	-0.042	0.211	-0.021	0.038	-2.853	-2.733	-3.585
	2	-0.035	-0.018	0.062	0.0001	-0.008	0.295	-0.068	-0.021	-2.978	-3.593	-4.624
	3	-0.055	0.016	0.072	0.034	0.174	0.245	-0.004	-0.028	-2.138	-3.033	-4.332
April	1	-0.104	-0.033	0.055	0.026	0.215	0.388	-0.069	-0.057	-3.051	-4.04	-5.55
	2	-0.052	-0.008	0.046	0.033	0.096	0.203	-0.09	-0.007	-2.163	-5.92	-5.6
	3	-0.062	-0.009	0.023	0.002	0.116	0.216	0.005	-0.005	-0.86	-7.72	-4.565
May	1	-0.017	-0.028	0.038	-0.009	0.02	0.226	0.04	0.013	-2.682	-6.721	-4.237
	2	-0.026	0.006	0.039	0.027	0.017	0.206	0.074	-0.029	-0.765	-6.884	-4.8
	3	-0.02	-0.022	0.033	0.005	0.001	0.233	-0.013	-0.031	-1.689	-6.396	-4.843
Avg		-0.03	-0.01	0.04	-0.001	0.01	0.223	-0.04	-0.03	-1.86	-3.2	-4.12

Table 2. The correlations statistics of maximum temperature for Rajshahi station

Month	10day	Pearson's r	Kandell's Tau_b	Significance
	1	0.397	0.266	0.009
	2	0.189	0.148	0.147
Nov	3	0.105	0.123	0.229
	1	0.199	0.155	0.13
	2	-0.046	0.116	0.256
Dec	3	-0.208	-0.135	0.185
	1	-0.272	-0.143	0.165
	2	-0.404	-0.234	0.024
Jan	3	-0.302	-0.227	0.028
	1	0.102	0.063	0.544
	2	-0.087	-0.069	0.506
Feb	3	-0.054	-0.062	0.551
Mar	1	-0.081	-0.072	0.496
	2	-0.129	-0.107	0.307
	3	0.088	0.003	0.976
	1	-0.188	-0.143	0.172
	2	-0.043	-0.055	0.599
April	3	-0.046	-0.002	0.984
	1	-0.141	-0.066	0.531
May	2	0.032	0.128	0.221

	3	-0.144	-0.089	0.386

A scatter plot between climatic variables and years, with a superimposed linear trend line, is shown in Fig. 2.

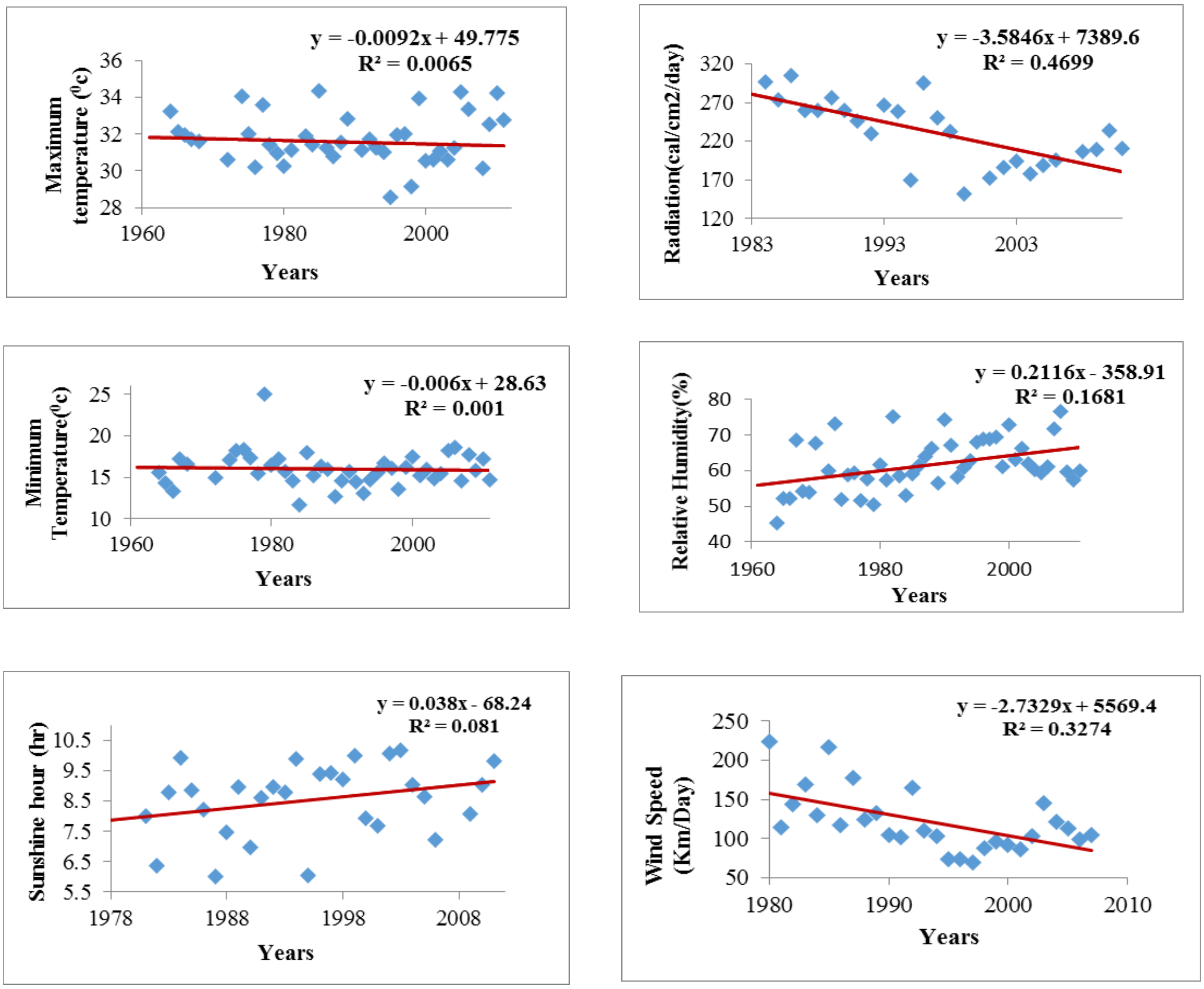

Fig.2: Trend of selected climatic variables in the 1st 10 day period of March at Rajshahi station

CONCLUSION:

Climate change on water resources is an important issue of this decade. An agricultural production and water use much more depend on climate change. By this study the trends in agro-climatic variables (temperature, relative humidity, sunshine hour, radiation, wind speed, rainfall) from 1961-2011 were analyzed. The average decrease in trends of maximum temperature in the dry season for Rajshahi and Rangpur are 0.1^0c and 0.3^0c per decade respectively. Minimum temperature has increasing trends which are 0.4^0c, 0.01^0c per decade at Rangpur and Rajshahi station. The average increasing trends of relative humidity are 0.1% and 2.23% per decade at Rangpur and Rajshahi station respectively. The average decreasing trends in sunshine hour at Rangpur and Rajshahi stations are, 0.4, 0.3 hours. Wind speed has decreasing tends at Rangpur and Rajshahi station which are18.6 km/Day and 32 km/Day respectively. The other important climatic variable is the solar radiation which has decreasing trends at Rangpur station.

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CHANGING PATTERN OF TEMPERATURE AND RAINFALL IN THE SOUTH EAST PART OF BANGLADESH

M. Reaz Akter Mullick[1] & Ridwan Mohammed Nur[1*]

[1] Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.
** Corresponding Author, <ridwanlink@yahoo.com>*

ABSTRACT

Climate Change and its impacts on the development issues such as agriculture, food security, ecosystem, health, wild life, society and overall economy have appeared as a global developmental concern for the last couple of decades. This paper aims to investigate the variation and trends of temperature and rainfall for Chittagong. To detect the trends, Sen’s slope method and the non-parametric Mann–Kendall test in combination with trend free pre-whitening approach for correcting the time series for eliminating the influence of serial correlation have been used. Daily maximum and minimum temperature and daily rainfall data for last 50 years (1964 – 2013) was collected and analyzed. Analysis for temperature shows increasing trend for the mean maximum and mean minimum temperature at a rate of 0.013°C and 0.015°C per year. Highest changing trend is observed in monsoon and pre monsoon season respectively. Rainfall analysis shows a decreasing trend in monsoon season, significantly increasing trend in pre-monsoon season with an overall increase of 1.48 mm per year. Trend in one day and consecutive three day maximum rainfall shows that the rate is increasing in case of three day rainfall. The trend of number of rainy days in Chittagong is slightly increased in pre-monsoon and decreased in the month of July. Paper concludes that change in climate in evident in Chittagong especially for temperature and extreme rainfall events.

Keywords: Climate Change, Trend Analysis, Mann-Kendall, Sen’s Slope, Pre-whitening, Chittagong.

INTRODUCTION

Bangladesh is currently recognized to be one of the most vulnerable countries in the world. Natural hazards originated from increased rainfall, rising sea levels, and tropical cyclones are projected to occur in higher frequency as the climate changes and all such events are extremely affecting agriculture, water and food security, human health and shelter. According to the Inter-Governmental Panel of Climate Change (IPCC) 5th assessment report, in terms of risks of increasing heat stress, there are parts of Asia where current temperatures are already approaching critical levels during the susceptible stages of the rice plant. In South Asia, seasonal mean rainfall shows inter decadal variability, noticeably a declining trend with more frequent deficit monsoons under regional in homogeneities. In South Asia, the frequency of heavy precipitation events is increasing (Rajeevanet al., 2008; Krishnamurthy et al., 2009; Sen Roy, 2009;), while light rain events are decreasing (Goswami etal., 2006). Spatially, the rainfall increase is stronger over northern parts of South Asia, Bangladesh and Sri Lanka, with a weak decrease over Pakistan (Turner and Annamalai,2012).It is projected that climate change will affect food security by the middle of the 21st century, with the largest numbers of food-insecure people located in South Asia(IPCC WRIIAR5). Murray et al. (2012) compared the response to cyclone Sidr in Bangladesh in 2007 and Nargis in Myanmar in 2008 and demonstrated how disaster risk reduction methods could be successfully applied to climate change adaptation.

Rainfall pattern will change due to global warming although the exact degree of change is not yet determined (SB Murshed 2009). This change will affect fresh water supplies that have already been stressed due to rising population and increased per capita consumption. This will cause more difficulty in estimating extreme rainfall events since there will no longer be a homogeneous series of values which can be extrapolated statistically. However, it is expected that higher extreme events will occur than before (Linarce, 1992).

Previous studied (Z Hasan 2014, MS Mondal 2013, M Noorunnahar 2013, S.Shahid 2010) in Bangladesh applied a Mann-kendall technique for the analysis of trend in temperature and precipitation variables. S.Shahid et al. (2010) analyzed Annual and seasonal rainfall and temperature of Bangladesh in last 50 year (1958–2007). Significant increases in annual and pre monsoon session and temperature increases significantly in monsoon season. The rainfall of Bangladesh have been increasing during the recent decades (Chowdhury et. al., 1997 and Karmaker et. al., 2000).The variability of rainfall (>22 mm/day) for summer has been studied by Mannan and Karmakar (2008). An analysis on summer monsoon rainfall over Bangladesh has been done by Ahasan et al. (2008), which focused on flood disasters. Islam (2009) estimated, from 34 meteorological climate sites in Bangladesh, that temperature increases over the past 100 years-for all Bangladesh of 0.62°C (maximum) and 1.54°C (minimum) occurred in February. Poulton and Rawson (2011) reported that temperature in Bangladesh increased over the past two decades by 0.035°C/year, if this trend continues, temperatures will have increased 2.13°C more than 1990 levels by 2050.

Whereas the non-parametric Mann-Kendall (Mann 1945, Kendall 1975) (MK) statistical test has been popularly used to assess the significance of trend in hydrological time series. The test requires sample data to be serially independent. When sample data are serially correlated, the presence of serial correlation in time series will affect the ability of the test to correctly assess the significance of trend. For this, Von Storch and Navarra (1995) suggest that the time series should be ‘pre-whitened’(PW) to eliminate the effect of serial correlation before applying the MK test. All previous work using Mann-Kendall test in Bangladesh, confuse the level of significance assess due to not apparent in Pre-whitening approach. Because the existence of positive serial dependence increases the probability of rejecting the no-trend hypothesis, it may cause trends to be detected that would not be found significant if the series were independent. PW effectively decreases the probability of rejecting the null hypothesis in the MK test.

In the present study, the nonparametric Mann-Kendall test in combination with trend free pre-whitening (TFPW) approach (Yue et al. 2002) for correcting time series for serial correlation have been used. The Mann-Kendall test was used to detect the trend and Sen’s slope method was used to determine the magnitude of change in the climate time series. Apart from temperature and precipitation, the present study analyzes trends in One day and consecutive three day maximum rainfall and number of rainy days trends as well.

METHODOLOGY

Data used

The observed daily temperature and rainfall data of Chittagong station of Bangladesh during the period 1964-2013 (50 years) were collected from the Climate Division of Bangladesh Meteorological Department (BMD). However from the meteorological point of view, there are four climatic seasons in Bangladesh. They are: Winter (December–February), Pre-monsoon (March– May), Monsoon (June–September) and Post monsoon (October–November). This seasonal classification as mentioned above found in several reports of SMRC and BMD.

Mean monthly minimum temperature (TMN), mean monthly maximum temperature (TMX), and precipitation (PPT) data were used in there. The time series of TMN, TMX and PPT the period 1964 – 2013. For maximum coverage of the station, and to ensure a uniform comparison period for all data types, an analysis period of 50 years spanning 1964-2013 was chosen.

Method of Analysis

Several parametric and non-parametric methods have been used for detection of trends (Zhang et al., 2006). Parametric trend tests are more powerful than non-parametric ones, but they require data to be independent and normally distributed. On the other hand, non-parametric trend tests only require the data be independent and can tolerate outliers in the data. One of the widely used non-parametric tests for detecting a trend in hydro-climatic time series is the Mann–Kendall (MK) test (Yue et al., 2002; Aziz and Burn, 2006; Gemmer et al., 2004).

One of the problems in detecting and interpreting trends in hydrologic data is the confounding effect of serial dependence. Specifically, if there is a positive serial correlation (persistence) in the time series, then the nonparametric test will suggest a significant trend in a time series that is, in fact, random more often than specified by the significance level (Kulkarni and Van Storch, 1995). The Mann-Kendall test is a ranked based approach that compares each value of the time series with the remaining values in a sequential or der (Hirsch et al. 1982). The test statistic S is given by:

$$S = \sum_{k=1}^{n-1} \sum_{j=k+1}^{n} \mathrm{Sgn}(x_j - x_k)$$

Where

$$\mathrm{Sgn}(x_j - x_k) = \begin{bmatrix} 1 & \text{if } (x_j - x_k) > 0 \\ 0 & \text{if } (x_j - x_k) = 0 \\ -1 & \text{if} (x_j - x_k) < 0 \end{bmatrix}$$

And xj and xk are the sequential data values, and n is the length of the dataset. A positive value of S indicates an upward trend, and a negative value indicates a downward trend. For samples >10, the test is conducted using the normal distribution (Helsel & Hirsch 1992), with the expectation (E) and variance (Var) as follows:

E[S]=0

$$\mathrm{Var}(S) = \frac{1}{18}\left[n(n-1)(2n+5) - \sum_{p=1}^{q} t_p(t_p - 1)(2t_p + 5)\right]$$

Where tp is the number of data points in the pth tied group and q is the number of tied groups in the dataset. The standardized test statistic (Z) is calculated as:

$$Z = \begin{bmatrix} \dfrac{S-1}{\sqrt{\mathrm{Var}(S)}} & \text{if } S > 0 \\ \dfrac{S+1}{\sqrt{\mathrm{Var}(S)}} & \text{if } S < 0 \\ 0 & \text{if } S = 0 \end{bmatrix}$$

Where the value of Z is the Mann-Kendall test statistic that follows a standard normal distribution with mean 0 and variance 1. Confidence levels of 90, 95 and 99% ($p < 0.10$, $p < 0.05$ and $p < 0.01$, respectively) were taken as to classify the significance of positive and negative temperature and precipitation trends.

RESULT AND DISCUSSION

Temperature

The maximum and minimum temperature of Chittagong station has been determined using historic available data from the all the meteorological stations of Bangladesh. Figure 01 shows month-wise distribution of the mean of Maximum and mean minimum temperature. Data within last 50 year period (1964-2013) of BMD is used to determine mean monthly temperature over Chittagong Station. Daily maximum data have shown its peak at 32.6°C during May. However, Daily minimum data have shown the highest rise of temperature of 25.2°C during June. Highest max. temperature 39.5°C found 27 May,2001.

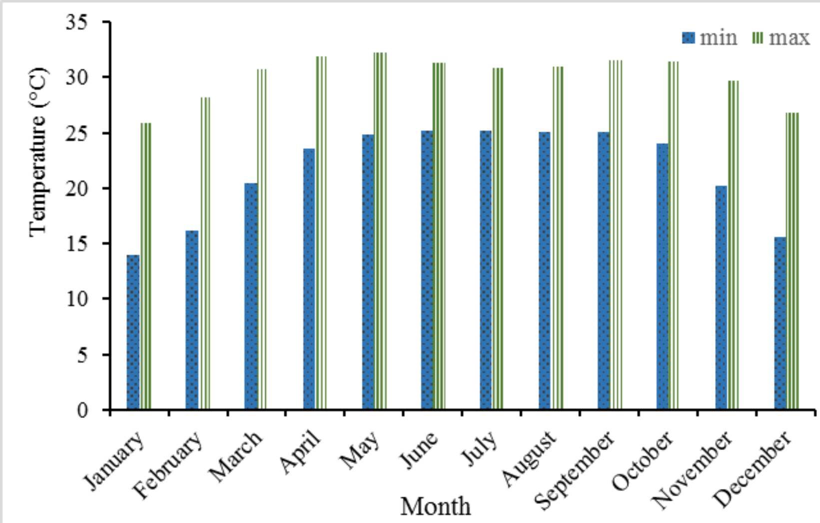

Fig. 01: Monthly average maximum and minimum temperature (°C) of Chittagong during the period 1964-2013

Trend of Monthly Maximum and Minimum Temperature

The trend statistics and Sen's slopes were computed for the TMN and TMX data for monthly base of Chittagong station and are presented in Table 1 and Table 2. It can be seen for TMN, most of the month is strongly significant trend except January, February, May and November which shows an insignificant trend. For all month increasing trend was observed. And the highest trend observed in March and December where the rate of increasing is 1.2°C per 50 yrs. and both are significant. For TMX, increasing trend were observed except May where the rate of decreasing 0.35°C per 50 yrs. Most of the month are significant trend except January, March, April, May, November and December which shows insignificant trend. And the significantly highest trend observed in August where the rate of increasing is 1.25°C per 50 yrs.

Table 1: Result of monthly minimum temperature trend at Chittagong from 1964-2013

Available Period (1964-2013)	**Mean**	**Standard Deviation**	**Mann-kendall Test value**	**P-Value**	**Sen's Slope (per yr in °C)**
Jan	13.95	0.89	0.58	0.56	0.006
Feb	16.20	0.87	1.42	0.15	0.012
March	20.41	1.09	2.07	0.03*	0.024
April	23.55	1.01	2.22	0.03*	0.023
May	24.78	0.84	0.76	0.44	0.007
June	25.23	0.63	2.79	0.005**	0.020

July	25.16	0.45	3.27	0.001**	0.018
Aug	25.11	0.49	4.51	0.0001***	0.023
Sep	25.11	0.39	3.66	0.0003***	0.015
Oct	24.00	0.58	2.36	0.018*	0.017
Nov	20.25	1.08	1.03	0.30	0.014
Dec	15.56	0.95	1.99	0.04*	0.024

Table 2: Result of monthly maximum temperature trend at Chittagong from 1964-2013

Available Period (1964-2013)	**Mean**	**Standard Deviation**	**Mann-kendall Test value**	**P-Value**	**Sen's Slope (per yr in °C)**
Jan	25.90	0.82	0.22	0.83	0.002
Feb	28.22	1.13	1.65	.099+	0.018
March	30.76	0.931	0.98	0.33	0.008
April	31.85	0.84	0.66	0.51	0.005
May	32.27	0.65	-0.94	0.35	-0.007
June	31.35	0.84	2.25	0.02*	0.023
July	30.82	0.71	1.71	0.09+	0.014
Aug	31.01	0.74	2.61	0.009**	0.025
Sep	31.55	0.58	2.11	0.034*	0.016
Oct	31.38	0.77	2.86	0.004**	0.023
Nov	29.64	0.97	1.63	0.103	0.016
Dec	26.74	1.04	1.32	0.186	0.022

Seasonal Trend of TMN and TMX

The trend statistics and Sen's slopes were computed for the TMN data for the different season of Chittagong station and are presented in Table 3. For all other seasons, increasing trends were observed. However, increasing trends with $p < 0.01$ were seen during monsoon (June – September) and pre monsoon (March-June) with $p<0.05$.On the other hand, the post monsoon (October – November) and winter (December-February) seasons are not significant. The pre monsoon and monsoon minimum temperature for the increased by 1.05 and 0.90°C per 50 yr, respectively, which indicates that Sen's slopes were greatest for the pre monsoon season in Chittagong.

Mann-Kendall test results and Sen's slopes of trends in TMX are presented in Table 4; the greatest trends with $p < 0.10$ were observed for the monsoon season. In the monsoon (June – September) season, Sen's slope were 1.1°C per 50 yr., The magnitudes of Sen's slopes clearly indicated that all 4 season were experiencing a strong warming trend, particularly for the monsoon season. An increasing trend for the pre monsoon (March – June) season, although these trends were found to be statistically insignificant. For the post monsoon season, although shows an increasing trend with $p<0.05$. In the winter season, the increasing trend with $p < 0.1$ and a trend slope of 0.55°C per 50 yr.

Table 3. Mann-Kendall test statistics (Z), p-values and Sen's slopes (Q) of trends in seasonal minimum temperatures at Chittagong from 1964-2013. Q: Sen's slope (trends are expressed as rate of change per year in degrees Celsius)

Season	**Z**	**P**	**Q**
Pre Monsoon	2.44	0.0147*	0.021
Monsoon	4.01	0.0001***	0.018
Post monsoon	1.57	0.116	0.009
Winter	1.53	0.126	0.012

Table 4. Mann-Kendall test statistics (Z), p-values and Sen's slopes (Q) of trends in seasonal maximum temperatures at Chittagong from 1964-2013. Q: Sen's slope (trends are expressed as rate of change per year in degrees Celsius)

Season	Z	P	Q (per yr)
Pre Monsoon	0.89	0.3735	0.004
Monsoon	3.47	0.0005***	0.022
Post monsoon	2.03	0.0424*	0.017
Winter	1.80	0.0719+	0.011

Rainfall

Annual rainfall in Chittagong is about 2832mm and approximately 74% of rainfall occurs during the monsoon. However from the trend analysis it has been observed that decreasing trend was founded in July, August and September. Increasing trend in March-June and October. None of them are statically significant except May.

Trend test results of precipitation are shown in Table 5. For the pre monsoon and post monsoon seasons, there were increasing trend, and post monsoon of them were statistically insignificant. For the pre monsoon (April – June) season, increase in precipitation at the rate of 223.95 mm per 50 yr. was observed with $p<0.05$. The monsoon season showed a decreasing trend, 12.15 mm per 50 yr., while the winter season no trend was observed.

Table 5. Mann-Kendall test statistics (Z), p-values and Sen's slopes of trends in seasonal precipitation at Chittagong from 1964-2013. Q: Sen's slope (trends are expressed as rate of change per year in degrees Celsius)

Season	Z	P	Q (per yr)
Pre Monsoon	1.98	0.0477*	4.479
Monsoon	-0.06	0.9522	-0.243
Post monsoon	0.94	0.3472	1.680
Winter	0.06	0.9522	0.000

From graphic illustration, the series of seasonal temporal changes of TMN, TMX, and PPT during the period 1964 – 2013 (Fig. 2) have been plotted. The observed seasonal temperatures indicated an increasing trend, particularly after 1990 (Fig. 2, TMN & TMX). The highest mean TMX was seen in 1999, (Fig. 2a, TMX). The lowest annual precipitation occurred in 1972.During Pre monsoon and monsoon seasons, precipitation occurred most, and therefore temperature had a major impact on the formation and transformation of water.

Table 6 shows the no. of rainy day occurred in July. And the highest season of rainy day is also staying in monsoon season and then pre monsoon season. Trend in rainy day found slightly increasing at a rate of 2.6 day 50 yrs. which is not significant.

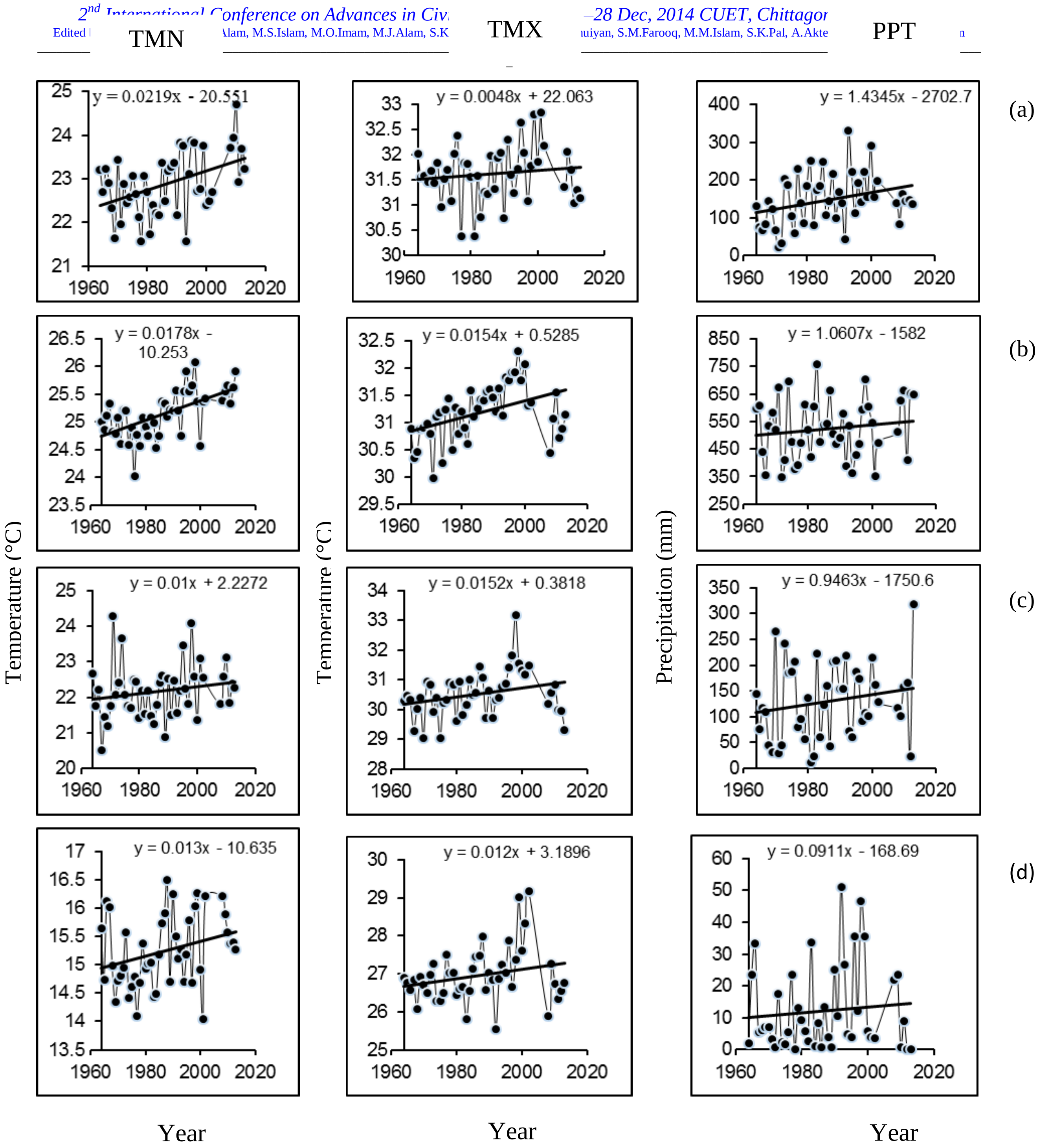

Fig. 02: Minimum (TMN) and maximum (TMX) temperature and precipitation (PPT) series of Chittagong station during the: (a) Pre-monsoon, (b) Monsoon, (c) Post-monsoon, and (d) Winter seasons on an annual basis over the period 1964–2013

Table 6: Monthly average number of rainy days in Chittagong during the period 1964-2013

Average number of rainy days (per month)	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**June**	**July**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
	1	2	3	6	12	19	22	21	17	10	3	1

Trend results of Rainfall in consecutive 1 day and 3 days max. shows the overall increasing trends is 1.74 and 2.2°C respectively. Highest changing trend in May in both. Decreasing trend were found in June to September in 1 day max. However in 3 days max decreasing trend found in July and September. None of them are statistically significant except May.

CONCLUSION

Daily temperature both maximum and minimum and the precipitation show increasing trends over Chittagong. Trend analysis shows an increase of maximum temperature of 0.0135°C and minimum temperature of 0.015°C in 50 years. The highest increase of maximum temperature has occurred in August at 1.25°C and minimum temperature has occurred in both March and December at 1.2°C in 50 years. Rainfall shows an increasing trend of about 1.48 mm per year. The highest increase and decrease of rainfall happened in May and in July at 3.8 mm and 2.3 mm per years respectively. Extreme rainfall events in Chittagong have increased remarkably over the last few decades. Such changes in temperature and rainfall indicate the climatic change of Chittagong area, which might carry several short to long run effects especially in the field of agriculture, health and ecosystem.

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DEVELOPMENT OF TRIP ASSIGNMENT SOFTWARE FOR AUTO TRIPS

M. S. Mamun[1*], J. Khalil[1], T. Islam[1], N. Haque[1] & M. A. Hossain[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh,*
**Corresponding Author, <smamun93@hotmail.com>*

ABSTRACT

In Bangladesh trip assignments are performed based on engineering judgment, which is totally arbitrary. This is in practice due to the fact that there is no cheap software available for trip assignment. The only available option is to use planning software packages like CUBE, EMME, TRANSCAD, VISUM etc., which are very expensive and needs extensive travel survey data. This research develops a framework for traffic assignment phase of transportation modeling to achieve user equilibrium condition for any given set of origin-destination travel demands and flow dependent link performance functions of a road network.

A C code has been introduced for the computation and analysis of user equilibrium based on Frank-Wolfe algorithm. The algorithm first find a decent direction then calculate the optimum step size, and converges after successive iteration. Direction is found by all or nothing method and step size is determined by bisection method. Arterial highway network of Sioux Fall, South Dakota consisting of 24 nodes and 76 links with 13 origins and destinations is analyzed for user equilibrium by using this developed software. The network is also analyzed by GAMS optimization software. The results obtained from the C code and GAMS software are identical.

Keywords: Trip assignment, User equilibrium, Auto trips, Four-step model, Frank-Wolfe algorithm.

INTRODUCTION

In modern age transportation plays a key role. At the heart of most transportation models stands the traffic assignment problem, which is to predict the route choice of travelers given the origin and destination of each traveler, under the assumption that each traveler seeks to minimize the time/cost associated with their chosen route (Bar-Gera, 1999). This research deals mainly with the modeling of traveler's route choice on the highway network, commonly known as the traffic assignment problem. For many years this problem was studied as a standalone problem. A first step towards a mathematical investigation of the problem has been done in 1952 by John Glen Wardrop, an English transport analyst. He developed the so-called Wardrop's first and second principle of equilibrium (Patriksson, 1994). The concepts are related to the idea of Nash equilibrium in game theory developed separately. In studies about traffic assignment, network equilibrium models are commonly used for the prediction of traffic patterns in transportation networks that are subject to congestion. The idea of traffic equilibrium originated as early as 1924, with Frank Knight. This research deals with the first Principle of Wardrop's User Equilibrium. A C code has been developed using Frank-Woolfe algorithm for the computation and analysis of user equilibrium on large regional transportation networks.

Although there are numbers of software (CUBE, EMME, TRANSCAD, VISUM, etc.) available in the market for traffic assignment, those are uneconomical for small project and country like Bangladesh where sufficient data are not available for using those software. The strategic transport planning in country like Bangladesh is done by taking some "Thumb Assumption" or using expensive software which may difficult to employ for lacking of available data or information. So for small and medium level research, this developed C code may be utilized for achieving User Equilibrium condition which will be helpful for policy making and transportation planning in future.

USER EQUILIBRIUM ASSIGNMENT

In User Equilibrium (UE) condition, no driver can unilaterally reduce his/her travel cost by shifting to another route. That means travel time on each used path between any Origin-Destination (O-D) is the same under UE condition. The following assumptions are made for UE assignment method:

- The user has perfect knowledge of the path cost.
- Travel time on a given link is a function of the flow on that link only.
- Travel time functions are positive and increasing.

A simple network consisting of two links having one origin-destination is shown in Fig. 1. If total travel demand from 1 to 2 is 12, we can easily determine UE flow condition. Suppose, link flow on link 1 is x_1 and link flow in link 2 is x_2. Under UE condition, travel time on link 1 (t_1) will be equal to travel time on link 2 (t_2) and x_1 plus x_2 will be equal to 12. Solving these two conditions will produce UE solutions.

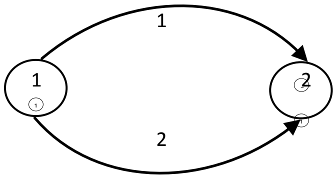

Fig. 1: Example network

However, large networks cannot be solved like network in Fig. 1, because there are so many O-D pairs and so many paths between each O-D pair that needs hundreds of equations to be solved simultaneously. In that case UE solutions can be achieved by solving following nonlinear mathematical optimization program (Sheffi, 1985):

Minimize $Z = \sum_a \int_0^{x_a} t_a(x_a)dx,$ (1)

Subject to,

$$\sum_k f_k^{rs} = q_{rs} : \forall r, s \quad (2)$$

$$x_a = \sum_r \sum_s \sum_k \delta_{a,k}^{rs} f_k^{rs} : \forall a \quad (3)$$

$$f_k^{rs} \geq 0 : \forall k, r, s \quad (4)$$

$$x_a \geq 0 : a \in A \quad (5)$$

where, k is the path, x_a equilibrium flows in link a, t_a travel time on link a, f_k^{rs} flow on path k connecting O-D pair $r - s$, q_{rs} trip rate between r and s.

The above mathematical program can solve any size network; however it needs optimization software to run with. In order to run without optimization software, Farnk-Wolfe introduced a heuristic method to achieve UE condition, on which a C code is developed in this research.

SOFTWARE DEVELOPMENT

A C code has been developed for the computation and analysis of user equilibrium based on Frank-Wolfe algorithm. The algorithm first find a decent direction then calculate the optimum step size, and converges after successive iteration. Direction is found by all or nothing method and step size is determined by bisection method.

Solution Procedure of Frank-Wolfe Method

Steps of general solution procedure of Frank-Wolfe method is as follows:

Step0: Free flow travel time, $t_a=t_a(0)$ of each link is determined and based on free flow travel time all-or-nothing assignment is performed (full demand of each O-D pair is assigned to the shortest path of the corresponding O-D pair) and total volume of each link (x^1_a) is obtained.

Step1: Travel time ($t^n_a=ta(x^n_a)$) of each link is updated.

Step2: Based on updated travel time all-or-nothing assignment is again performed and direction (y^n) is obtained.

Step3: By using bisection method step size is determined from line search. Find α_n that solves $\sum_a(y^n-x^n).t_a(x^n+\alpha_n.(y^n-x^n))=0$, Where value of α is 0 to 1.

Step4: Link volume is updated ($x^{n+1}=x^n+\alpha_n(y^n-x^n)$).

Step5: Convergence is tested based on

$$\frac{\sqrt{\sum_a(x_a^{n+1}-x_a^n)^2}}{\sum_a x_a^n} \leq 0.0001$$ stop, otherwise go to step 1.

Programming Procedure

The basic components of the program are described below. Due to space constraint the full programming code is not provided here.

(1) Initialization of the network: The whole network is initialized by calling the “initializeNetwork()” function.

(2) Input data from the file: All network data, O-D matrix, link travel time and capacity are read by calling the “readInputFromFile()” function.

(3) Determination of initial x value based on free flow travel time: This is done by calling “findXY()” function. From this function Dijkstra algorithm is first called and applied for all origin nodes. So all pair shortest path is determined and based on this all-or-nothing is applied inside the function and initial flow value is obtained.

(4)Updated travel time: Based on initial link flow travel time is updated.

(5)Determination of direction: This is done by calling “findXY()” function with updated travel time. First all pair shortest path is determined and then applying all-or-nothing direction (link volume) is obtained.

(6)Determination of step size: Step size is determined by calling “bisection()” function.

(7)Updated x value: Then flow value x is updated.

(8)Convergence test: If convergence test fail go to step (4), if pass then the loop breaks and printing output.

Running the Code

After compilation of the above mentioned code, executable file was obtained which can be used as standalone software to get UE solution for any kind of network. The users need to put all the necessary input files and executable file of the software in the same directory. After running the executable file one will get the UE link volumes in an out file.

NUMERICAL EXAMPLES

Arterial highway network of Sioux Fall (Fig. 2), South Dakota consisting of 24 nodes and 76 links with 13 origins and destinations is analyzed for user equilibrium by using this developed software. Free flow travel time and capacity of each link, and O-D travel demands can be obtained from the B.Sc. thesis paper by Haque et al. (2014).

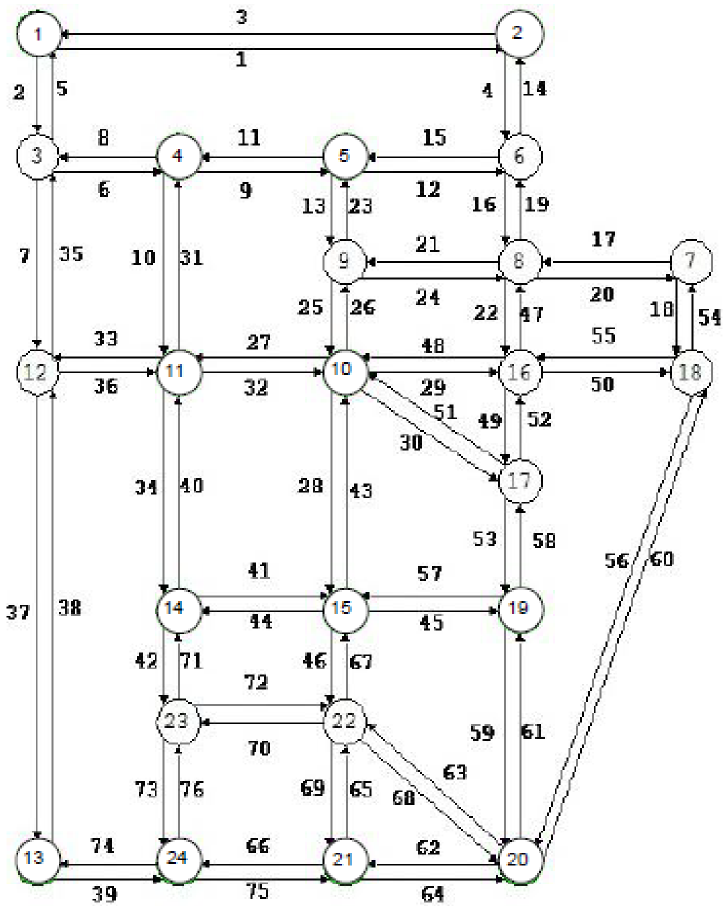

Fig. 2: Sioux Falls network

The network is also analyzed by GAMS optimization software. It is found that the results obtained from the C code and GAMS software are the same. The UE link flows of the Sioux Fall network are provided in Table 1.

Table 1: UE link flows of Sioux Fall network

i.j	LINK	x_a (10^3 veh/hr)		i.j	LINK	x_a (10^3 veh/hr)
1.2	1	3.92	1	13.24	39	9.4
1.3	2	10.55		14.11	40	10.642
2.1	3	3.92		14.15	41	4.977
2.6	4	10.36		14.23	42	6.894
3.1	5	10.55		15.10	43	7.128
3.4	6	6.92		15.14	44	4.352

3.12	7	10.71		15.19	45	5.549
4.3	8	6.92		15.22	46	8.68
4.5	9	12.48		16.8	47	3.39
4.11	10	5.89		16.10	48	1.53
5.4	11	12.228		16.17	49	3.39
5.6	12	6.88		16.18	50	1.53
5.9	13	6.08		17.10	51	1.92
6.2	14	10.36		17.16	52	3.39
6.5	15	6.88		17.19	53	4.869
6.8	16	8.84		18.7	54	3.6
7.8	17	3.6		18.16	55	1.53
7.18	18	3.6		18.20	56	5.13
8.6	19	8.84		19.15	57	4.38
8.7	20	3.6		19.17	58	5.31
8.9	21	1.85		19.20	59	5.146
8.16	22	3.39		20.18	60	5.13
9.5	23	5.828		20.19	61	4.418
9.8	24	1.85		20.21	62	5.27
9.10	25	7.93		20.22	63	2.7
10.9	26	7.678		21.20	64	4.978
10.11	27	4.46		21.22	65	8.582
10.15	28	7.38		21.24	66	11.15
10.16	29	1.53		22.15	67	8.972
10.17	30	1.479		22.20	68	2.234
11.4	31	6.142		22.21	69	8.29
11.10	32	4.019		22.23	70	3.25
11.12	33	4.43		23.14	71	7.36
11.14	34	10.831		23.22	72	2.784
12.3	35	10.71		23.24	73	4.11
12.11	36	4.43		24.13	74	9.4
12.13	37	9.62		24.21	75	11.15
13.12	38	9.62		24.23	76	4.11

Total system travel time per hour is 1,118,400 veh-minutes.

CONCLUSIONS

A code has been developed to achieve UE solutions for any network. This code can be utilized in many purposes. In Bangladesh sufficient data are not available for proper utilization of the conventional software and they are expensive too. The C code that is developed in this research may have an extensive impact in the research area as it is easy and free to use. Enthusiastic students can use this code for their research purpose. User Equilibrium condition is a specific condition of the traffic network where the travel time of each path between an origin and a destination is equal. This particular equilibrium condition of the traffic network have importance in "Transportation Modeling" and "Planning" for evaluating future transportation facility development and implementation of new policy. This C code also can evaluate the impact of a new road in a transportation network. When a new road is developed some drivers switch to the road from their regular path which may cause change in the link flow in the other road in the existing traffic network. This change in the link flow can be measured or evaluated by the developed method in this research.

Further modification of this procedure could be done by adding “System Optimal” condition which is based on Wardrop's second principle. The procedure showed in this paper is valid for auto trips only. Therefore, students or professionals can modify this programming procedure for transit trips and combine them to make more complete software.

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EXAMINING THE EFFECT OF INTERSECTION ANGLE ON THE SAFETY BEHAVIOR OF LEFT TURNING VEHICLE DRIVER AT SIGNALIZED INTERSECTION

Hasina Iasmin[1*], Hisashi Kubota[1] & Aya Kojima[1]

[1]*Graduate School of Science and Engineering, Saitama University, 338-8570, Japan.*
Corresponding Author, <hasina_iasmin@yahoo.com >

ABSTRACT

Pedestrians and cyclist are vulnerable road users and their safety is at high risk than other road user. Conflicts between pedestrians/cyclists and turning vehicles, especially left-turners (left-hand traffic system) are very common at signalized intersection. As left turning vehicle make a turn along the corner of an intersection, intersection geometry has an effect on the movement of a left turn vehicle. Intersection angle is an important element of intersection geometry and no study is done yet on pedestrian safety considering intersection angle. The purpose of the study is to analyse the effect of intersection angle on the behaviour of left turning vehicle with pedestrians or cyclists by using traffic Conflict analysis method: video observation of risky behaviour. In this study events related with sudden brake for avoiding collision are considered as conflict. For measuring the severity of these events Swedish traffic conflict technique (Time to accident/Conflict speed Value) is used. About 6 hours video data were collected from nine signalized Intersection considering three typed (<90º, 90º, >90º) intersection angle in Japan. Results showed that serious conflicts between left turners and pedestrians or cyclists are more at obtuse angled intersections than other typed intersections.

Keywords: Turning Vehicles, Pedestrian safety, Conflict technique, Intersection angle

INTRODUCTION

At signalized intersection Pedestrian "WALK" signals are functioned for pedestrians to get to the other side of the road by stopping the conflicting motorized vehicles. For traffic operational efficiency right- and/or left-turning vehicles are often allowed to perform their maneuvers during the pedestrian "WALK" signal indication (Fig. 1). Although signalized crosswalks are operated to give pedestrians prioritized right of way, accident data reveals that turning vehicles are involved in most of the accidents at signalized intersections. Making turn in an intersection is a weak point for many drivers. Approximately one out of five accidents at signalized intersections involves a turning vehicle hitting a pedestrian (ITARDA, 2014). The split between left-turning and right-turning accidents is about 60/40(right hand traffic system) (ITARDA, 2014; JSTE, 2002) . In Japan 49% pedestrian accidents occurred during the five year period from year 2008 to 2012 at signalized intersection. Among which 7.8% fatalities are took place between left turning vehicle and pedestrian (Robertson and Carter, 1984). Among many reasons invisibility, intersection geometric layout, road user behavior are notable. Visibility from within the vehicle (due to a pillar) and poor driving habits are the factors responsible for most of the difference between left turn and right turn accidents (Zegeer et al., 1982).

Fig. 1: Interactions between left turning vehicle and pedestrian

To improve the safety of signalized intersections it is important to find the reasons for which the behavior of turning vehicles is affected in detail. So that it will be possible to control the behavior of road user. The Manual on Intersection Accident Countermeasures of Japan (Habib,1980) suggests modifying intersection corner geometry to improve safety performance regarding accidents between left turning vehicles (left hand traffic) and pedestrians. These measures clearly suggest that understanding the effects of intersection corner design elements on the turning maneuvers of vehicles is essential, as left turning vehicle has to turn along the corner of an intersection. Some previous studies identified some issues considering intersection geometry for which a considerable variability in turning path of vehicle was found (Leden, 2002; Wael et al., 2013). But these studies did not provide any information about the effect of such variation on pedestrian safety.

Intersection angle is a very important design element of intersection corner. Intersection angle means the angle at which two roads are intersecting to each other. The "normal" intersection (Fig. 2) consists of two streets intersecting and crossing at near-90° angles. However, there are many instances where the intersection angle of crossing streets is not close to 90°. These intersections are termed "skewed intersections". Skewed intersections can be grouped into two categories shown in the figure, (Fig. 3, 4), the "right skewed" intersection and the “left skewed" intersection. The left skewed intersection is skewed such that the obtuse angle created (the one greater than 90°) is to the left of a driver on the minor road approach. The right skewed intersection is skewed such that the acute angle created (the one smaller than 90°) is to the left of a driver on the minor road approach.

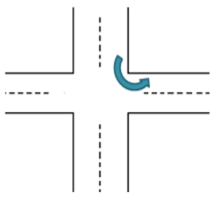

Fig. 2: Normal intersection

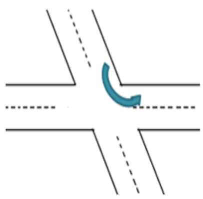

Fig. 3: Right skewed intersection

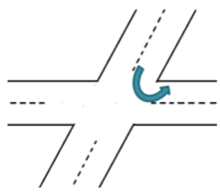

Fig. 4: Left skewed intersection

No study is done yet considering this issue on the safety of pedestrian. The main objectives of this study is to analyze the safety effects of intersection angle on left turning vehicle-pedestrian/cyclist interaction in signalized intersection by using traffic conflict analysis method: video observation of risky behavior. Which intersection angle is dangerous for pedestrian? The definition of turning movements throughout this study is based on left-hand traffic system.

METHODOLOGY

This section describes the methods used in this study. The first three sub-sections summarize the site description, data collection and data extraction. The final sub-section presents the analysis method, i.e. conflict analysis method by using TA-CS graph.

Observation site description

For observing non-yielding behavior of left turn driver nine signalized intersection in Japan was choose. The main point of this selection is three type of intersection angle (Fig.6). From each type three intersections are choose for taking video of left turn vehicle and pedestrian/cyclist interaction. In all intersection left turn vehicle are permitted to make turn through the crosswalk when pedestrian signal is green. Other characteristics are tried to keep similar (see Table. 1).

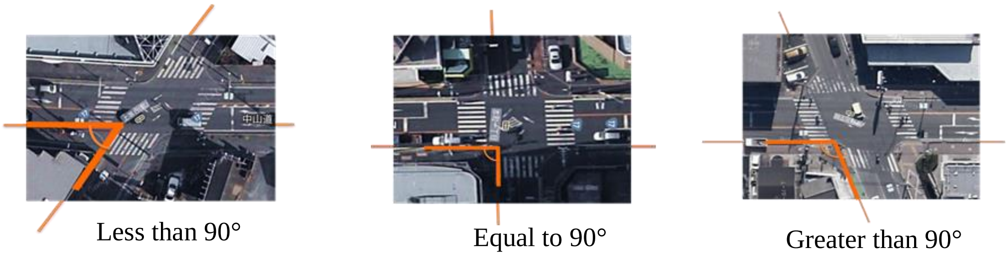

Fig. 6: Three type of intersection based on intersection angle

Table 1: characteristics of all intersections

Angle	Sl. No	Name of intersection	Intersection angle	No of lane (major road)	Road width (m) (major road)	No of lane (minor road)	Road width (m) (minor road)
<90º	1	Kamiochiai	60 º	3	8	2	6
	2	Myamachi	75 º	3	8	2	6
	3	Minamimae kawa	70 º	3	8	2	6.5
=90º	4	Near kitaurawa station	90 º	3	9	2	5.25
	5	Rokukenmon dori	90 º	3	9	2	7
	6	Honmachi icchome	90 º	3	8.5	2	6
>90º	7	Sasame sanchoume	120 º	3	10	2	7.5
	8	Suzuya intersection	113 º	3	10.5	2	6
	9	Omiya keisatsusho Iriguchi	110 º	3	9.25	2	6

Data collection

One video camera was used to obtain visual data during the permissive left turn phase (Fig. 7). One approach from each intersection was taken. Data were collected from one weekday during 7.30am to 1.00pm from each intersection. Total 49 hour 30 min video was taken from all intersections.

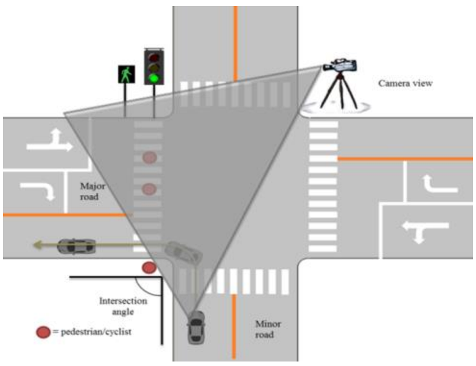

Fig. 7: observation site and data collection

For extracting the event related with accepting small gap with high velocity, accepted and rejected gap size by left turn vehicle was measured from video. For extracting gap acceptance data it is important to know where and when a driver decides to accept or reject an available gap. Since a precise determination of this decision point is very difficult. When a driver reaches near the crosswalk of minor road he should give a look on the crosswalk of major road, it is assumed that when a left turn driver reaches the crosswalk of minor road he takes decision to go or not to go (Fig 8). For measuring the severity of this type of event speed at conflict area was extracted from video.

Fig. 8: Assumed decision point of left turn driver

To select the most severe situations created by sudden brake, the approach of the Swedish Traffic Conflict Technique is used. For using this technique, Distance to collision point (d) and Conflicting speed (CS) was extracted from video.

For extracting time, distance, velocity data from video, video analyzing software (Kinovea) was used (Fig. 9).

Conflict analysis method

Traffic conflict techniques have been extensively discussed in the literature. It was first proposed by Perkins and Harris (Perkins and Harris, 1968). Conflicts were defined as those events involving swerving, braking or traffic violations. There was no scaling of conflicts by Severity. A formalized definition of a traffic conflict was later adopted as “an observable situation in which two or more road users approach each other in space and time for such an extent that there is a risk of collision if their movements remain unchanged” (Hyden, 1987). Various conflict indicators have been developed to measure the severity of an interaction by quantifying the spatial and temporal proximity of two or more road users.

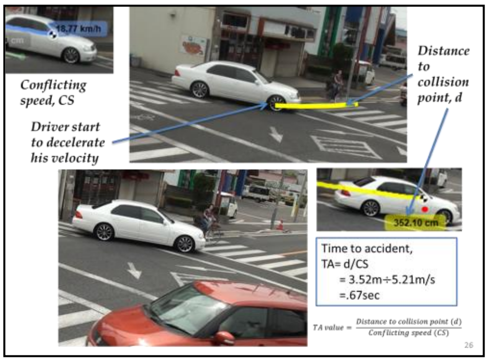

Fig. 9: Extracting data using software

Sometimes driver comes with a very low velocity and if he found any pedestrian or cyclist on the crosswalk he make a sudden brake, which may not be so dangerous. To select the most severe situations created by sudden brake, the approach of the Swedish Traffic Conflict Technique is used (Hyden, 1987). This technique is developed at Lund University. In Swedish traffic conflict study they use TA-CS graph (Fig. 10) to show the severity of each sudden brake event.

TA is the time that remains from one of the road users have started an evasive action, until a collision would have occurred if the road users had continued with unchanged speeds and directions.

The TA value can be calculated based on the estimates of distances d and conflicting speed CS.

$$TA\ Value = \frac{Distance\ \ to\ collision\ \ point\ \ (d)}{Conflicti\ \ ng\ speed\ \ (CS)} \qquad (1)$$

Where, d = Distance to collision point = is the remaining distance between the point where car takes evasive action (sudden brake) and the potential point of collision. The conflicting speed (CS) is the speed of the involved road user at the moment when the evasive action (sudden brake) starts.

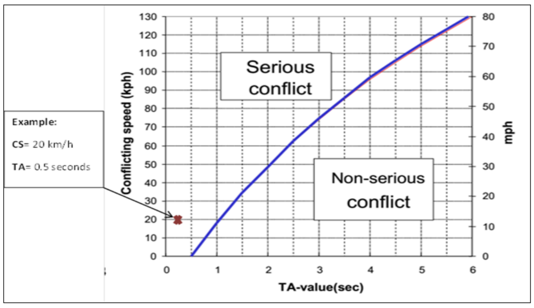

Fig. 10: severity of Swedish traffic conflict technique

RESULTS AND DISCUSSIONS

Table 2 shows the sample of observation of left turn vehicle driver and pedestrian/cyclist interaction.

There are total 22 sudden brake occurred in all intersections. Whereas 16 sudden brakes are occurred at obtuse (greater than 90 degree) angled intersection and 6 are at 90 degree angles intersection. No sudden brake was found at less than 90 degree angled intersection (see table 2).

Table 2: Observation samples of left turn vehicle driver and pedestrian interaction

Type of intersection	Acute angled	Right angled	Obtuse angled
No of total interaction	38	52	78
No of sudden brake	0	6	16

Several previous studies found that vehicle speed when a crash occurs (Crash speed) significantly contributes to the severity of the crash (Kloeden et al., 2001; Kruysse, 1991; Wand and Abdel, 2008). Fig 11 shows the severity of each sudden brake event by using TA-CS graph. In this graph speed of vehicle at conflict point and remaining distance of all sudden brake event are plotted. Serious events are defined by considering high velocity with less distance. In the graph it is clear that sudden brake event at 90 degree angled intersection are in non-serious conflict zone and 10 out of 16 sudden brake at greater than 90 degree angled intersection are in serious zone. There is no sudden brake occurred at less than 90 degree intersection. Drivers at less than 90 degree intersection or equal to 90 degree intersection are very much careful about their turning, so they don't interact with any pedestrian or cyclist. If they try to interact, these interactions seem not so dangerous. Whereas drivers at greater than 90 degree intersections tend to cross the crosswalk faster before a pedestrian or cyclist come. Reason is that drivers at greater than 90 degree intersection can easily turn.

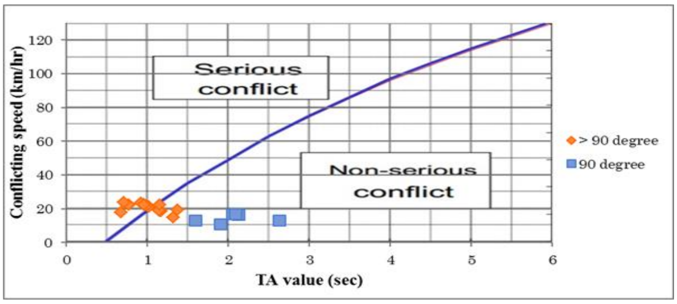

Fig. 11: Seriousness of sudden brake which is taken by left turn driver using TA-CS graph

CONCLUSION

For traffic operational efficiency it is difficult to provide separate traffic signal phase for left turn vehicle and pedestrians. In this type of situations driver’s yielding behavior towards pedestrian is very necessary. When driver faces any pedestrian or cyclist on the crosswalk he should give priority to that pedestrian or cyclist first. But always driver don’t show this type of behavior towards pedestrian or cyclists. As left turning vehicle make a turn along the intersection corner. So intersection corner design may have some influence on the behavior of left turning vehicle driver. The purpose of this study was to find the effect of intersection angle on the safety behavior of left turn driver. For evaluating the safety effects of different intersection angle TA-CS graph based conflict study was done. The conflict study confirmed the negative effect of greater than 90 degree intersection angle on left turn vehicle driver and pedestrian interaction. The results showed that driver at obtuse angled intersection showed more non-yielding behavior. At intersection with intersection angle greater than 90 degree, left turn vehicle driver can turn easily than acute angled intersection. This easy movement gives a driver confidence of passing the conflict area quickly. Some risky driver takes this advantage and accept small gap, which is unsafe for pedestrian/cyclist. Driver’s sight of view is very wide at obtuse angled intersection. For this reason driver can take decision of turning before he reach the crosswalk of minor road. This can be another reason of this type of non-yielding behavior. Keeping this matter in mind further study will be necessary for finding a solution of this problem.

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IMPLEMENTATION OF CONGESTION PRICING: A SOLUTION FOR BANGLADESH

S. M. R. Rahman[1*], M. R. Mannan[1] & M. S. Mamun[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <rahat.austce@gmail.com>*

ABSTRACT

In order to abate the negative consequences of traffic congestion, implementation of congestion pricing is a matter of time demand nowadays. Congestion pricing will help to encourage the use of public transportation facilities. This paper conducted a comprehensive overview of congestion charging systems in different countries and reviewed its benefits. The study considered congestion charging experiences in Central London, Singapore, Stockholm and Hong Kong. According to our review across the developed countries it is found that congestion price can reduce congestion by 15 to 25% and increase public transportation ridership by 20 to 40%. Travel speed is almost same as the percentage of reduced traffic volume. It is also seen that car trips in city centres are dropped by 20 to 70%. Implementation of congestion pricing reduces the emission of greenhouse gases more than 15%. In developed cities implementation of congestion pricing is easy because of proper funding and well organized political environment. But for developing countries like Bangladesh it is a challenging issue due to insufficient funding, political restriction and lower socio economic condition. Although there are many difficulties in Bangladesh, congestion pricing is one of the potential solutions to alleviate traffic congestion problems. After evaluating worldwide congestion pricing success congestion charge is a reality based project to overcome congestion problem and promote public transportation facilities.

Keywords: Congestion pricing, Traffic congestion, Public transportation.

INTRODUCTION

Increased population will add new traffic and increasing travel demand. Due to unplanned existing road network these traffic decrease free flow speed of vehicles and make the road network congested. New infrastructures have been built to tackle the situation. But due to insufficient funding it is not possible to fulfill the required demand. Developing infrastructure also a time consuming issue and sometimes there will exist no other way to build new roads. A potential solution like congestion pricing system will help to reduce traffic congestion. Congestion pricing may be implemented without increasing supply. Congestion pricing has been recommended as a long time competent way to minimize the road congestion by the transport economists and traffic planners.

Among economists, no utility should be free of cost for e.g. public good. Market failure will be happened when a utility is free. Here road surface and time is regarded as a utility, so its uses have to be charged. As congestion pricing is very popular perception only few developed countries implemented it practically. Developing countries not yet adopted this concept.

This paper regards the charging experiences in London, Singapore and Stockholm and Hong Kong pilot program that (Hong Kong) did not conduct full-scale implementation. The advantages and objectives of pricing schemes are reviewed for the best use of congestion pricing in developing country like Bangladesh. It is seen that, congestion pricing can reduce congestion as well as increase public transportation ridership. Travel speed also increases with the reduction of car trips in city center. Emission of green house gases also decreased which is one of the big environmental reliefs.

CONGESTION PRICING

Congestion pricing is defined as a charging method used in urban city to reduce congestion during rush hours. Vehicles especially private cars, goods vehicles, taxis, motorcycles have to pay a fee for entering into a charging zone. Congestion pricing can also be implemented for the best use of public vehicles like buses, metros.

Worldwide used congestion pricing methodology

Cordon pricing

Cordon pricing is a type of congestion pricing that would be implemented in the city center to restrict vehicle movement by assessing a charge during peak travel period.

Toll system

Road toll is collected to recover the construction work and maintenance of road as well as to reduce congestion especially in highway roads.

Dynamic volume pricing

Dynamic volume charging system is mainly the replacement of existing high occupancy vehicle lane with a high occupancy toll lane.

Network pricing studies

Network pricing system is implemented rarely for the whole network based on either how much area of the system is used by a vehicle or considering for how long it uses the system.

REVIEW OF THE PAST WORK

Numerous researches have been conducted based on different countries on congestion pricing and its many issues, including de

Congestion pricing and public transport (PT)

The main obstruction for successful implementation of congestion pricing is acceptability. Promoting more public transport (PT) can be the remedy of acceptability. Many authors have inscribed these issues in their research.

According to Larson and Sasanuma (2010), if PT is inconvenient suburbanites have to bear full cost of congestion charge (CC).

Jones (1998) discussed that acceptance is the main obstacle to introduce congestion pricing. One of the main reasons for incorporating Bus Rapid Transit (BRT) and congestion pricing is public acceptance.

Armstrong-Wright (1986) emphasized that if congestion charging is implemented PT has to be sufficient and comfortable for conducting motorists who have diverted to PT due to congestion charging.

Jaesirisak et al. (2008) also stated that for public acceptance and creating PT very effective, congestion pricing and PT should be integrated.

Eliasson et al. (2009) revealed that congestion charging made it easier for modal switch from car to PT.

According to Pahaut and Sikow (2006), increased revenue cannot be benefitted if congestion pricing is negatively impacted. This negative impact can be minimized, if the revenue earned from congestion pricing will invested in BRT for instance. So congestion pricing and BRT may be integrated together for the proper congestion extenuation strategy.

OBJECTIVES AND ACHIEVEMENTS OF DIFFERENT CHARGING SCHEMES

The objectives and achievements of different congestion pricing schemes like Singapore, England, Sweden and Hong Kong are discussed below.

Singapore

Singapore adopted paper based Road Pricing Scheme (RPS) on an expressway (East Coast Parkway) in June 1995. RPS was by the by extended to other expressways. The objective of RPS was to minimize congestion on the expressways at the time of office rush hours and to introduce Singaporeans with both linear passage tolls and road charging outside the Central Business District (CBD) (Goh, 2002). RPS covers the initial entry to the 5.96 km^2 central area of the city from 7.30 – 10.15 hours. This scheme charged US$2 per business day on any given weekday for every vehicle which contained 3 or fewer people. Vehicles reduced about 45% after applying this scheme for each business day (Larson and Sasanuma, 2010). Entry of car reduced by 70% and average vehicle increased from 19 – 36 km/h (Santos, 2005). Bus ridership increased about 20 % for this scheme (Ed Pike, 2010).

Electronic Road Pricing (ERP) replaced RPS on 1 September 1998. The aim of ERP was to maintain average speed of 45-65 km/h on expressways and 20-30 km/h on major roads (Santos, 2005). The charging area of ERP is categorized into central business districts (CBD's), which is also included to the previously covered area by RPS. Charging applies from 7.30 – 19.00 hours in CBD's and 7.30 to 9.30 hours in expressways or outer ring road. A radio transponder named In-vehicle Unit (IU) is used in which a Cash Card is inserted. A fee is deducted from the Cash Card when the vehicle passes the restricted zone and displays the remaining balance. Overall traffic had dropped to 15 % during whole day and 16 % during peak periods. If compared with RPS there is no momentous effect of ERP because RPS already motivated people switchover to public transport.

London, England

The London Congestion Charging Scheme (LCCS) became operational in 17 February 2003. The aim of LCCS was to minimize congestion in and around the charging zone (Santos, 2005). LCCS covers 22 km^2 in Central London which is the 1.3% of Greater London. It performs from Monday – Friday, from 7.00 – 18.30 hours, except public holidays. The charge was £5 per day for all motor vehicles. Some vehicles are not charged, such as – certain military vehicle, local government service vehicles etc. During charging hours the total vehicles entering central London dropped about 25% the day congestion pricing were implemented (Larson and Sasanuma, 2010). Car trips reduced about 50-60%, as car users diverted to PT (Santos, 2005). Traffic speeds increased between 10 and 15% (Ison and Rye, 2005). During the first year bus passengers entering the charging area elevated to 37% (Larson and Sasanuma, 2010). Buses increased in charging zone by 20% (Beeversand and Carslaw, 2005). Carbon dioxide emissions have minimized more than 15% (Larson and Sasanuma, 2010).

Stockholm, Sweden

Stockholm experienced an exception because congestion charging turned off after the implementation from January 3 – July 31, 2006. The objective of that system is to reduce congestion, increase accessibility and improve the environment (Eliasson et al., 2009). Approximately 1.4 – 8.5 US$ was charged to motor vehicles entering the inner city of Stockholm from 6.30 – 18.30 hours on weekdays. Volume of traffic reduces by 25% after implementing the pricing scheme (Larson and Sasanuma, 2010). Car trips were minimized by 20% to/from inner city and for this PT travel was increased by less than 9% (Kottenhoff, 2009). Work trips by car reduced by 24% across the cordon area (Franklin et al., 2009). Vehicle emission reduced between 10 to 15% due to reduction of vehicles. From the inner city it found that carbon-dioxide emission decreased to 14% (Eliasson et al., 2009).

Hong Kong

Hong Kong implemented trial Electronic Road Pricing (ERP) from July 1983 – March 1985 and integrated Automatic Vehicle Identification with vehicles holding electronic number plates fixed under the vehicle (Dawson and Brown, 1985). To examine the economic, technical and utility of ERP Hong Kong adopted road pricing and it is seen that private cars and taxis are the most dominated vehicle in the Central Area, i.e. 76% (Hau, 1990; Harrison, 1986). The study examined that 40% car trips reduced because of diversion to PT (Ed Pike, 2010). Furthermore it is also estimated that the gross environmental betterments in the pricing zone were estimated to 12-16% (Ed Pike, 2010). Average vehicle speed improved about 40% in urban areas (Hau, 1990). Car trips will reduce approximately 20-24% in the charging zone and overall vehicle speed increased to 10% (Hau, 1990).

Overview summary

From our review it is seen that private cars are the main source of congestion and public transportation system is the most effective alternate of private cars to reduce the congestion. In our study we have studied on 4 countries and from our study we invented that Asian countries are more concern about congestion pricing than that of European countries. Singapore is the first country which started congestion pricing for the very first time in the world and Hong Kong is the second country which adopted pricing system to overcome congestion problem. Before concluding this section, Table 1 gives an overview of LCCS in Central London and ERP in Singapore which are the most successful charging schemes over the world.

Table 1: Summary Table of LCCS and ERP

	LCCS	ERP
Area covered	22 km^2	not applicable
Number of entries	174	45
Reduction(%) in private cars in the zone	33(daily)	15
Increase(%) in average speed in the zone	14-21	no change
Reduction(%) of CO_2 emission	>15% (CO_2)	not available

From the objectives of the charging schemes it is clear that congestion reduction is the primary objective. In, Figure 1 has shown the comparison of Singapore, London and Stockholm.

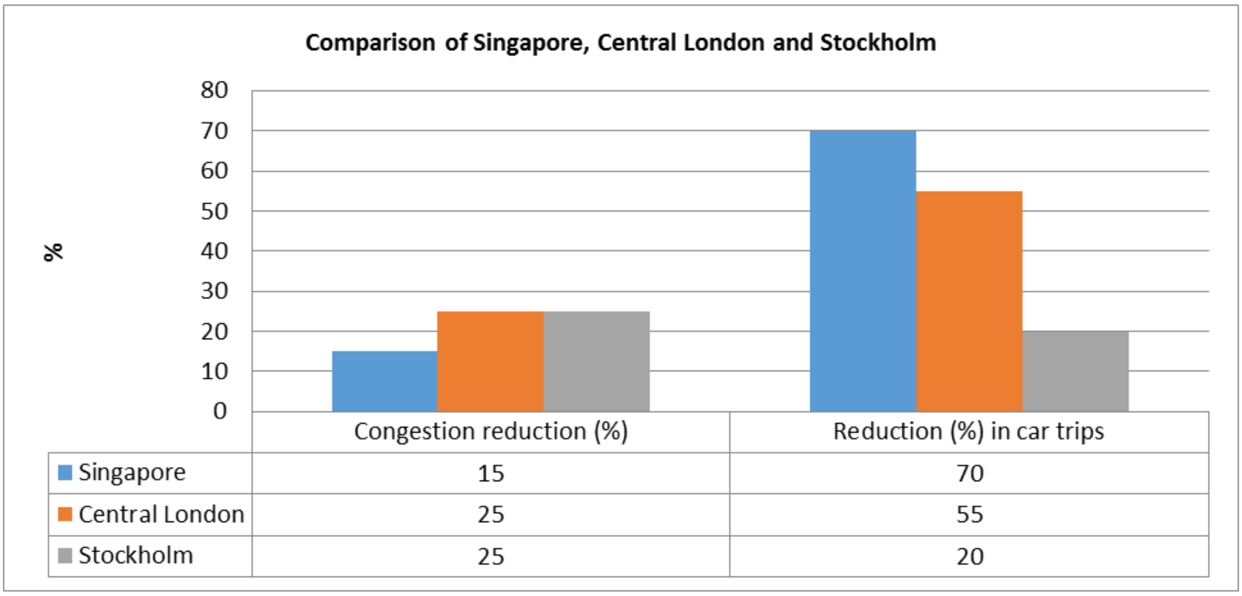

Fig. 1: Comparison of Congestion Reduction in Singapore, Central London and Stockholm

An effective reduction of car trips in the charging zone is possible only for good PT system. Singapore achieved a greater reduction of car trips because of ERP system. The reduction depends mainly on well established PT system as well as good charging technology.

SCOPE OF CONGESTION PRICING IN BANGLADESH

Dhaka the capital of Bangladesh facing a severe congestion problem and whole traffic system in this city is in worse condition. To make the situation better government has commenced different projects; such as BRT, MRT and elevated expressway in Dhaka. As it is seen from previous researches congestion pricing is a very effective system, so it can be implemented in Dhaka with BRT and MRT as an integrated manner.

Motijheel area is the heart of Dhaka city, as it is the business and commercial center of Dhaka city. So it is one of the most congested areas in the city. From our study it can be assured that to reduce the congestion in this area cordon pricing scheme would be helpful. This cordon area should cover 4.69 km^2 area and vehicles have to pay a fee to enter this cordon zone during rush hours like office hours.

In Bangladesh congestion in highway is very common. To reduce congestion in highway toll collection system for specific lanes or would be very effective. It will not only reduce congestion but also help to make a fund for future maintenance work of the road and also used for constructing new lane.

DISCUSSION AND CONCLUSION

This paper explores worldwide achievements of different congestion pricing schemes. It is seen that congestion charging will help to reduce congestion during rush hours. Dhaka is the most populated city in Bangladesh with an unplanned road network. So congestion pricing would play a significant role in reducing congestion all over the Dhaka city. But public transportation facilities were not yet well established the city. In that case congestion pricing would be unrealistic without ensuring good transportation facilities. The project of BRT and MRT would be a great initiative has recently taken by the government and it would be helpful to implement congestion pricing.

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KEY ISSUES OF RAIL SAFETY IN BANGLADESH

Hasib Mohammed Ahsan[1*], Mutamid Billah Azzacy[2], & Md. Shajedul Islam[2]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka 1000, Bangladesh.*
[2] *Gas Transmission Company Limited, Petrobangla, Dhaka 1000, Bangladesh.*
**Corresponding Author, <hmahsan@ce.buet.ac.bd>*

ABSTRACT

In Bangladesh, though railway is considered as the safe mode of land transport of the country, rail network still not connected with all the major districts and cities. Beside, existing railway route length is decreasing instead of increasing. In addition, both passengers and freight movement were decreased continuously to a substantial amount for about thirty years after liberation and trying to revive its position for passenger service in the most recent years. Furthermore, Bangladesh Railway (BR) has been experiencing a large number of train accidents since its development in 1972. The total number of accidents from 2001-2010 was approximately 5938. Among these, most of the accidents were train derailments, with others as collisions, fire in trains and train run into obstructions. From 2001-2010, about 5461 train derailments were taken place, which is approximately 92% of the total incidence. Disruption to services, substantial financial losses and safety risk to staff and passengers are highly obvious from such occurrence. Overall analysis shows that Bangladesh Railway lacks in using advanced technologies in railway operation.

Keywords: Rail Safety, Bangladesh Railway, Derailment, Collision

INTRODUCTION

Increasing demand for transport, increasing road congestion, increasing concern over safety and environmental matters, and technology-led cost reductions have given rising interest in railways for inter-urban, urban and even rural transport services. Many countries, both in the developed and developing worlds, are finding that rail transport is cost effective at meeting some of the growth in demand through the construction of new lines or more effective and efficient use of existing lines and services as well as provides a safe, efficient means of transport with minimal environmental impact.

However, the provision of safe and reliable services is a fundamental requirement of the railway. Passengers are entitled to expect to travel in safety and on time where Staffs are entitled to work in safe conditions (Department of Transport, 2007). Thus, Safety can be stated as the timely transit of passengers and goods without risk of casualties and damage which is basically the product of good practices at all levels of functioning i.e. design, manufacturing, maintenance and operations (Amitabh, 2004). Though, railway transportation system is recognized as a safest mode of land transport around the world, railway accident has always been the major challenge for rail safety as well as a point of attention to the engineers and researchers (Agarwal, 2005; Brabie, 2005; Wu and Wilson, 2006; Ramesh and Kumar, 2011).

Very few research work has been carried out on Bangladesh railway regarding train accidents to determine the underlying causes and development of effective countermeasures to enhance railway safety. Hence, the main objective of this paper is to present the key issues of rail safety in Bangladesh using available statistics and information.

RAILWAY SAFETY AND TRAIN ACCIDENTS

Railway Safety

Around the world, railways have come to be recognized as a safest mode of mass transportation because of its inherent characteristics. Therefore, safety becomes the foremost issue while transporting man and materials in railway system. To ensure railway safety around the world all the activities includes according to the three-tier approach as follows (New Zealand, 2011).

Education: includes (i) Advertising, (ii) Publicity and media relations, (iii) Awareness raising events and campaign, (iv) Development of education resources for schools (v) Publication and display of rail safety pamphlets brochures (vi) Training of etc.

Engineering: includes (i) Ensuring structural and functional integrity of the infrastructure and its subsystem (Track improvements, periodic maintenance, level crossing up gradation, Enhanced track inspection technology etc.) (ii) Ensuring structural and functional integrity of the rolling stock, (iii) Ensuring appropriate operational procedures and information management for effective train handling etc. (improved signaling and interlocking system).

Enforcement: includes (i) Application of appropriate warnings or prosecution against those who fail to obey the rules and regulations.

Train Accidents

A train accident is termed as any occurrence which does or may affect the safety of the railways, its engine, rolling stock, permanent way, works, passengers or servants which either does or may cause delays to trains or loss to the railways (Arora, 2006; Mundrey, 2010).

TRAIN ACCIDENTS SCENARIO IN BANGLADESH

Bangladesh Railway (BR) has been experiencing a large number of train accidents since its development in 1972. The total number of accidents from 2001-2010 was approximately 5938. Among these, most of the accidents were train derailments, with others as collisions, fire in trains and train run into obstructions. Train running into obstruction mainly includes train accidents at Level crossing. Accident types from 2001-2010 are shown in Table 1.

Table 1: Train Accidents in Bangladesh Railway (July 2000-June 2010)

Year (July-June)	Collisions	Derailments	Fire in Trains	Train Run into Obstructions	Total
2000-01	5	510	-	37	552
2001-02	14	624	3	67	708
2002-03	13	482	2	27	524
2003-04	8	723	-	23	754
2004-05	7	592	30	78	707
2005-06	3	790	-	37	830
2006-07	1	510	-	17	528
2007-08	3	419	-	25	447
2008-09	7	408	-	34	449
2009-10	2	403	-	34	439
Total (%)	63 (1.1)	5461 (91.9)	35 (0.6)	379 (6.4)	5938 (100)

Source: Bangladesh Railway, 2008; 2009; 2010.

Trend of Rail Accidents

Fig. 1 shows that accidents trend in Bangladesh railway is very dramatic with several ups and downs throughout the period 1998-99 to 2009-10. The minimum number of accidents counts in the fiscal year 1998-99 was 358 and the peak in the year 2005-06 with a value 830.

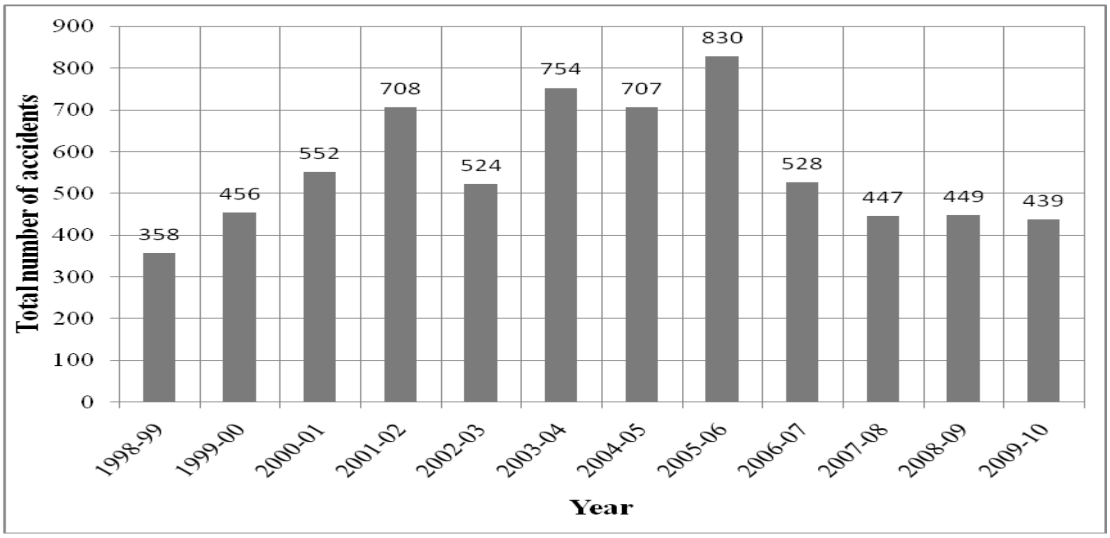

Fig 1: Trend of Total Reported Rail Accidents (July 1998- June 2010)

Casualties in Rail Accidents

These accidents from 1998-99 to 2009-10 caused 368 deaths and 2449 injuries and compensation cost about 14.4 million. Numbers of human casualties in rail accidents are the key indicator of the severity of railway accidents. The reality is that railway fatalities and serious injuries are almost entirely attributable to collision and level crossing accidents. Due to data limitation it is not possible to represent casualties for various rail accidents but the whole picture of railway fatalities is shown in Fig. 2.

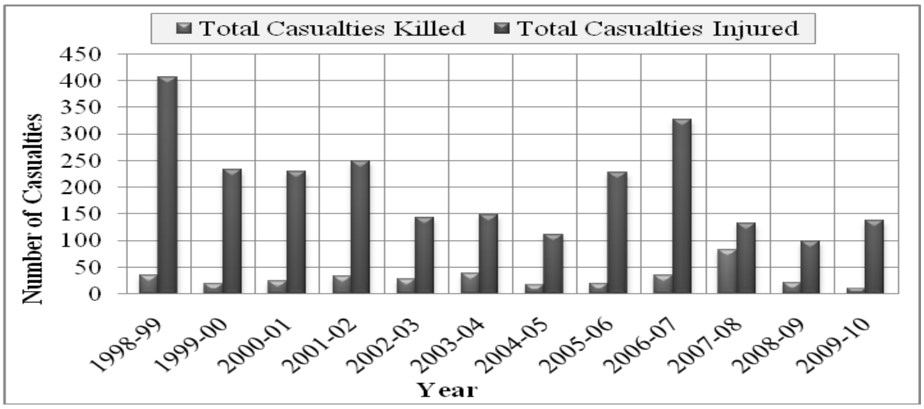

Fig 2: Total Casualties for Railway Accidents (July 1998- June 2010)

CAUSES OF MAJOR RAIL ACCIDENTS

Derailment Casues

Table 2 shows a various causes of train derailments (study period 2005-06 to 2010-11) from which 'top 10' causes are highlighted. It is found that the cause 'spread of gauge' was on the top of the list which shares 45% of train derailments. From this analysis it is also evident that old and worn rail, defective sleeper, inadequate ballast, insufficient fittings-fastenings etc. were found to be the potential factors for infrastructure caused derailments. Therefore, it can be said that only strengthening of the track by providing adequate fittings-fastenings can reduce train derailments to a considerable amount. However, operation related derailments resulted from improper loading-unloading, sudden speed controls, wrong settings or manipulations of points. Again, rolling-stock caused derailments aggravated by axle and wheel defects, failure of spring, bogie and suspension, defective buffer, coupling failure, equalizing beam broken etc.

Table 2: Distribution of derailments by causes

Sl No.	Elements Category	Causes of Derailments	Derailments	
			Nos.	(%)
1	Infrastructure	Spread of Gauge	1361	45.0
2	Infrastructure	Large Number of Worse Sleeper	321	10.6
3	Infrastructure	Shrinkage of Track	207	6.8
4	Rolling-Stock	Sudden Speed Control	165	5.5
5	Infrastructure	Level Difference in Track and Tight Gauge	93	3.1
6	Operational	Wrong Settings & Manipulations of Points	83	2.7
7	Infrastructure	Breaking and Wearing of Rail	56	1.9
8	Rolling-Stock	Axle and Wheel Defect	52	1.7
9	Operational	Breach of Rules, Signal Violation	50	1.7
10	Infrastructure	Defects in Points and Crossings	45	1.5
11		Others & Miscellaneous Causes	589	19.7
		Total Numbers of Derailments =	3022	100

Source: Ahsan, HM; Islam, MS and Azzacy, MB. 2014.

Level Crossing Accidents Causes

It is observed from the Table 3 that people are responsible for almost 99 % of Level Crossing accidents. Study shows that the violation of road traffic rules at railway-highway grade crossing by road users is the main cause for the level crossing collisions. Unguarded, unprotected, unauthorized, and busy level crossing without safety measures also fall within this category (ESCAP, 2000; Huang, 2006).

Table 3: Causes of level crossing accidents (study period 2005-06 to 2010-11)

Causes of level crossing accidents	No.of Repetition	Responsible people	Injured	Killed
Careless train operation	1	Loco Master & Asst. Loco Master		
Pumping due to heavy load	1	Track Failure		
Signal given without closing LCG & Traffic rules violation by Container Lory	1	Gate man & Road User		
Traffic rules violation by Driver of different types road vehicle	248	Road Users	337	180

Under Investigation	1				

Source: Ahsan, HM; Azzacy, MB and Islam, MS. 2014.

OTHER RELEVANT ISSUES

Manpower Status

Bangladesh Railway has huge shortage of manpower now-a-days. Status of manpower in every department of BR is decreasing after the period 1969-70. During the 40 years of independence total manpower of Bangladesh Railway reduced almost 50% (from 55825 in 1969-70 to 27971 in 2009-10) instead of increasing which has paralyzed railway services along with the old, limited and ramshackle condition of tracks and rolling stocks.

How It is Managed & What Way We Move

Some plimpses of concern are reflected in the following photographs.

CONCLUDING REMARKS

In Bangladesh, rail accident becomes as everyday’s incidence which hampers the safety of passengers and goods and creates serious disruption in railway transport systems. Bangladesh railway has severe drawbacks in infrastructure development, regular maintenance of existing tracks and operation and thus need immediate strengthening of the age old track, up-gradation of engine, coaches and wagons along with recruitment of sufficient manpower's especially for in engineering and operation sectors. Railway managements should adopt a policy of manning currently unprotected crossings and equipping them with inexpensive locally manufactured barrier and warning systems to avoid the most dreaded level crossing accidents. Moreover Bangladesh Railways should carry out intensive social awareness, on a regular basis, to educate road users, villagers and motor vehicle drivers and make them aware of the provisions of motor vehicles act and railway act.

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NEED FOR A SUSTAINABLE PUBLIC TRANSPORT IN DHAKA, THE CAPITAL CITY OF BANGLADESH

Hasib Mohammed Ahsan[1*]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET), Dhaka 1000, Bangladesh.*
**Corresponding Author, <hmahsan@ce.buet.ac.bd >*

ABSTRACT

The rapid rise in population along with increased and versatile land use patterns, and increase in motor vehicle ownership along with non-motorized vehicles on streets have resulted in enormous environmental pollution, traffic congestion and accident problems in Dhaka city. At present, the population of Dhaka is around 12 million and is expected to rise alarmingly in the foreseeable future. Dhaka’s transport demand, as well as in other major cities in the country, is still predominantly met by non-motorized modes and despite large growth in the number of motor vehicles, its level of motorization is still far below compared to other Asian countries. Inadequate road infrastructure, lack of traffic policy and management practices, absence of a dependable public transport system along with somewhat uncontrolled manner of land use development have resulted a chaotic traffic situation in Dhaka. Disparities between urban transport demand and supply are increasing day by day. The annual economic wastage occasioned by traffic congestion and traffic accidents is estimated to be about 1% of GDP.

A well-planned public transport system or in other words a mass transport system, preferably rail based, in consistent with the population and land use changes is considered to be the key solution to Dhaka’s transport problem. Such a system will encourage people to live in the suburbs and attract from the drive-alone mode. It is, therefore, needed to explore a technically, economically, and politically sustainable option of public transport for Dhaka city to provide a satisfactory level of accessibility, mobility, and safety.

Keywords: Dhaka, Capital City, Transport Modes, Demand, Supply

INTRODUCTION

Dhaka, the capital city of Bangladesh contributes about 15% of the national GDP, but is ranked as one of the poorest in the world in terms of Per Capita GDP (BBS, 2001). It has developed into the capital from a mere provincial capital since the birth of Bangladesh, unfortunately in an unplanned way. Moreover, Dhaka is one of the only seven cities in the world which has experienced urban population growth higher than 2.4% in between 1975 to 2005 (United Nations, 2006). As a result, public investment in urban transport infrastructure may rank very low compared to the need for other public goods such as health care, education, housing and sanitation.

The rapid rise in population along with somewhat uncontrolled manner of land use, increase in motor vehicle ownership along with non-motorized vehicles on streets, inadequate road infrastructure, lack of traffic policy and management practices, absence of a dependable public transport system – importantly mass transit in particular have resulted a chaotic traffic situation in Dhaka. In general, Rapid growth, low incomes, and extreme inequality are among the fundamental reasons of transport problems in Dhaka, similar to every other megacity of developing countries (Ahsan and Hoque, 2002; Pucher et al., 2005; Hossain, 2006).

URBAN DEVELOPMENT

Population

Dhaka with current population of almost 15 million people in an area of 1,528 km² (about 17 million in the Greater Dhaka) has been growing at astonishing levels since the independence. By 2020, the megacity's population is expected to rise to 20 million and 25 million in 2025. It is also one of the most densely populated cities in the world, with more than 45,000 people per square meter in the core area (ALG, 2011). Per capita income averages around US$ 900 per year, and around 30 percent of the population lives in miserable conditions, with very poor access to transport services (Bangladesh Economic Review, 2011).

Land Use Changes

The urban activity components such as; business, industry, residential, etc. zones in the old part of Dhaka are uniformly distributed and thus can be termed as unstructured area. Activity zones in the new part expanded following the urban dynamic process and the suburbs basically developed as residential zones to accommodate the surplus population in and around the CBD. Centre for Urban Studies (CUS) classified the land use of Dhaka City into five categories based on a land use study in 1995 as in Table 1 (DCC, 2004).

Table 1: Categories of Land Use in Dhaka City.

Land Use	% of Total Land
Residential	45
Commercial and Industrial	15
Administrative and Institutional	20
Roads and Transport	10
Open space	10

Road Network Development

The major roads in the old part of Dhaka have been developed in the east-west direction. Major roads in the new part have gradually been developed in the north-south direction with link roads in the east-west direction. An irregular pattern of road network has developed, rather than more efficient pattern as grid-iron or radial-circumferential pattern. The road network has emerged with wide primary roads, but narrow secondary and tertiary roads due to lack of planning and building controls (see Table 2). There are no effective bi-cycle lanes and safe walkways, and the footpaths are occupied in great proportion by vendors and others. Most of signals are manually controlled by police without properly coordinated automated systems.

Table 2: Road network within Dhaka City Corporation (DCC) area

Type of Road	Definition	Length (km)	(%)
Primary	Minimum of 31m wide, 6 lanes of 3.25m, footpath and median	68.45	5.29
Secondary	Minimum of 25.50m wide, 4 lanes of 25m, 2 NMV lanes of 2m wide, footpath and median	108.20	8.37
Connector	Minimum of 22 m wide, 2 lanes of 3.25m each, 2 NMV lanes of 2m each, footpath and median	221.35	17.12
Local	Minimum of 8.75 m wide, two lanes of 1.36m and footpath (1.5m each way)	573.75	44.37
Narrow	Minimum of 4.5 m wide with two lanes of 1.36m	321.27	24.85

Total	1293.02	100

PASSENGER TRANSPORT CHARACTERISTICS

Vehicular Growth

Bangladesh has been experiencing rapid vehicular growth in recent years. Around 300 new vehicles are coming to road every day. Total number of registered motorized vehicles has become 17,51,834 in June, 2012 increasing from 7,03,215 in 2003 (more than 200% increase in less than 9 years). But astonishing fact is, more than 40% of all registered vehicles are in Dhaka. The trend of motor vehicle growth in Dhaka is shown in Fig. 1 below (BRTA, 2012; Hoque et al, 2012).

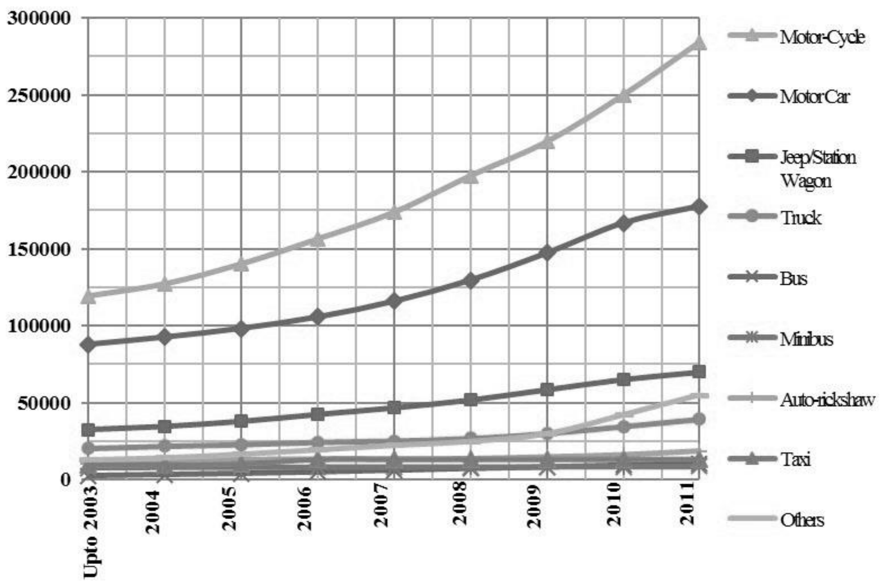

Fig. 1: Motorized vehicle growing trend in Dhaka.

Modal Shares

In many respects, the distribution of modal choices in Dhaka is unique among cities of comparable size in the Asia region. Almost 60% of the 8.5 million weekday person trip share walk trips and about 19.2% use rickshaw (tri-cycle). For the remaining 20% trips on motorized modes, 1.4% use auto-rickshaw (three wheeler), 9.2% travel by bus, 3.1% by private car, and the other 7.1% by various other modes (DITS, 1994). The latest study (ALG, 2011) estimated that on an average day 21 million trips are taking place in Dhaka metropolitan area. Despite the rapid growth of motorised traffic in Dhaka, non-motorised transport still remains the dominant mode for the city dwellers with more than 40% by walking and rickshaw (see Table 3).

Table 3: Modal share in metro Dhaka

Mode	Percentage of Share				
	DITS *(1994)*	DUTP *(1997)*	JBIC Study *(1999)*	STP *(2005)*	JICA Study *(2009)*
Walk	60.1	62.82	62.05	14.0	19.09
Rickshaw	20.1	20.04	13.28	34.0	38.19

Bus	12.8*	10.42*	10.22	44.0*	29.83
Auto-rickshaw			5.83		5.73
Passenger Car	7.0**	6.72**	3.97	8.0**	4.30
Others			4.65		2.86
Total	100	100	100	100	100

* Transit; ** Motorized (Non Transit)

The modal distribution by income groups shows that trips on foot is made by the low income group, 73% while most of the rickshaw trips are made by the middle income group, 59% (JICA, 2010). The significance of walk and rickshaw trips is clearly evident as they relate to 97% of the city dwellers. Further study (Mannan and Karim, 2001) shows that about 80% of walk trips are less then 20 minutes, while 90% of trips by non-motorized mode (walk and rickshaw) are less than 40 minutes. Trips lengths of motorized modes are about two times compared to non-motorized modes. The average expenditure of households on transportation is 10.8% of monthly income and about 80% of the people used rickshaw because of availability and about 62% used because of door-to-door service (DUTP, 1996).

PRESENT BUS NETWORK AND SERVICE

Relative Growth of Bus and Minibus

Despite the increasing demand for public transport, bus and minibuses have not grown substantially. There are 11,060 buses and 8,583 minibuses registered (as of June, 2012) which combinedly represent only about 3% of total motorized traffic. Though the number of large buses remained nearly constant, the percentage share of bus fleet (buses and minibuses combined) has been in fact declining day by day due to decrease in number of minibuses (see Fig. 2).

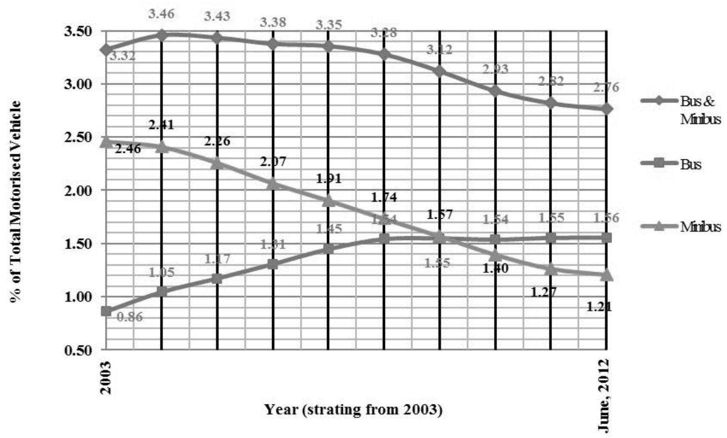

Fig. 2: Percent Growth of buses and minibuses in Dhaka.

Bus Route Layout

According to Bangladesh Road Transport Authority (BRTA), the number of bus routes in Dhaka is 160. Only 12.5% of the entire road network is suitable for bus services (ALG, 2012). In consistent with road development, most of the bus routes are also in the north-south direction (ALG, 2011). Satellite towns like Savar, Narayanganj, Gazipur, Narsingdi, Demra etc. are connected to Dhaka by sub urban bus services mostly being minibuses and within the city areas these buses illegally operate as city services. Areas close to the riverside, mostly part of old Dhaka, have almost no bus supply and high density of buildings is found without following any specific pattern amongst narrow streets. Around 55% of the total metropolitan area is unserved by buses (Dainichi and others, 1998).

Poor Bus Services

Dhaka is perhaps the only city of its size without a well-organized, properly scheduled bus system or any other mass transport system (Ahsan and Hoque, 1991). Women and urban poor are particularly disadvantaged in accessing the existing facilities due to extreme overcrowding. The private sector is dominating and providing a monopoly service (95% of total bus services). Bus operation is characterized as far short of the desirable mobility needs of the people in terms of reliability, comfort, speed and safety (Hoque et al., 2012; Rahman, M. and Nahrin, K. 2012). There are many concerns related to the current bus services, among them are: Long waiting time, Non availability of a seat and over-crowding, Inefficient bus fare and ticketing system, Unfavorable condition inside the bus, Long delay time, Physical harassments of the female passengers, Misbehavior of the bus staff, location of the bus station and accessibility, Poor facilities of the bus stops and Non availability of information about bus services.

THE WAY FORWARD FOR PUBLIC TRANSPORT

Revitalization of public transport is a core issue in the context of rapid motorization. Improving quality of public transport, increasing public transport capacity and thus relieving traffic congestion are the significant strategies. The Strategic Transport Plan (STP, 2005) focused predominantly on formulating strategies for the development of transport infrastructure over the next 20 years. STP underscores large size of transport investment needs in Dhaka and recommends a program that includes three Bus Rapid Transit (BRT) routes, three Metro Rail routes and fifty highway projects with a total investment of US$ 5.5 billion (see Fig. 3).

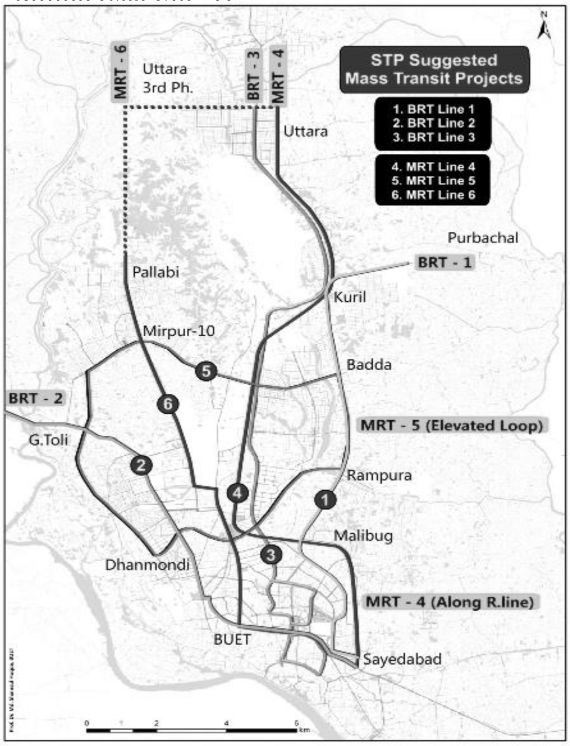

Fig. 3: Proposed BRT and MRT Corridors

CONCLUDING REMARKS

The existing transportation infrastructure in Dhaka is unable to bear the growing traffic demand. The level of service and options of transport modes are not at all convenient for the passengers and neither for the environment. One form of mass transit supply is needed with potentially high capacity and which also has the potential of encouraging people to live in the suburbs and attract from the drive-alone mode. Literature show that the worldwide growth of population and urbanization and the accompanying problems of congestion, environmental disruption and pollution along with improved underground excavation and tunneling methods point to the need for using underground space.

Indeed, future actions would require renewed governmental as well as organisational commitments. Supports, essentially fund and collaboration, from international agencies and specialised institutes would be particularly important in tackling the problems.

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MODIFICATION OF PAVEMENT BINDER USING WASTE POLYETHYLENE

Md. Jibon[1*], Md. M. Rahman[1] & S. K. Palit[1]

[1] *Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong - 4349, Bangladesh.*
**Corresponding Author, <jibonce@gmail.com >*

ABSTRACT

Bitumen as a binding material is commonly used in our country for roadway pavement construction. The polyethylene is biologically non-degradable and has severe adverse environmental impact being reused in the field of transportation engineering for enhancing the properties of bitumen binder. The amount of inert drawn from quarries can also be reduced by recycling waste polyethylene in pavement construction. In this investigation, waste polyethylene as sort of modifier is used to investigate the potential prospects to enhance binder properties and to check the design criteria of modified bitumen mix at optimum binder content. The investigation concentrated on the test of modified binder properties and Marshall Mix design was used to determine the optimum binder content. On the basis of experimental result, it is concluded that the bitumen binder with waste polyethylene up to 3% can be used for flexible pavement construction from standpoint of enhanced properties.

Keywords: Waste polyethylene, Bitumen mixture, Optimum bitumen content, Pavement, Marshall Test.

INTRODUCTION

Bitumen is a complex mixture of organic and inorganic compound which remains after vacuum distillation of crude oil. It is widely used in the construction of the flexible pavement. Various types of waste polymers are used to change bituminous binder to achieve wider performance range. It contributes to recirculation of plastic wastes as well as the solid waste disposal problem is relatively solved. In Bangladesh the amount of waste polyethylene is increasing day by day as the availability of this waste is enormous. They either get mixed with Municipal Solid Waste or thrown over land area. This increased waste is rapidly filling the remaining sites for future landfills, causing a blown out in the cost of waste disposal. It remains at the site for uncertain time causing the appreciable amount increase into the landfill, and adverse impact on environment. To encounter this trend, considerable effort is being taken turning it into reusable by products.

Polyethylene on heating molten at around 100 to 250℃ (Rahman et al, 2013**)**. It can be mixed with binder like bitumen to increased binding property. Many investigations have found that the strength of paving mixes can be enhanced by the use of a binder formed by modifying available bitumen with certain additives like sulphur and organic polymer. The modified polymer also improve temperature susceptibility and viscosity characteristics (Rahman et al, 2013**).**

Bitumen at high temperature or low rate if loading, it behaves as a viscous liquid. This phenomena indicates that there is a need to improve the performance of bitumen to minimize cracking at low temperature and plastic deformation at high temperatures.

A number of efforts were made in the past to develop pavement materials that helped in attaining longer fatigue and rutting lives. Different additives were used to modify the properties of paving bitumen on the specific requirements and situations. These include mineral fillers, extenders, Plastic (polymer), molasses, crumb rubber, fibres, antioxidants, anti-strips and hydro-carbons. Out of these waste polyethylene seems to have drawn the attention of researchers in this field in the past, mainly because of its multifarious beneficial effects (Nemade et al, 2013).

In Bangladesh, we commonly use bitumen for road construction. But it has short life and low stability under adverse condition of environment. As a result it has been necessary to go for constant maintenance and repairing works which results in huge amounts of money loss in our country. In order to remove this problem, we can use modifier to improve the life time and stability of the bitumen. Among the different types of modifier we can use waste polyethylene as this is available and cheap in our country. This polymer is non-biodegradable and has adverse impact on environment creating environmental hazard. By using this polymer environmental impact can also be removed.

From the above discussion the following objectives have selected for the present study

a) To evaluate the properties of modified bitumen.
b) To compare the result between the modified binder and neat bitumen.
c) To select the suitable content of modified binder.
d) To select optimum modified bitumen content from Marshall Mix properties

MATERIALS AND METHODS

Polyethylene used as modifier on bitumen mixtures in this investigation. Broken stone chips are used as coarse aggregate. Fine aggregate was taken from Karnaphuli river sand. The fines from sand and stone dust finer than 0.075 mm were used as filler material.

AGGREGATE

The broken stone chips retained on 4.75 mm sieve were regarded as coarse aggregate. A fine aggregate was taken from coarse sand which was passed through 4.75 mm sieve and retained on 0.075 mm sieve. The sand finer than 0.075 mm sieve was used as mineral filler. The specific gravity of coarse aggregate, fine aggregate and filler were 2.62, 2.78 and 2.86 respectively.

GRADATION OF AGGREGATE

To investigate the behaviour of bituminous concrete mix modified with waste polyethylene, a continuous graded aggregate of bituminous macadam is essential. In order to obtain a dense mix with a controlled void content, the aggregate blend is designed to be uniform graded in continuously graded bituminous macadam. IRC, 81 specification for aggregate gradation for 50-65 mm thick bituminous surface course is used in this investigation. The mix proportions of total aggregate with mineral filler have been shown in Table-1.

Table-1: Aggregate gradation used for Marshall Mix design with mineral filler

Sieve size	Specification %	Individual passing%	Individual retain %	Individual weight for 1100g	% of CA, FA & MF

1 in	100	100	0	0	CA=605g
¾ in	79-100	89.5	10.5	115.5	
½ in	59-79	69	20.5	225.5	
3/8 in	52-72	62	7	77	
#4	35-55	45	17	187	
#8	28-44	36	9	99	FA=440g
#16	20-34	27	9	99	
#30	15-27	21	6	66	
#50	10-20	15	6	66	
#100	5-13	9	6	66	
#200	2-8	-	4	44	
		5	-	55	MF=55g

BITUMINOUS MATERIAL

The binder of 80-100 penetration grade bitumen was collected from Jamuna oil company limited, Chittagong. Routine test as per AASHTO were performed on the bitumen sample and get the following properties shown in table-2.

Table-2: Properties of Bitumen

Penetration (0.1mm)	Ductility (cm)	Specific gravity	Solubility (%)	Softening point (°c)	Flash & Fire point (°c)
88	100+	1.03	96	48	326 & 342

POLYETHYLENE

Polyethylene is a thermoplastic polymer consisting of long hydrocarbon chains. This polyethylene is available in local markets in the form of a bag with various colors. In this investigation, low density polyethylene bags were used which were collected from local market and domestic wastes. After cleaning the waste polyethylene it was shredded to form the size of the particle 4-6 mm. The melting point for commercial low density polyethylene is typically 105-115°c (Rahman et al, 2013). The melting point of polyethylene used in this investigation were 115°c.

MIX DESIGN

Among five popular mix design method we used Marshall Method in this investigation.

Weighted quantity of aggregate (approximately 1100gm) and bitumen are heated to about 160°c. Heated aggregate and bitumen are mixed thoroughly and placed in a preheated mold (95-150°c) and compacted with compaction hammer (4.54 kg) by giving 75 blows on either side. The compaction temperature arbitrarily has been taken as 140°c. The compacted specimen should have thickness of 63.5 mm. The weight of mix is adjusted to get the correct height of the prepared specimen. Marshall Specimen was prepared by adding different percentages (4.0%-6.0%) of bitumen by weight of total mixes into hot aggregates. Then Marshall Stability, bulk density, flow, air voids, void filled with bitumen and void in mineral aggregate were determined for neat bitumen and modified bitumen at optimum content of binder.

RESULTS AND DISCUSSION

EFFECT OF WASTE POLYETHYLENE ON PROPERTIES OF BITUMEN

The molten polyethylene were homogeneously mixed with the hot bitumen by blending with 1500 rpm at 160°c up to 45 minute. But the excess percentage cause the segregation of these from hot mix. The penetration and ductility value decreases with the increase of polyethylene content which makes modified binder harder. By adding polyethylene the purity of the bitumen decreases so that the stability of modified bitumen decreases. The flash point and fire point of the modified binder decreases with the increase of polyethylene. Due to lower specific gravity of polyethylene the specific gravity of modified binder decreases with the increase of polyethylene content. On the other hand, the softening point value increases with the increase in polyethylene due to higher soften temperature of polyethylene. The properties of bitumen after modifying with polyethylene shown in Table-3.

Table-3: variation of properties of bitumen at 2 to 6% modifier content

Properties	Neat bitumen	Percentage of waste polyethylene (%)				
		2	3	4	5	6
Penetration (1/10mm)	88		84	81	72 65	56
Ductility (cm)	100+	95	86		78	62
Specific gravity	1.03	1.023	1.018	1.012	0.982	0.924
Solubility (%)	96	94.4	93.2	92.1	88.2	85
Softening point (°c)	48	62	53 67 76			55
Flash & Fire point (°c)	326 & 342	314 & 331	298 & 314	282 & 298	262 & 279	246 & 257
Striping value & loss on heating (%)	0.0 & 0.0	0.0 & 0.0	0.0 & 0.0	0.0 &0.0	0.0 &0.0	0.0 & 0.0

OPTIMUM MODIFIER CONTENT

The properties of modified bitumen at 3% modifier content (optimum binder content) was close to the ASTM standard value for the following properties shown in Table-4.

Table-4: comparison of modified binder properties (at 3% modifier) with ASTM standard

properties	Penetration (1/10) mm	Ductility (cm)	Specific gravity	Flash & Fire point (°c)	Stripping & loss on heating
ASTM standard value	80-100	80-100	1.01-1.04	min^m 200 & 220	max^m 25% & less than 1%
Obtained value	81	86	1.018	298 & 314	0.0 & 0.0

Solubility (93.2) at 3% modifier content was close to the ASTM standard value for solubility (min^m 95% soluble). In Bangladesh temperature in summer varies from 35 to 42°c. At 42°c the pavement temperature will be about 56°c which is beyond the softening point of neat bitumen (48°c) caused of temperature susceptibility. But at 3% modifier content the softening point (55°c) is close enough to pavement temperature in summer which result in the reduction of temperature susceptibility.

EFFECT OF WASTE POLYETHYLENE ON BITUMINOUS MIX

The following figure-1 shows the comparative Marshall properties between modified bitumen at optimum binder content (3% polyethylene) and neat bitumen. Maximum stability and unit weight are higher for modified bitumen than that of neat bitumen. Air void, flow value, VFA & VMA were within the Marshall specified limiting value. Addition of 3% waste polyethylene with neat bitumen gave higher stability and density without affecting the volumetric properties of bitumen. The optimum modifier content in this investigation was 4.93% by the total weight of mix.

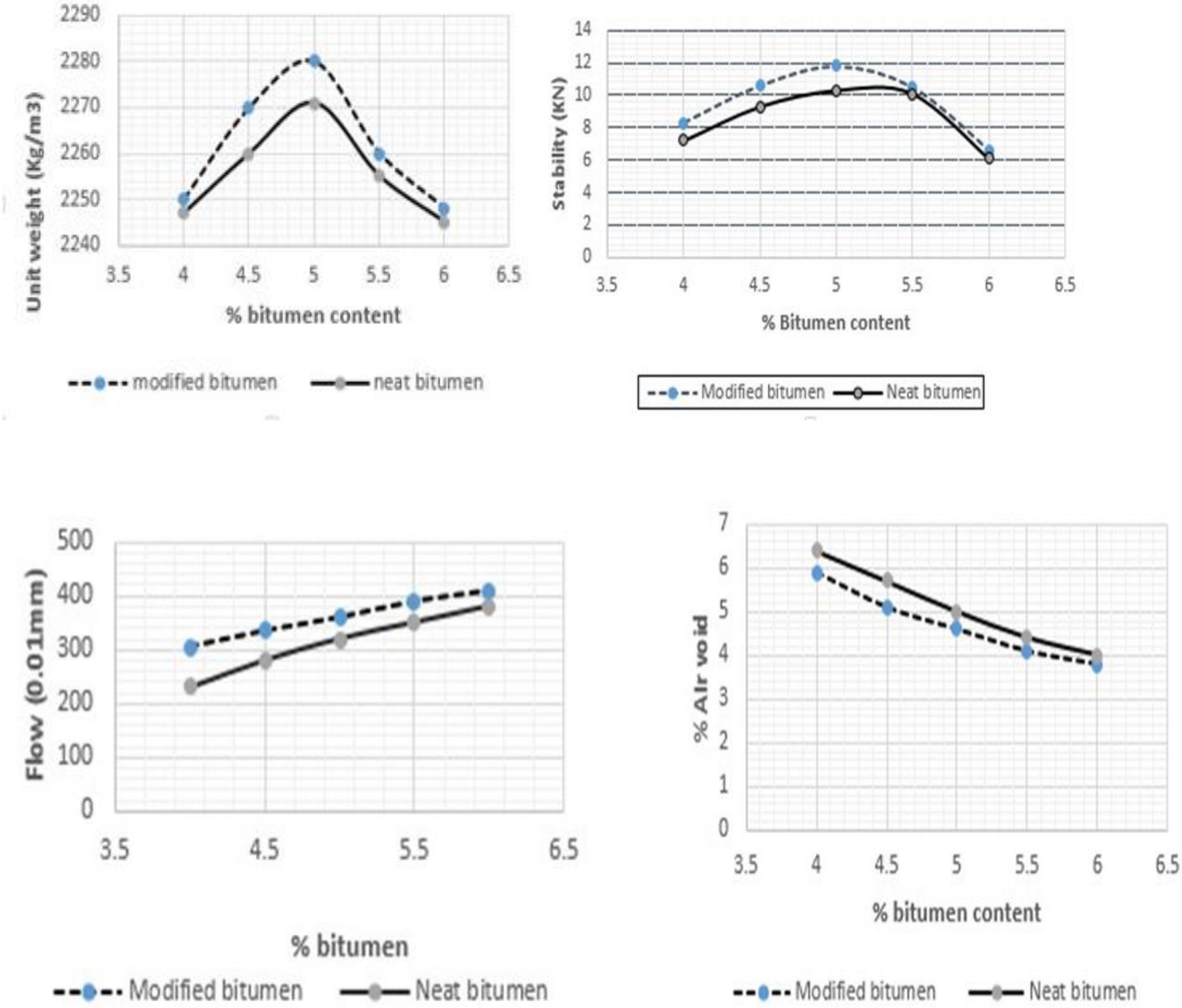

Figure-1: Marshall Mix properties

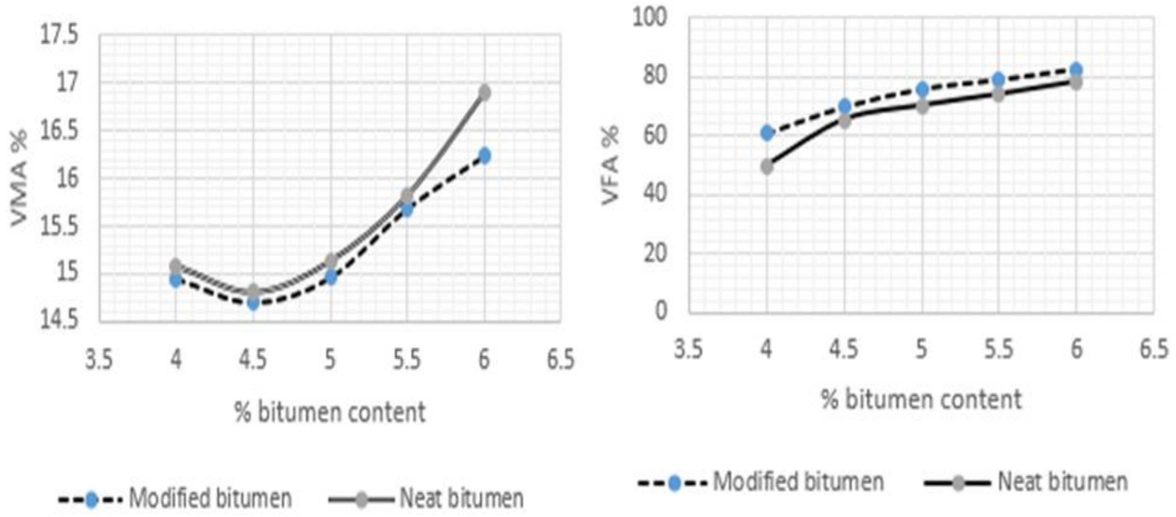

Figure-1: Marshall Mix properties

CONCLUSION

The following conclusion are drawn on the experimental result of this investigation.

- The polyethylene available from domestic and other waste can be utilize to modify the bitumen to obtain high strength mixes.

- The recommended proportion of the polyethylene modifier is 3% by the weight of bitumen content can be used for construction of road in hot climate where low penetration grade bitumen is used.

- Moreover using waste polyethylene with neat bitumen leads to the reduction of environmental hazard and cost savings.

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A STUDY ON IDENTIFYING THE POSSIBILITY OF BICYCLE AS A POTENTIAL TRANSPORTATION MODE IN DHAKA

D. Dey[1*], S. Akter[1] & M. A. B. Siddique[2]

[1] *Department of Civil Engineering, University of Asia Pacific, Dhaka, Bangladesh.*
[2] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <ndabaraj@yahoo.com>*

ABSTRACT

Acute traffic congestion is one of the major problems of Dhaka. With a view to get rid of this problem, miscellaneous measures like constructing new roads and infrastructures, regulations on the roads etc. has been implemented which actually exhort motorized vehicles and circumstances remain almost same. At the present situation Dhaka need a sustainable solution and bicycle could be the effective alternate mode of transportation like Amsterdam, Copenhagen, China, and Vietnam etc.

Recently, the trend of cycling in Dhaka is rising. By the last three years, bicycle selling have grown up to 40%. Becoming encouraged from the interest of the people, a survey has been conducted among the current bicycle users of Dhaka. The survey result has revealed some problems experienced by them as well as proposed some solutions to reduce. Then, for bicycle non users, a stated preference (SP) online survey has been conducted representing the hypothetical scenario of bicycle lane in Dhaka to estimate the utility of the bicycle.

A Multinomial Logit model (MNL) has been developed taking into consideration the socio-economic characteristics of respondents, where more than 80% responded voted on behalf of lane facility. As, Mass Rapid Transit (MRT) and Bus Rapid Transit (BRT) are going to be activated in Dhaka, providing bicycle lane can be very much effective solution as home to station connecting mode to reduce traffic congestion.

Keywords: Bicycle, Bicycle lane, MNL, SP.

INTRODUCTION

Dhaka is the centre of political, cultural and economic life of Bangladesh. Although its overall infrastructure is the most developed of the country, Dhaka suffers from major urban problem traffic congestion due to the rising population. According to BRTA the numbers of motor vehicles registered in 2003 were 303215 which became 562851 in June 2010 and risen 817425 in June 2014 (BRTA, 2014). As the rates of development of infrastructures that are required to support these growths have not been achieved, traffic congestion is becoming an over lasting and uncontrollable problem with time.

To get rid of traffic congestion problem, people are finding an alternate mode of transportation along with development of infrastructure. The annual demand of bicycle which is about 500000 pieces, 40% more than last three years (The Daily ProthomAlo, 2014) reflects the interest of adopting bicycle as inexpensive to build, buy, ride and maintain. World's top developed countries are very familiar with bicycle trips. In Netherlands almost 30% of their all trips type have made with Bicycle so on Denmark having almost 20% trips. Without that world's biggest cities have taken several steps to improve bicycle riding quality and facility. City like Copenhagen is invested $200 million to add another 136 KM bicycle road network to its existing network. Amsterdam capital of Netherlands has also allocated $160 million to improve their cycling facilities (EPI, 2002). It means bicycle has a great potential to be a sustainable alternate transportation mode and an appropriate solution to the present traffic congestion problem.

Objectives

The overall objectives of this research are to explore the demand, expectation and suggestion from the current bicycle users and non users by conducting face to face survey and online base survey. Beside that the other objective is to develop a mode choice model with the help of SP survey conducted from the bicycle non-user using advanced discrete choice modeling techniques that can capture the potential of implementing bicycle lane in Dhaka.

METHODOLOGY

To estimate the model two surveys have been conducted. The initial survey is done by face to face survey and discussion among the bicycle companies, stores, dealers and bicycle groups which indicates an overall scenario of bicycle condition in Dhaka city. Being inspired from that the biggest task of data collection has been done among the bicycle users and non users. Vast and new technologies have been implemented to find fluent and efficient data from the sources. Both the bicycle users and non users have been asked several questions regarding bicycle present condition. An online survey using Google Docs system has been done to get the response of bicycle user. Meanwhile bicycle non user survey has been done by face to face questionnaire survey and by using online facility like Google Docs to gather opinion of the people of different part of Dhaka.

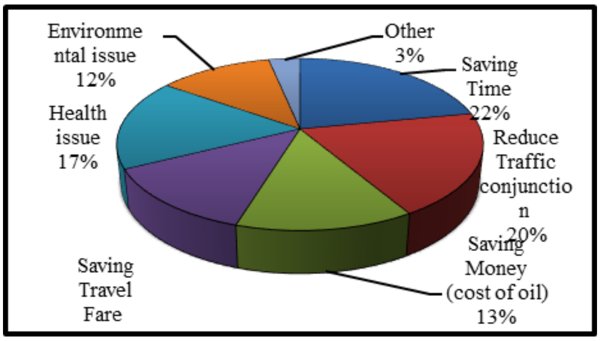
DATA ANALYSIS

Data have been analysis taking 368 bicycle user responses and 320 bicycle non user’s responses. Some fundamental questions have been arranged in all survey questionnaires to know the socio economic conditions of the respondents. These included the questions about the age, gender, profession and education.

Bicycle User Survey Analysis

The SP survey for bicycle user has been conducted among 368 respondents. The respondents were both male and female of various ages and various socio-economic conditions of which 70% respondents were students, 23% were jobholders. All the respondents had the opportunity to use other vehicles for their trip. But they use the bicycle for their trip for various reasons. Among the total respondents 22% use bicycle for saving time, 20% use it to improve the present condition of traffic congestion, 17% thinks riding cycle is good for health, 12% says bicycle environmental friendly mode of transport [Figure1].

People always select their transport according to situation, in what condition what type of transport one should need it varies person to person. The responses of bicycle user gives an idea that 58% of all respondent use bicycle as a commuter trip, this type of trips means university, colleges or work place mainly the place where they have to go regularly by maintaining a fixed time, 17% of them use it only for as a pleasure trip, 13% use it as for their daily necessary like at the time for buy buying something or for meet with someone mainly when it is needed for any work [Figure2].



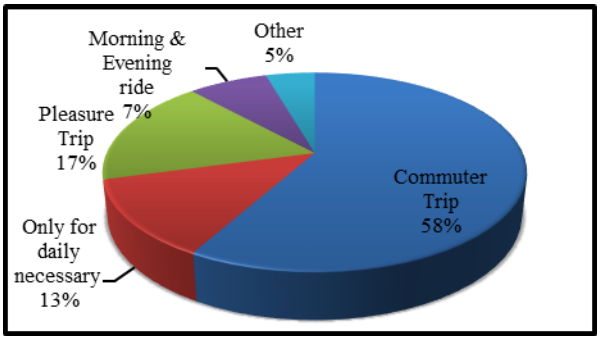


Figure 1: Reason of Choosing Bicycle

Figure 2: Types of Bicycle Trips

Among all respondents 95% cycle users think Cycle lane should be introduce in DHAKA. Here the scenario clear that almost every user want bicycle lane in Dhaka city. In theses regard to get maximum utility from bicycle lane they have mentioned about some problems of which they face every day while cycling. Survey results showed that 30% respondents says safety on road is a big problem, 29% respondents says that security of cycle, 26% says parking problem. The positive impact about cycling is clearly visible as only 6% bicycle users face social acceptance problem [Figure4]. If it is possible to solve the problem and provide facility like segregated lane, parking facility user may be arise for cycling.

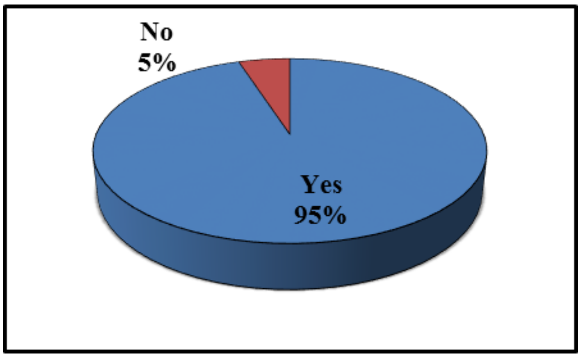


Figure 3: Bicycle Lane adaptations Possibility

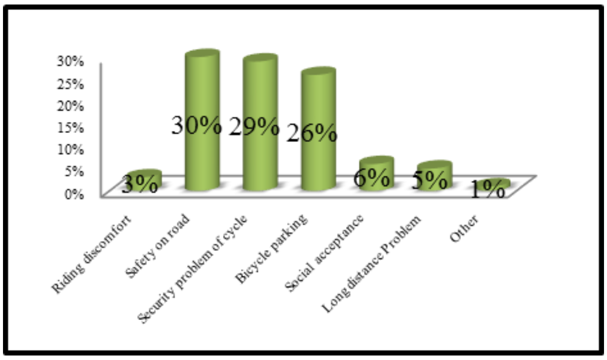


Figure 4: Problems Facing by Bicycle User

Bicycle Non User Survey Analysis

The bicycle non users' surveys have been done by face to face questionnaire survey and by using online facility like Google Docs. Meanwhile online surveys using Google Docs system have been done to get the proper response of bicycle non user around the different part of Dhaka city. Among all the respondents 70% students, 24% jobholders and 4% were businessmen.

The SP survey has founded several reasons why people are choosing other transport mode instead of selecting bicycle for their commuter trip and other trips. Among the entire respondents majority 35% think safety on road is the main reason along with security problem (19%), bicycle parking (15%), long distance covering problem (16%) [Figure5].

Some positive impact have been found, only 7% of total 320 respondents think about social acceptance problem and 86% people think it is the right time to introduce bicycle lane in Dhaka. It seems if bicycle lane is provided people will use bicycle as an alternative mode of transport as regular basis.

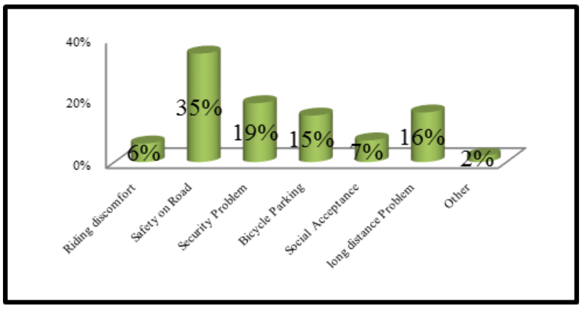


Figure 5: Problem of Bicycle as Non User

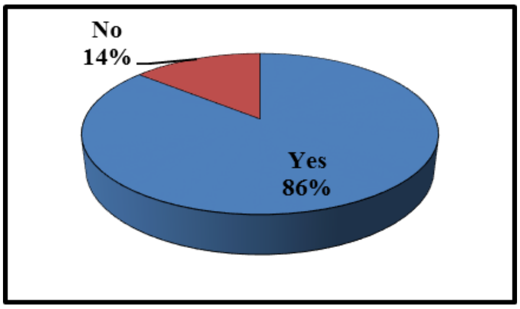


Figure 6: Choice of Lane

MODEL ESTIMATION RESULTS

A choice set probability model (Multinomial Nested Logit Model) is developed using the non user opinion.To develop the utility function of MNL some socio-economic attributes [Table 1] are considered. Respondent's replies along with socio-economic characteristics have been used to estimate the model using Multinomial Logit Model technique by BIOGEME Version 1.8.

Table 1: Candidate variables and usual choice relationship

Attributes	General Casual Relationship
Price of Bicycle	The price of the bicycle can affects the choice of system among the respondents, considering our social and economic position. In a general if bicycle price is high the people usually do not want to get a bicycle.
Age	Greater the age, less the possibility to accept the bicycle as an alternative transportation system regarding the comfort issue and the stamina. Without that social acceptance and hindrance can be a major consideration in that regard as our social system is not allow one to do as like as you want.
Gender	Due to social norms and culture male and female bicycle riders do not feel free to use cycle especially without lane situation and also female passengers do not want to use cycle due to safety and privacy concerns.
Travel time	Increase in current travel duration may increase the choice in the favour of switching to bicycle.
Occupation	Highly educated people and Enterprise businessmen are likely to have higher propensities of using private cars instead of bicycle. On the other hand the choice and utility should be positive for the students regarding their mentality and way of thinking.

Utility Function for Estimation

A discrete choice modelling technique is used to estimate the utility parameters from different choice sets. A linear utility function is associated with each choice set. The utility of choice set *i* of individual *n* can be expressed as in Equation 1.

$$U_{in} = \beta_i X_{in} + \varepsilon_{in} \forall_i \epsilon C_n \qquad \text{(Equation 1)}$$

Where

X_{in} = socio-economic characteristics of the individuals and attributes of different modes

β_i= Coefficient of X_{in}

ε_{in} = Random error term

C_n = Universal choice set or the choice set determined deterministically for individual n

Base Estimation Model

Choice set is affected by the attributes mentioned upper which exhibit the social, economic, culture, norms and other major issue of individual respondents. But the values of parameters may not be significant or the signs may not be intuitive for all the variables considered. The estimation is started with considering three basic parameters which are age, gender and price of bicycle. The rest considerable parameters are considered step by step.

The utility functions for bicycle lane considering the age, gender and price of bicycle as the direct considerable effect to bicycle riding are presented below & model result is presented in Table 1.

U_{lane} = ASC_cycle * one + βage * age + βgender * gender + βprice * price

U_{no_lane} = ASC_no_cycle * one

Where, utility function of no lane (U_{no_lane}) has been considered fixed and its value is zero for the model developed here.

Here,

ASC = Alternative Specific Constant, ASC_cycle = Alternative Specific Constant of choosing cycle lane, βage = Coefficient of Age, βgender = Coefficient of Gender, βprice = Coefficient of Price

Table 2: Bicycle Lane Choice Model (Age, Gender, Price)

Parameter	Estimated Value	t-test	Rho-square
Constants			0.397
ASC_cycle	5.02	3.50	
Variables			
Coefficient of Age, β_{age}	-0.0775	-1.55	
Coefficient of Gender, β_{gender}	-0.754	-2.20	
Coefficient of Price, β_{price}	-0.193	-1.39	

The results indicates that the probability of adopting the utility of Bicycle lane. The higher value of Alternative Specific Constant of choosing cycle lane (ASC_cycle) tells the higher likelihood among the bicycle nonusers. The value of two Variables Coefficient of Age & Coefficient of Price is statically significant at more than 80% confidence level. The Coefficient of gender has statically significant at or more than 90% confidence level. The sign of the Coefficient of Age & Coefficient of Price indicates that the utility of Bicycle lane will decrease with the increase with the age and increase with the price. This shows an intuitive sign. The sing of the Coefficient of gender also indicates females are less interested do adopted the utility of bicycle lane in a compare with the males.

Final Model

Various variables have been included in the base model function to improve that statistically and logically. Running several model and by doing a trial and error method a final model has been selected.

U_{lane} = ASC_cycle * one + βgender * gender + βprice * price + βctt * tt_dummy + βoccupation * occupation_dummy

U_{no_lane} = ASC_no_cycle * one

Where,

Occupation_dummy = occupation = 1 (1 is the respondent is student)

Current Travel Time_dummy = Travel time >= 20 minute

Table 3: Final Bicycle Lane Choice Model (Gender, Price, Current Travel Time, Occupation)

Parameter	Estimated Value	t-test	Rho-square
Constants			0.402
ASC_cycle	2.19	3.14	
Variables			
Coefficient of Gender, β_{gender}	-0.770	-2.23	
Coefficient of Price, β_{price}	-0.221	-1.58	
Coefficient current travel time, β_{ctt}	0.570	1.35	
Coefficient of Occupation, $\beta_{occupation}$	0.851	2.34	

To take an effective effect of Current Travel Time as well as Occupation dummy function has been added to the same utility function the meanwhile variable Coefficient of Age has neglected due to a much lower estimated value.

By adding the dummy function of current travel time and occupation a final estimated result has been found shown in Table 3.The person who has a current travel time less than 20 minute utility has been considered 0. A person having a travel time less than 20 minute in current condition does not bother about bicycle utility as their current travel distance is shorter. On the other hand as the majority survey data have been found by student's opinion, so the utility function of businessmen and job holders have not been considered. Businessmen and job holders are not likely to adopt bicycle at the present road condition.

All the values of constants and variables are statically significant at 95% confidence level or more only expect the values of Coefficient of Price and Coefficient current travel time which are at significantly 80% confidence level or more. The estimated value of β_{gender} (-0.770) indicates that the utility of cycle is more adoptable to male compare to female. Another most important consideration which makes an operative effect is price of bicycle. The estimated value of β_{price} (-0.221) evidence of that is the higher the price lower the likelihood of switch over bicycle. The estimated value of β_{ctt} (0.570) point out that travel time is one of the major operating factors. By the use of the dummy function of current travel time more considerable value has been found. Coefficient of Occupation, $\beta_{occupation}$ (0.851) is much higher to the students.

Explanation

Generally shorter commuter trips between 5KM~10KM are made by bicycle. Bicycles is generally considered a better option for students and youth as they are more interested about cycle and have much stamina. Beside that most of the students have scarcity of money as do not afford privet transport like cars or other luxurious transportation mode. Bicycle riding is less preferable to females of our country and the higher price of bicycle is not admissible due to the socio-economic circumstance of Bangladesh. Moreover, the people who use car are less aware of the public transit systems which influence the choice option of the respondents.

CONCLUSION

In this research, to investigate bicycle as a potential transportation mode in Dhaka a bicycle utility model has been developed using SP data with simple Multinomial Logit Model. Bicycle user survey design show almost 95% bicycle user think that it is the high time to introduce a bicycle lane in Dhaka instead of having some problems like safety on road, parking problem, etc. On the other hand non user survey design founded mainly students are the bicycle user in present no lane condition. More than 85% non user point out that introducing a bicycle lane will increase the use of cycle.

The model has defined the utilities of bicycle lane considering the socio-economic attributes and the cycle riding related factors which can affect the choice of bicycle trip of the respondents. The model result indicates the feasibility of bicycle and potentiality to implement bicycle lane in Dhaka to ensure a sustainable and alternate transportation mode. These bicycles lanes can be very much effective for short trips. Thus this model opens a new window to the planers for planning a sustainable and congestion free Dhaka city and to ensure the maximum utility of proposed MRT by Bangladesh Government.

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SYMPOSIUM ON FORGIVING ROADS AND ROADSIDES: AN INNOVATIVE CHALLENGE

Rabbani Rash-ha Wahi[1*]

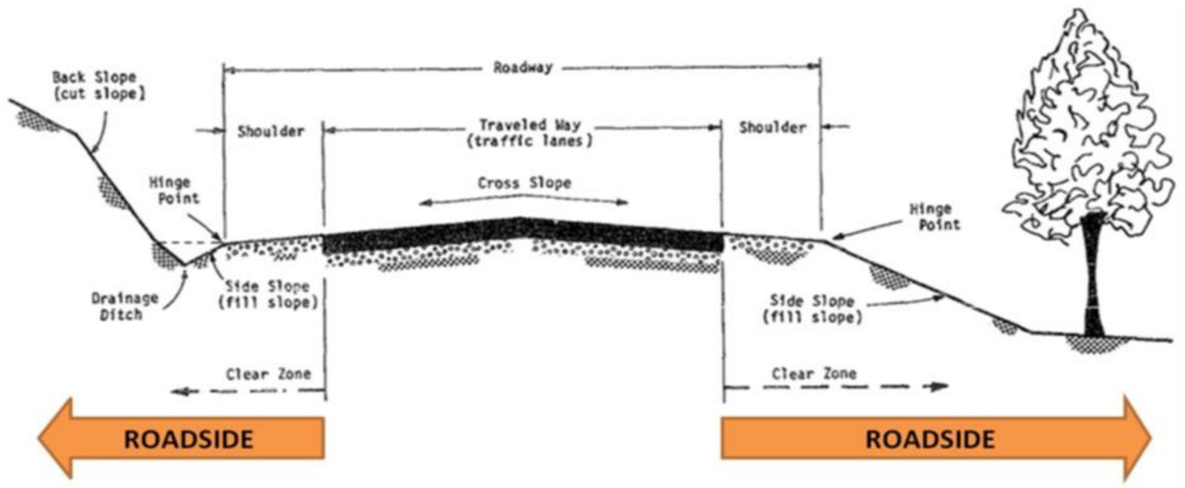
[1] *Department of Civil and Environmental Engineering, Uttara University. Dhaka - 1230, Bangladesh.*
**Corresponding Author, < rabbanirash.ha@gmail.com>*

To increase road safety, the concept of forgiving roadsides is one of the main priorities in the area of road infrastructure measures. A Forgiving Road designs in the Developing countries like Bangladesh is the best practices to reduce the severity of an impact once the driver has made a mistake. Ideally, this means recovering from the error and being able to continue on the journey without striking anything. There is widespread disregard of explicit safety considerations in the road planning, design and rehabilitation program. In Highway crashes occur when something goes wrong. It could be a mechanical failure, roadway deficiency, driver error, medical emergency, or a combination of these factors. Addressing the roadside safety problem requires that all elements of the roadside-vehicle-driver system be considered. Each element must work together in harmony if the system is to provide mobility at an acceptable level of safety and at a reasonable cost. Indeed, road safety issue in Bangladesh thus posits a considerable challenge to the road engineering professionals. This paper presents an overview of road safety in Bangladesh and introducing new approaches of road safety audit, inspection and Strategic evaluation of a number of innovative systems contributing to the creation of a more forgiving road environment.

Keywords: Forgiving Road, Road safety audit, Inspection and risk assessment

INTRODUCTION

The concept of a sustainable, safe traffic environment has been developed to increase safe driving. This involves issues of road design and issues on driving style. On the one hand, there are opportunities to increase safety by developing a forgiving road (FOR) environment. A forgiving road (FOR) is defined as a road that is designed and built in such a way as to counteract or prevent driving errors and to avoid or mitigate the negative consequences of such errors (Wegman and Aarts, 2005). Forgiving road environments can be considered a basic tool to prevent or mitigate an important percentage of road accidents related to driving errors. The existence of a forgiving road environment would prevent accidents of this type (and generally accidents that involve driving errors) or, at least, reduce the seriousness of the consequences of an accident. The roadside is defined as the area beyond the carriageway and mostly includes elements such as slopes, ditches and various obstacles (e.g. trees, utility poles or masonry structures). Strategies for forgiving roadsides have already been stated by the Federal Highway Administration (1986), AASHTO (2002) or in numerous national guidelines. A high number of studies and research works were carried out to determine the impact of various roadside features on frequency and severity of accidents (cf. Holdridge et al. 2005; Lee & Mannering 2002; Ray 1999; Stamatiadis & Pigman 2009). A study by Nitsche et al. (2010) summarizes state-of-the-art treatments to make roadsides forgiving, as well as harmonizes currently applied standards and guidelines. Accordingly, the following three groups of treatments can be applied: 1) Remove or relocate obstacles on the roadside to provide clear zones.2) Modify obstacles to make them break-away or crashworthy.3) Shield obstacles by installing road restraint systems (RRS).The forgiving road should be the vision for the safe road infrastructure in the future. The aim is simplicity and clearness instead of complexity and ambiguity. (Herrstedt 2001). Despite differences in strategies across States and countries, the ‘Safe System’ approach aims to build a road transport system which tolerates human error and prevents death or serious injury in the event of a crash (The Australasian College of Road Safety 2010). A safe infrastructure depends on a road-user-adapted design of different road elements such as markings, signs, geometry, equipment, lighting, road surface, management of traffic and speed, traffic laws etc. (Herrstedt 2006). Fig. 1 shows a typical roadway cross section (cut and embankment section) including some roadside elements. As part of a research initiative to bridge this gap of the knowledge base in different stakeholders conducted survey to collect data on different road classifications. Then using this analysis for the overall objective of this research which is to update the current forgiving guidelines to accommodate the stakeholders in a quantitative manner.



[Fig. 1] Roadway Cross Section with Examples for Roadsides with Clear Zones (F.D.A,USA)

MATERIALS AND METHODS

A multi-actor multi-criteria analysis (MAMCA) is a multi-criteria analysis (MCA) whereby the decision tree is structured on the basis of stakeholder criteria. Three stakeholders were identified in this application, namely (1) users, (2) society/public policy makers and (3) manufacturers. Each of these stakeholders attaches importance to a specific set of criteria. The overall relative priorities reflecting the society's point of view were taken as the starting base for policy purposes. The overall relative priorities of the two other stakeholders (users and manufacturers) make it possible to assess the implementation and market potential of the scenarios.

The research methodology, involved the following steps:

1. Preparation for the experiment included designing and planning of the experimental study, recruiting volunteers for the experiment and training them on the procedure, and selection of the test route. The alignment was composed of urban freeways and two-lane highways.

2. The second component involved the actual experimenting phase. It was concerned with collecting reliable data about stakeholders along the test route's alignment.

3. The third component was the preparation of the data base used for pairwise comparison Metrix method. This database included the synchronized driver behaviour data collected during the experiment data of the roads traversed.

4. The fourth component involved examining, analysing, and modelling the interaction between the users, society and manufactures derived from these experiments.

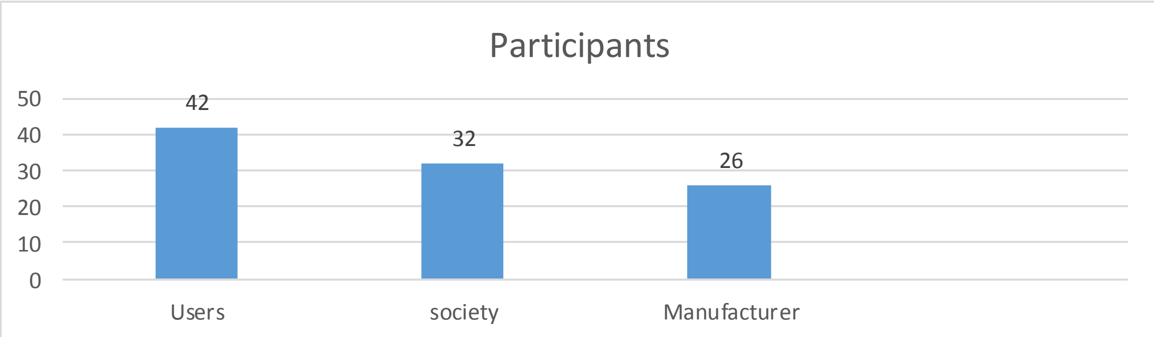
5. The final outcome of this research was concerned with revising and updating the current forgiving design guidelines in order to accommodate stakeholders in a more accurate and quantitative manner.

In literature, delineation is often mentioned as treatment if all of the three measures above are unfeasible. Delineating can help a driver to avoid hitting roadside hazards. However, this measure is not included as a separate chapter, because it belongs to the strategies for self-explaining and not for forgiving roads.

RESULT AND DISCUSSIONS

After having identified the criteria and the alternatives, the next step is then to perform a partial evaluation, i.e. an evaluation in terms of each specific criterion. Therefore, for each alternative a score should be derived expressing the contribution of that alternative to that specific criterion. The first step in the AHP methodology is to derive weights for the criteria. Three main stakeholders were identified in the decision tree -namely users (drivers), society/authorities and manufacturers, which each have their own specific criteria.

In order to obtain the inputs necessary for these pairwise comparison matrices, a forum of policy makers and representatives of the users and manufacturers was created. For the workshop to elicit weights, the room was rearranged to facilitate a Group Decision Room (GDR) session. A total of 100 participants actively participated in the GDR session. Table 1 shows the number of participants for each stakeholder group. The complete list of participants is given in Fig. 2.



[Fig. 2] participants of different stakeholders.

All these stakeholder representatives had to compare the importance of the criteria in pairs, using the pairwise comparison scale. In order to synthesize the various pairwise comparisons given by each representative, the geometric mean was calculated. The geometric mean (and not the arithmetic mean) is the statistical measure that is relevant in this case (Saaty, 1995:265), since the average of ratios is to be calculated here.The final results of all these pairwise comparisons made by each representative of the various stakeholders and synthesised using the GDR software are shown in Table 1, Table 2 and Table 3. Part A of these tables shows the synthesis (i.e. the geometric mean) of the various pairwise comparisons and Part B contains the final relative priorities for the criteria (i.e. the criterion weights) calculated on the basis of these pairwise comparisons.

Table 1: Pairwise comparison matrix and relative priorities for the criteria from the point of view of the stakeholder 'users'

Part A Pair wise comparison				Part B
Stakeholders Users	Driver comfort	Driver safety	Travel time duration	Relative priority
Driver comfort	1	1/2	3/1	.3194
Driver safety		1	4/1	.5595
Travel time duration			1	.1211

Table 2: Pairwise comparison matrix and relative priorities for the criteria from the point of view of the ‘society/public policy makers’

Part A Pair wise comparison				Part B
stakeholder society	overall safety	public Expenditure.	environmental effects	Relative priority
overall safety	1	3	4	.601
public Expenditure.		1	1/3	.165
environmental effects			1	.24

Table 3: Pairwise comparison matrix and relative priorities for the criteria from the point of view of the ‘manufacturers

Part A Pair wise comparison				Part B
stakeholder society	investment risk	liability risk	Technical feasibility	Relative priority
investment risk	1	1/2	3	.35
liability risk		1	2	.49
Technical feasibility			1	.16

Table 1, Table 2, and Table 3 represent the relative priorities of the criteria, i.e. the priorities in terms of the overall objective of one specific stakeholder, resp. ‘users’, ‘society/authorities’ and ‘manufacturers’. the users gave the highest weight to the criterion ‘driver safety’ (55.2%).they gave less weight to ‘travel time duration’ (12.11%) and still more than to ‘driver comfort’ (31.94%).manufacturers gave the highest weight to the criterion ‘liability risk’ (49%), then ‘investment risk’ (35%) and technical feasibility received a much lower priority (16%). from the societal point of view, the criterion ‘overall safety’ turned out to be the most important criterion (60.1%). the criteria ‘environmental effects’ received a lower weight 24.0%. the criteria and ‘public expenditure’ received the lowest weight 16.5%. the overall evaluation phase consists of deriving overall relative priorities for the alternatives in terms of each stakeholder’s point of view.

These final relative priorities indicate the degree to which the alternatives contribute to the overall objective or focus of that specific stakeholder.it should be noted, however, that the actor ‘society/authorities’ is in fact not a stakeholder sense, since that actor represents the societal point of view. the two other actors or stakeholders, namely the ‘users’ and the ‘manufacturers’ are indeed stakeholders sense, since they reflect the objectives of only one specific group of people in society.

CONCLUSION

A preliminary selection and prioritisation of alternative ways for the design of innovative alternatives contributing to a forgiving road (FOR) environment was performed in this paper. Forgiving road, which means gives the opportunity to get back on mistakes to avoid collapse There are substantial differences in the rankings of alternatives depending on the point of view of the stakeholder groups ‘society’, ‘users’, and ‘manufacturers’. In terms of forgiving road environments, the identification of error patterns that lead to accidents is the first step, in order to conclude to measures to be taken for rendering a road environment of forgiving nature. What is of outmost importance is to select the appropriate measure for each type of error, either in terms of infrastructure enhancement or application of telematics, or even their combinations, which are seen as the most promising solution, especially in terms of cost efficiency. In Bangladesh need to more experience with this new innovative method and new roads should be reflecting the nature of the new criteria.it is the best long-term program which will eventually yield enormous safety benefits through the cost benefit treatment of a limited number of road way each year. Today most countries with sophisticated highway systems and a desire for safer roads are incorporating these passive restraint devices into their highway design. Their goals continue to be a cost effective means to create "Forgiving Highways." These products are tools that can be, and must be used to reach their goals.

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DETERMINATION OF TRIP ATTRACTION RATES OF SHOPPING CENTERS IN DHAKA CITY

M. S. Mamun[1*], S. M. R. Rahman[1], M. M. Rahman[1], Y. B. Aziz[1] & M. A. Raihan[1]

[1] *Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <smamun93@hotmail.com>*

ABSTRACT

Establishment of any new development generates additional trips that may have negative effects on the existing road network. To assess the impact of the development traffic on the transport network and to identify reasonable solutions Traffic Impact Assessment (TIA) is performed. In the absence of sophisticated travel demand model TIA is carried out manually. In manual process, total trip generated from the development site is estimated by multiplying the trip rate with the development size. In this process the task can be completed very easily, but the accuracy of the calculation depends on the reliability of the trip rates used in the calculation. As Bangladesh does not have standard trip rates, TIA is generally performed by using trip rates obtained from the Institute of Transportation Engineers' "Trip Generation" report, which is ideal for Western countries. In this research, trip rates of shopping centers in Dhaka city are estimated.

This study conducts only for trip attraction rates of shopping centers having different sizes and located at different places (Dhanmondi, Gulshan and Siddheswari) in Dhaka city. In order to do so number of persons and vehicles entering the shopping centers in every fifteen minutes interval during peak periods are counted then converted for an hour. From the survey data, it is found that the trip attraction rates of small shopping centers are much higher than that of medium size shopping centers. Macroscopic model is also developed from the survey data. The macroscopic model relates the attraction rates of the shopping centers as a function of the physical features such as gross floor area, number of car parking, number of shops and availability of restaurants (available or not) in the shopping centers.

Keywords: Trip Attraction Rate, Shopping Center, Macroscopic Model, Dhaka City

INTRODUCTION

World population is increasing rapidly, so new developments have become a major issue nowadays. These developments are adding and graduating new travel demand. Due to unplanned existing road network, this traffic is making the performance (i.e. lower free flow speed, increased delay) of the road network worse. Therefore, nowadays traffic impact assessment (TIA) is badly needed for the approval of any kind of new development. The trip generation is an important step to the traffic engineers and planners in assessing the impact of new development such as activity centers and residential development. The main purpose of trip generation is to correlate land use and trip making activity to identify future travel pattern (Paquette et al., 1981).

The procedure of estimating trip generation is categorized as the procedure of estimating trip production and trip attraction (TA) at each Traffic Analysis Zone (TAZ). The number of trips attracted by different activity centers in any TAZ is defined as trip attraction. Trip attraction is obviously the most appropriate relative to traffic at specific land use activity. It also plays a role in many phases of transportation planning and traffic engineering related activities. Shopping trips are the second most contributing aspect of TA. Therefore, knowing trip attraction rates of shopping centers is very important.

Developed countries have their own trips rates or models to estimate trip generation. Moreover, in absence of their own trip rates, western countries use trip rates from ITE's Trip Generation Manual (ITE, 2008). But, due to dissimilarities in land-use pattern, socio-economic characteristics and road network facilities, the developed trip rates and models are not applicable in our country. Therefore, in this research trip attraction rates of shopping centers are determined in the context of Bangladesh. Hourly rates are determined with respect to gross floor area, number of employees, total number of parking spaces and number of stores, as from previous research (Fillone et al., 2003; Badoe and Miller, 2000; Innes et al., 1990) it is found that these physical features have significant influence on trip attractions.

This study also develop macroscopic model to estimate the trip attraction rate of shopping centers. The model will be utilized for planning and design of shopping centers for the geometric design as well as for traffic control schemes on the roadways around the shopping centers. This model can be the alternate option to the ITE Trip Generation Manual (Kikuchi et al., 2004). In this study the information about gross floor area, total number of parking space, number of store of shopping centers etc. are collected from six shopping centers having different sizes and located at different places (Dhanmondi, Gulshan and Siddheswari) in Dhaka city.

SURVEY AND DATA COLLECTION

The survey was conducted for six shopping centers (Gulshan Pink City, Plaza A. R., Fortune Shopping Mall, A. R. A. Center, Aarong and Navana Baily Star) in Gulshan, Dhanmondi and Shiddeshwari area in Dhaka city. The locations of the studied shopping centers are shown in Figure 1. Among them first three are medium size (Type 2) and last three are small size (Type 3) shopping centers. The definitions of different types of shopping centers are provided below.

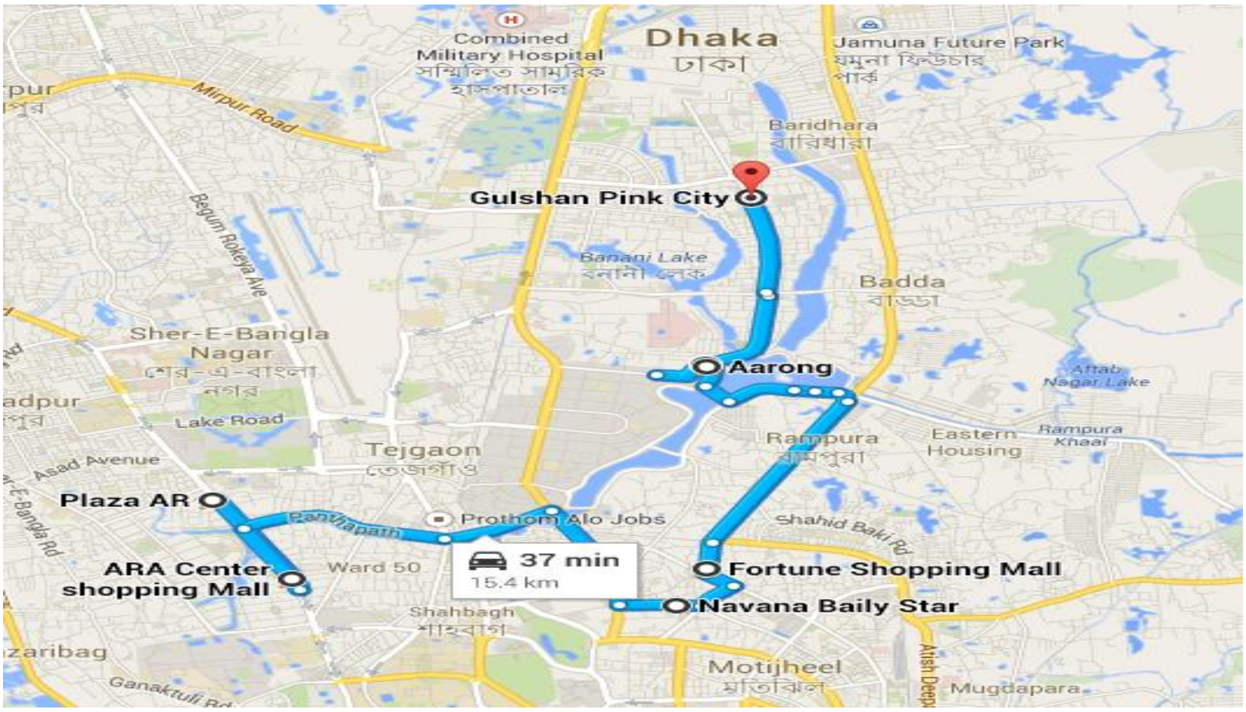

Fig. 1: Surveyed shopping centers in Gulshan, Dhanmondi and Shiddeswari Road in Dhaka city (Source: Google map)

Type 1: This is a large shopping centers containing large retail Spaces, mega Stores, a large super shop, theme park, cinemas, food court, a corporate office, a health club and swimming pool, a community center and many small stores. Bashundhara City and Jamuna Future Park etc are this type of shopping centers.

Type 2: This category is for medium shopping centers containing medium sized super shop, a medium restaurant, a medium office, a medium sized discount retail shop and numerous small stores. Gushan Pink City, Plaza A. R. and Fortune Shopping Mall are this type of shopping centers.

Type 3: This is a small shopping center containing one super store and many small stores. A small restaurant may be available or not in this type of shopping center. The shopping centers in this type are A. R. A. Center, Aarong and Navana Baily Star.

Type 4: This is a collection of specialty stores, but does not include a supermarket or discount retail store. The shopping centers in this type are Multiplan Center, BCS Computer city and Hatil.

The project group surveyed six shopping centers at peak hours, i.e. 5.00 p.m. to 8.00 p.m. in week days. People are attracted to the shopping centers because of various purposes like shopping, fitness centers, having food in restaurant and for other services. Surveyors counted people and vehicles entering the shopping center in every 15 minutes interval by standing near the entrance of the shopping center to determine the trip attraction rate. The 15-minute interval is selected because Highway Capacity Manual uses this interval as the base unit for capacity computation (Kikuchi et al., 2004). Shopping trips were estimated by visual observation. Survey was done on two typical week days. At first day, surveyors just counted the shopping trips for three hours (6 consecutive 15-min interval) and second day was for collecting information about different physical features (i.e. number of shops, gross floor area, parking space, number of employee). The physical features and average 15-minute trip (person) attraction rates of six shopping malls are provided in Table 1. These collected data was processed for building regression model in SPPSS software to determine trip attraction rate with respect to different physical features for the development of macroscopic model.

Table 1: Physical features and average 15-min trip attraction rates of the surveyed shopping centers

Type	Name of the Shopping Center	Floor Area/1000 (sq^2 ft)	No. of stores	Parking Space	Restaurant (yes=1,no=0)	Parking Space (available=1, not available=0)	Average trip attraction rate per 15 min interval
Type 2	Gulshan Pink City	165.888	160	63	1	1	120.25
	Plaza A. R.	75.000	72	40	0	1	65.00
	Fortune Shopping Mall	99.000	387	130	1	1	142.33
Type 3	Aarong	12.500	1	30	1	1	93.50
	A.R.A. Center	24.000	33	12	1	0	37.33
	Navan Baily Star	50.079	128	20	1	1	158.58

DATA ANALYSIS AND MODEL BUILDING

Trip attraction rate was determined with respect to different physical features, as for example person trips/1000 ft^2/hour, person trips/100 employees/hour, person trips/shop/hour and person trips/10 parking spaces/hour. The trips rates of different shopping centers are provided in Table 2. The average value and the standard deviation of each type of shopping centers are also given in the table. From the table it can be observed that the average trip attraction rates of small size shopping centers (Type 3) are much higher than the medium size shopping centers (Type 2). Moreover, from the data analysis, it is found that the trip rates per 1000 sq. ft. gross area is easier to determine and results are consistent and reliable. Trips rates per shop are most inconsistent measure due to the fact that the shopping center like Aarong is a big place consisting of only one large store.

Table 2: Trip attraction rates of shopping centers based on different physical features

Type	Name of the Shopping Center	Person trips/1000 ft^2/hour	person trips/100 employees/hour	person trips/shop/hour	person trips/10 parking spaces/hour
Type 2	Gulshan Pink City	3.83	113.77	3.975	7.46
	Plaza A. R.	4.53	161.13	4.72	1.50
	Fortune Shopping Mall	6.38	151.55	1.63	0.54
	Average	4.913	142.15	3.442	3.17
	Standard Deviation	1.317	25.04	1.61	3.48
Type 3	Aarong	18.38	202.81	432	30.00
	A.R.A. Center	12.66	253.33	6.9	10.00
	Navan Baily Star	15.81	293.33	6.187	2.00
	Average	15.617	249.823	148.362	14.00
	Standard Deviation	2.86	45.36	245.638	14.42

MACROSCOPIC MODEL

Regression models are built to correlate 15 minutes trip attractions and physical features (gross floor space, number of stores, availability of parking, availability of restaurant, etc.) of the shopping centers. In regression model, 15 min trip attractions are taken as dependent variable and physical features are taken as independent variables. Many models were estimated, but only the models having corrected signs and reasonable magnitudes of the parameters and higher values of goodness of fit are presented here. The models are estimated from the surveyed data provided in Table 1. All the model parameters are statistically significant. The Best two models are described as follows:

Model 1: $Y = 35.480 + 0.077 X_1 + 63.136 X_2 + 23.739 X_3$; $R^2 = 0.786$; *Adjusted* $R^2 = 0.464$

Where X_1, X_2 and X_3 are the gross floor area per 1000 sq ft, availability of parking and restaurant respectively; and Y is the average trip attractions of shopping center with respect to number of persons per 15 minutes. The sign coefficients of X_1, X_2 and X_3 are positive, which is according to our expectation. The value of R^2 is 0.786 represents that there is strong positive correlation and if the value of the variables increase, the trip attractions of the shopping center will increase.

Model 2: $Y = 34.036 + 0.10 X_1 + 53.984 X_2 + 23.778 X_3$; $R^2 = 0.850$; *Adjusted* $R^2 = 0.625$

Where X_1, X_2 and X_3 are the total numbers of stores, availability of parking and restaurant respectively; and Y is the average trip attractions of shopping center with respect to number of persons per 15 minutes. This sign and magnitude of the model parameter is also according to our expectation. The goodness of fit ($R^2 = 0.850$) is very much satisfactory.

SUMMARY AND CONCLUSION

This research was conducted to determine trip attraction rates of shopping centers in the context of Bangladesh. Unlike previous studies, average trip attraction rates are determined for two types of shopping centers: one for medium size (Type 2) and small size (Type 3) shopping centers, located at

Dhanmondi, Gulshan and Siddeswari within Dhaka City. It is found that the average attractions rates of small size shopping centers are much higher than the medium size shopping centers. This implies the justification of determining average trip attraction rates of different type shopping centers based on sizes, rather than using same rate for every size shopping centers. Attraction rates are determined with respect to per 1000 sq. feet gross floor area, per 100 employees, per shop and per 10 parking spaces. It was found that the attraction rates per 1000 sq. feet gross floor area are consistent and easier to determine than others. On the other hand attraction rates per shop were most inconsistent and difficult to determine.

In this study regression models were also developed to determine attractions of the shopping centers. Two models were developed: one is function of gross floor area in 1000 sq. feet, availability of parking and availability of restaurant; and other is the function of number of stores, availability of parking and restaurant. This study gives fundamental information about trip attraction characteristics of shopping centers in populated city like Dhaka, thereby giving a comprehensive idea of contributing factors influencing trip attraction rate for establishing new shopping centers. Therefore provides a potential help to the transportation planners to estimate trip attractions of any shopping centers by using either of the models based on the availability of the data.

The main drawbacks of this study are the limited number of studied shopping centers and factors considering for the study. Macroscopic model would not consider the type of stores in the shopping center. It only considers the physical features as a function of trip attraction rate of the shopping centers. It will create complication if two shopping centers with different composition of stores exists similarities in physical features. The model also considers food court area, restaurant and office area of the shopping center, which is not correctly indicating gross floor area. For more accuracy of the model number of people visited each stores should be counted and visible factors should be considered. As very few studies have been conducted regarding trip attraction rate of shopping centers in Dhaka, this study will be helpful for further researches. In future, attraction rates of large shopping centers like Bashundhara City and Jamuna Future Park, and special retail stores (type-4) will be estimated.

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DEVELOPMENT OF AN M/M/C/K STATE-DEPENDENT MODEL FOR THE MANAGEMENT OF PEDESTRIAN MOVEMENTS ON SIDEWALKS

K. Rahman[1*], N. A. Ghani[1], A. A. Kamil[1] & A. Mustafa[2]

[1] School of Distance Education, Universiti Sains Malaysia 11800 USM, Penang, Malaysia.
[2] School of Mathematical Sciences, Universiti Sains Malaysia 11800 USM, Penang, Malaysia.
**Corresponding Author, <khalid_sust@yahoo.com>*

ABSTRACT

On sidewalks in the Central Business Districts (CBD) of a city, overflow of pedestrians is the main source of queuing delay which is caused by the capacity of sidewalks and management practices. The concept of flow management can be used to avoid the building of extreme queues of pedestrians on sidewalks in the CBD. Thus, in this paper, the application of M/M/c/K state-dependant queuing models for managing the flow of pedestrians and the design of sidewalks in the CBD is discussed. The paper also derives some performance measures that are useful for such management and the design of sidewalks in the CBD.

Keywords: Pedestrians; Sidewalks; Management; Design; State-dependent Model.

INTRODUCTION

Flow management is one of the common means to ensure proper quality of service with limited facilities in transportation and communication industry. An efficient flow management approach is the one that adjusts the input flow based on the appropriate operating characteristics to provide optimum performances. Therefore, the real factors that have significant influences on the operating characteristics and the performances should be incorporated as much as possible in the flow management approach. The incorporation and the assessment of influences are usually done by using an analytical model. However, the opportunity to find an appropriate analytical tool is very limited as few real-world situations conform to the requisite assumptions.

The Central Business District (CBD) of a city is ideally suited for the provision of efficient pedestrian networks. Thus, the pedestrian facilities in CBD areas should not turn into bottlenecks/ high congestions. On the other hand, although the increment of capacity (additional line) eases the pedestrian jam, the necessity of additional line on the valuable land in CBD should be justified against the monetary cost and based on how the users rank delays due to congestion (Seneviratne & Morrall, 1985). Braess (1968) noticed that, in the congested traffic condition, addition of line to the capacity leads to an increase in the average travel time for everyone. This is because self-serving individuals cannot refrain themselves from utilizing the additional capacity, even though such utilization leads to deterioration in the average travel time. Thus, one way to avoid the bottlenecks is to manage the flow based on the local pedestrian flow characteristics (Rahman et al., 2012, 2013a) and to design the pedestrian facilities according to their demand for flow. The pedestrian flows and facilities should also be analysed with an appropriate analytical tool that can incorporate important determinants and functional factors. However, there are a small number of researches that have devoted to manage the pedestrian flow as well as the capacity based design of pedestrian facilities. Among those, Yuhaski & Smith (1989) and Mitchell & MacGregor Smith (2001) presented comprehensive analyses of pedestrian facilities and developed an analytical approximation methodology based on M/G/C/C state dependent queuing models to compute certain performance measures for pedestrian facilities. However, to overcome some limitations and to incorporate some important determinants and functional factors, as mentioned in the next section and thereafter, in this study we will develop the

M/M/c/K state-dependent queuing models to manage pedestrian flows and capacity based design of pedestrian facilities in open outdoor walking environment e.g. sidewalks.

The rest of the paper is organized in the following manner. The reasons for using M/M/c/K state dependent queuing models and the pedestrian speed model that can represent the state dependent services on sidewalks are presented in the next Section. Then the background to the use and development of the M/M/c/K state-dependent pedestrian flow management model to analyze a single link of a network of sidewalks is provided, which could be extended to analyse pedestrian flows through the network. The paper ends with conclusion and further research.

Queuing and Congestion Models for Sidewalks

Crowd on the sidewalks in a CBD sometimes brings the pedestrian movements to a standstill. In such situations, a pedestrian could not freshly join the queue on the facility because the facility is already at the capacity (i.e. balk). Balking is not only a common phenomenon in the pedestrian movements, but it is also frequently observed in vehicle traffic, machine repair models etc. In 1917 Danish mathematician A K Erlang gave a formula for loss and waiting time based on M/M/s/s system, which was soon used by many telephone companies in different countries (Bojkovic et al., 2010; Chung et al., 1993). The loss formula is known as Erlang's B formula, where B stands for blocking, and can be used for estimating the probability of balking in telephone, cable etc. This loss formula has also been used by Yuhaski & Smith (1989) and Mitchell & MacGregor Smith (2001) in the performance measures and the planning of pedestrian facilities and networks. However, the following reasons have stimulated to adopt the M/M/c/K system based queuing models rather than M/G/C/C queuing models for the design and analysis of pedestrian networks in open outdoor walking facilities:

i. The M/G/C/C state dependent queuing models consider that the queue consists entirely of the walking facility without any buffer space. Such consideration may be applicable in an emergency evacuation from a building or in circuit switching. However, it is not reasonable for the uninterrupted and moving pedestrians in an open outdoor walking facility/ sidewalk.

ii. The waiting/lingering time, and the lateral spacing required for the movements of a pedestrian on a walking facility are not explicitly reflected in the formulation of the corresponding congestion models.

iii. The congestion models for M/G/C/C state-dependent queuing models do not support the observation of Polus et al., (1983), that is to say, up to the densities of about 0.6 ped./m^2 the free flow condition remains valid on sidewalk facilities.

The M/M/s system based congestion models as derived in Eq. (1) based on the model developed by Rahman et al. (2013b), which has been empirically validated, and M/M/c/K state-dependent pedestrian flow models, as formulated in the following Section, could triumph over these shortcomings to study the pedestrian movements and flow management on sidewalk facilities, and designing of such facilities.

Pedestrian Walking Speeds on Sidewalks

Many studies on pedestrian movements have been carried out under different conditions. The studies found that the patterns of pedestrian movements under the normal and emergency conditions are not same. However, there are some common personal factors such as age, gender, intelligence, and physical fitness of a pedestrian have significant influence on the pedestrian speeds in any walking condition and environment. Since pedestrian free flow speed is mainly influenced by the pedestrian variables or personal attributes, the inclusion of most common factors to the modelling of pedestrian movements can be done by bringing free flow speed to the corresponding congestion models.

In a public walkway facility, the usual movements of a pedestrian are hindered by the presence of other pedestrians (Older, 1968). Thus, on the sidewalks in a CBD, interaction with other pedestrians is the most important factor that influences the pedestrian speed and flow. People moving on the sidewalks are not always in huge numbers, but certainly at high densities. On the sidewalks, pedestrians usually travel at a maximum density of 1.55 ped./m^2 (normal capacity), whereas pedestrian free flow speeds start to decline at a density of 0.6 ped./m^2 and 'usual jam' (the facility is at the capacity) occurs at densities of about 3.32 ped./m^2 (Polus et al., 1983; Rahman et al., 2013a, 2013b). As far as continuous pedestrian movements on sidewalks is concerned, 'usual jam' leading to 'solid jamming' seemingly takes place at densities in the range 4 to 5 ped./m^2, which is very rare to be occurred. Rahman et al. (2013b) have developed a non-linear analytical model for pedestrian speeds on sidewalks, which supports the above mentioned empirical results and can be expressed as Eq. (1) as a function of the number of pedestrians on a sidewalk.

Congestion Model for Sidewalks

In a congested situation, the three variables that completely describe the pedestrian traffic flows are speed, flow and density. Pedestrian flows moving on a facility represent pedestrians' demand. Depending on the volume of flow and other factors (e.g. personal attributes) speed and density will fluctuate. Confronting the demand for flow and the capacity of the facility (supply) determines the operating characteristics and the performances of a pedestrian infrastructure under investigation. Speed is a key measure to the quality of service (service rate) provided to the pedestrians on the facility and as such determines the effectiveness of the facility infrastructure (Council, 2000). In addition to personal attributes, the average speed of pedestrians is influenced by many other factors including the purpose of the journey, the physical nature of the walkway, the nature of the surrounding area, and weather (Al-Azzawi & Raeside, 2007). However, for the purpose of capacity analysis only concentration (the number of pedestrians in the facility) should be considered (Navin and Wheeler, 1969). Hence, the speed-density relationship on sidewalks developed by Rahman et al. (2013b) has been adopted in this study and it can be expressed as the following as a function of the number of pedestrians on a sidewalk.

$$v_m = \frac{v_f}{\left[1 + \dfrac{\left(\dfrac{m}{c}\right)^s}{s\left(1-\dfrac{m}{s^*c}\right)\left(\dfrac{m}{c}\right)^s + s(s!)\left(1-\dfrac{m}{s^*c}\right)^2 \sum\limits_{n=0}^{s-1}\left(\dfrac{\left(\dfrac{m}{c}\right)^n}{n!}\right)}\right]} \tag{1}$$

where v_m = average walking speed of *m* pedestrians on the facility (m/sec);

v_f = average free flow speed of a pedestrian (m/sec);

c = *1.55*W*L* = the normal capacity of a sidewalk facility (ped.);

W = the width of the facility (m);

L = the length of the facility (m);

m = number of pedestrians on the facility, $m = 1, 2, \ldots, c, \ldots, K(= 2c)$;

s = $\frac{W-1.07}{b}$ = number of pedestrian lanes on the facility, where 1.07 m of width is reduced to calculate the effective width of the facility (Navin and Wheeler, 1969) and

b = lateral spacing required for a pedestrian to move on the facility (m) = 0.8m (Council, 2000).

As mentioned in the previous sub-section, pedestrians on sidewalks usually move at maximum density of 1.55 ped./m^2. Therefore, the normal capacity, c, is equal to 1.55 times the area of the facility in square meters (m^2). Hence, in the above, c is expressed as 1.55*W*L. In addition, the 'usual jam' (the facility is at the capacity) occurs at densities of about 3.32 ped./m^2. Thus, it is reasonable to consider that the facility will be at jam capacity when there are more 2c pedestrians on the facility. Therefore, in the above, it is considered that the highest number of pedestrians on the facility could be up to K = 2c. For illustration and experimentation purposes, we will be confined ourselves to the above considerations and assumptions.

Analytical Model for a Single Sidewalk

It is mentioned in the previous sub-section that pedestrian speed, flow and density (the number of pedestrians on a facility) completely describe the pedestrian traffic flows in a congested situation. Thus, it is always favourable if the performance measures of a pedestrian facility could be formulated based on these three variables. In the study of stochastic nature of pedestrian flows, when two of the three variables are known, the traffic operator can manage the third one to meet performance measures to the targeted magnitudes. Based on the capacity of a facility, the pedestrian flow ascertains the operating characteristics and the performances of an infrastructure under investigation. Thus, the managing of pedestrian flows for given speeds and the number of pedestrians on a facility is more convenient in terms of sharply response to the stochastic changes in pedestrian traffic conditions.

In the development of analytical models for understanding the stochastic nature of pedestrian flows, we can define a single sidewalk as a station where pedestrians are served. It is considered that the sidewalk has c servers in the normal capacity (as discussed in the previous section), which provide facilities to the pedestrians to pass the sidewalk without overflowed. A pedestrian can be both an output and an input to the queuing system within the sidewalk. Here, we assume that pedestrians enter the sidewalk in accordance with a Poisson process with rate λ (and thus the inter-entrance times are exponentially distributed), enter the sidewalk if it is not in jam capacity K, and then spend an exponential amount of time on the sidewalk with rate μ_m being served. The service time is equal to the travel time required for a pedestrian to pass the entire length of the sidewalk. The travel time and hence the service rate, μ_m, is state dependent as the travel time of each pedestrian within the sidewalk depends on the number of prevailing pedestrians on the sidewalk. It is assumed that, at each moment of time, pedestrians are uniformly distributed over the sidewalk. Thus, for m pedestrians on the sidewalk, the service rate will be a function of m i.e. $f(m)$. Since the sidewalk normal capacity is c and jam capacity is K, we can model the stochastic nature of pedestrian flows on a sidewalk with a queuing model. Thus, our considered model, in Kendall notation, could be described as M/M/c/K.

From the balance equations for M/M/c/K queuing system, we have the steady-state probabilities p_m (m = 1, 2…,c,…..K) of *m* pedestrians on the sidewalk as

$$p_m = \begin{cases} \dfrac{\lambda_0 \lambda_1 \ldots\ldots\ldots \lambda_{m-1}}{\mu_1 \mu_2 \ldots\ldots\ldots \mu_m} p_0 & \text{for } 1 \le m \le c \\ \dfrac{\lambda_0 \lambda_1 \ldots\ldots\ldots \lambda_{m-1}}{\mu_1 \mu_2 \ldots\ldots \mu_c (\mu_c)^{m-c}} p_0 & \text{for } c \le m \le K \end{cases} \tag{2}$$

From the normalizing condition for p_0, probability of no pedestrian on the sidewalk, we have

$$p_0 = \left(1 + \sum_{m=1}^{c-1} \frac{\lambda_0 \lambda_1 \ldots\ldots\ldots \lambda_{m-1}}{\mu_1 \mu_2 \ldots\ldots\ldots \mu_m} + \sum_{m=c}^{K} \frac{\lambda_0 \lambda_1 \ldots\ldots\ldots \lambda_{m-1}}{\mu_1 \mu_2 \ldots\ldots \mu_c (\mu_c)^{m-c}} \right)^{-1} \tag{3}$$

In the flow management modelling, we consider that the arrival rates will be controlled (for example, by using a roundabout on the middle or in the entry and exit points of the sidewalk) and hence arrival rates are not influenced by the number of prevailing pedestrians, *m*, on the sidewalk. We, therefore, assume that the arrival rates of flows are constant such that $\lambda = \lambda_0 = \lambda_1 = \ldots\ldots = \lambda_c = \ldots\ldots = \lambda_{K-1}$ and then we have from Eqs. (2) and (3)

$$p_m = \begin{cases} \dfrac{\lambda^m}{\prod_{i=1}^{m} \mu_i} p_0 & \text{for } 1 \le m \le c \\ \dfrac{\lambda^m}{(\mu_c)^{m-c} \prod_{i=1}^{m} \mu_i} p_0 & \text{for } c \le m \le K \end{cases} \tag{4}$$

and

$$p_0 = \left(1 + \sum_{m=1}^{c-1} \frac{\lambda^m}{\prod_{i=1}^{m} \mu_i} + \sum_{m=c}^{K} \frac{\lambda^m}{(\mu_c)^{m-c} \prod_{i=1}^{m} \mu_i} \right)^{-1} \tag{5}$$

where μ_i, for i = 1, 2…..*m*, is a function of i, the number of pedestrians on the sidewalk. Since the number of prevailing pedestrians affects the average pedestrian speed/travel time and hence the service rates, we can use the non-linear congestion model of Eq. (1) to describe μ_m on a single sidewalk. Note that the service rate, r_m, when there are *m* pedestrians on the sidewalk, is equal to the inverse of the average time that is required for a pedestrian to traverse the length of the sidewalk; therefore,

$$r_m = \frac{v_m}{L} \tag{6}$$

Since, in our consideration, *m* servers simultaneously serve on the sidewalk facility, we will have the overall service rate for *m* pedestrians as

$$\mu_m = m r_m = m\frac{v_m}{L} \tag{7}$$

By substituting the expression for μ_m, from Eq. (7) into Eqs. (4) and (5), we obtain the steady-state probabilities as

$$p_m = \begin{cases} \dfrac{[\lambda E(S)]^m}{m!\prod\limits_{i=1}^{m} f(i)}\, p_0 & \text{for } 1 \le m \le c \\[2ex] \dfrac{[\lambda E(S)]^m}{c!(c)^{m-c}\prod\limits_{i=1}^{m} f(i)}\, p_0 & \text{for } c \le m \le K \end{cases} \tag{8}$$

and

$$p_0 = \left(1 + \sum_{m=1}^{c-1} \frac{[\lambda E(S)]^m}{m!\prod\limits_{i=1}^{m} f(i)} + \sum_{m=c}^{K} \frac{[\lambda E(S)]^m}{c!(c)^{m-c}\prod\limits_{i=1}^{m} f(i)}\right)^{-1} \tag{9}$$

where $E(S) = \frac{L}{v_f}$ is the expected service time/ traverse time for a pedestrian in free flow condition in a sidewalk of length L and

$$f(m) = \frac{v_m}{v_f} = \left[1 + \frac{\left(\frac{m}{c}\right)^s}{s\left(1-\frac{m}{s*c}\right)\left(\frac{m}{c}\right)^s + s(s!)\left(1-\frac{m}{s*c}\right)^2 \sum\limits_{n=0}^{s-1}\left(\frac{\left(\frac{m}{c}\right)^n}{n!}\right)}\right]^{-1}$$

is the ratio of average speed of *m* pedestrians on the sidewalk to that of speed in free flow condition. Since the speed has been expressed as a function of number of pedestrians or density, for a particular number of pedestrians on a given facility, the steady-state probabilities and the corresponding performance measures (as will be discussed in the next section) will depend on the arrival rates i.e. on the pedestrian flows.

Conclusion and Further Research

The use of M/M/c/K queuing models for analysing a single sidewalk allows us to compute certain performance measures in equilibrium condition. The most relevant performance measures include

i. The probability of balking (p_{Balk}) is equal to p_m where m equals K,

ii. $E(Q)=\sum_{m=c+1}^{K}(m-c)p_m$ is the average number of pedestrians waiting in the queue in the equilibrium condition,

iii. $E(T)=\dfrac{\sum_{m=1}^{K} mp_m}{\theta}$ is the expected amount of time a pedestrian spends on the facility,

iv. $\theta=\lambda(1-p_{Balk})$ is the pedestrian effective arrival to the sidewalk or throughput through the sidewalk.

The most important factors that influencing the behaviour of pedestrian flows and the related performances of a walking facility include pedestrian arrival flow rate, λ, length of the facility L, width of the facility W and pedestrian personal capacity v_f.

What is intended to do in a further study is to examine the effects of these factors on performances. Such examinations will be useful for proper management and the design of sidewalks in the CBD.

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SEASONAL VARIATION AND THE EFFECTS OF SUBGRADE RESILIENT MODULUS ON PAVEMENT RUTTING USING MEPDG

M. M. Rahman[1]*, S. L. Gassman[1] & M. M. Tamim[2]

[1] *Department of Civil and Environmental Engineering, University of South Carolina, Columbia, South Carolina, USA.*

[2] *Department of Civil Engineering, Southern University Bangladesh, Chittagong, Bangladesh.*

**Corresponding Author, <rahmanmm@email.sc.edu >*

ABSTRACT

Rutting is one of the most important asphalt pavement distresses because it is responsible for both the functional and structural condition degradation of the flexible pavement. There are limited studies on the effect of modulus of resilience (MR) of subgrade on pavement rutting in the Mechanistic-Empirical Pavement Design Guide (MEPDG). Moreover, there have not been any studies that related seasonal variation of subgrade to rutting in MEPDG. In 2008, the American Association of State Highway and Transportation Officials (AASHTO) released a modified pavement design method based on Long Term Pavement Performance (LTPP) data from all over the country. The MEPDG default design parameters developed from the LTPP database are expected to be significantly different for South Carolina material, traffic and weather conditions, thus the default design parameters may not be accurate for South Carolina. Therefore, the new pavement design method should be calibrated for South Carolina conditions by performing MEPDG local calibration. A complete literature review on seasonal variation and the effects of subgrade MR on pavement rutting to perform MEPDG local calibration for South Carolina is presented in this paper. Results showed that, among different material inputs, MR of subgrade has the most significant impact on pavement rutting with a regression coefficient of -0.65. Subgrade strength varies with different weather conditions, changing temperature and water content. One previous study showed that the pavement subgrade had a higher MR in the winter than in the summer. Therefore, the effect of the seasonal variation of subgrade MR on pavement rutting should be studied in MEPDG.

Keywords: Pavement Subgrade, Rutting, MEPDG, Local Calibration

INTRODUCTION

Flexible pavement deteriorates with time and traffic activity in different weather conditions. Among the 16 types of flexible pavement deterioration, rutting is one of the most important distresses (Behzadi and Yandell, 1996). Rutting is defined as the pavement surface depression in the wheel paths and is closely related to the permanent deformation of any of the pavement layers or subgrade layers. It originates from the lateral movement of pavement material due to repeated traffic loading. Rutting is categorized as a structural distress which affects both the riding quality and pavement structural health. Severe rutting can cause a major structural failure of the pavement (Shahin, 2005). The structural life of a pavement depends on both the strength of the pavement and subgrade materials and the traffic and climate conditions.

The Mechanistic Empirical Pavement Design Guide (MEPDG) is currently being used throughout the United States to design asphalt pavements because of its ability to deal with in-situ materials, new materials and the changing load types (Souliman et al., 2010). This change in pavement design from an empirical based method to a mechanistic based method is recent, and currently different departments of transportation in the U.S. are performing local calibrations of the MEPDG for their states. To predict the rut depth, MEPDG determines the permanent deformation of different structural layers based on different transfer models (Waseem and Yuan, 2013). Different layer coefficients and subgrade parameters are required to run these transfer models. The current MEPDG represents a general national condition based on Long Term Pavement Performance (LTPP) database and does not represent South Carolina material and climate conditions; therefore local calibration is required (Baus and Stires, 2010).

Given the need for local calibration of MEPDG in South Carolina, the purpose of this paper is to present a summary of the studies performed on seasonal variation and the effects of subgrade MR on pavement rutting to perform MEPDG local calibration. All available literatures on MEPDG local calibration with material inputs were located and studied to identify research gaps that need to be filled for the local calibration process. These findings were then used to develop research questions aimed to understand the relationship between the seasonal and moisture variation of subgrade MR and pavement rutting in MEPDG and to propose a plan for future study.

MEPDG CALIBRATION METHODOLOGY

The research framework in the form of a concept map is shown in Fig. 1. This figure shows the MEPDG local calibration process with the major primary literature cited in different parts of the literature review that will be presented in the next section. MEPDG local calibration methodology is shown at the centre of the concept map with a circular arrow. Three different types of inputs should be considered to perform the design of the Asphalt, Base and Subgrade layers: local material, local climate and local traffic. MEPDG compares the output or the predicted distress with the original distress to minimize the residual error and to determine the calibration coefficient.

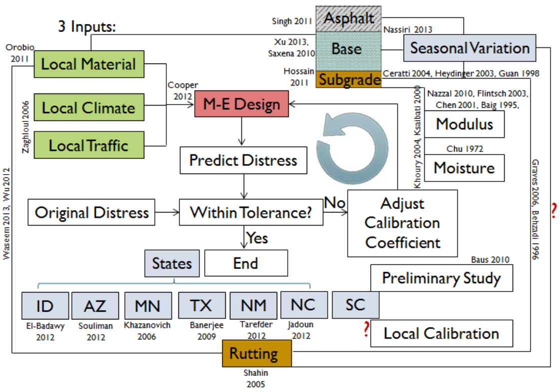

Figure 1 MEPDG Local Calibration Process with Number of Major Literatures Cited

The approach used in the MEPDG is based upon calculating incremental distortion or rutting within each sub-layer. The model for calculating total permanent deformation uses the plastic vertical strain under specific conditions for the total number of trucks within that condition. The empirical model for pavement rutting transfer function in MEPDG is shown below (Tarefder and Rodriguez-Ruiz, 2013):

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k1} T^{k2\beta r2} N^{k3\beta r3} \qquad (1)$$

$$k_z = (C_1 + C_2 d) 0.3281960^d \qquad (2)$$

$$C_1 = -0.1039 h_{AC}^2 + 2.4868 h_{AC} - 17.342 \qquad (3)$$

$$C_2 = 0.0172 h_{AC}^2 - 1.7331 h_{AC} + 27.428 \qquad (4)$$

where ε_p = plastic strain (in./in.), ε_r = resilient strain (in./in.), h_{AC} = total asphalt concrete thickness (in.), T = AC layer temperature (F), N = number of load repetitions, d = depth of the point where strain in being determined (in.), k_z, k_1, k_2, k_3 = laboratory constants, $\beta_{r1}, \beta_{r2}, \beta_{r3}$ = calibration coefficients.

The empirical model for subgrade rutting transfer function in MEPDG is shown below:

$$\delta_a = \beta_{s1} k_1 \varepsilon_v h \frac{\varepsilon_0}{\varepsilon_r} e^{-\left(\frac{\rho}{N}\right)^{\beta}} \qquad (5)$$

where δ_a = permanent deformation of the unbound layer (in.), ε_v = average vertical strain (in./in.), h = thickness of the unbound layer (in.), $\varepsilon_o, \beta, \rho$ = material properties, β_{s1} = calibration coefficients to optimize for both base and subgrade.

LITERATURE REVIEW

A summary of the different literatures on are presented in this section.

El-Badawy and Bayomy (2012) studied the performance of MEPDG dynamic modulus predictive models for the asphalt layer in local calibration for Idaho. In that study, the authors compared two MEPDG models: National Cooperative Highway Research (NCHRP) Program 1-37A and NCHRP 1-40D for dynamic modulus prediction. Several cases were considered in that study to analyse the performance of the two predictive models in the MEPDG software. Statistical goodness-of-fit was used to determine the accuracy of the model. Degree of bias was determined by the regression analysis of the measured and the predicted modulus. Laboratory testing was conducted for Hot Mix Asphalt (HMA) mixes in Idaho, which included binder testing (Dynamic Shear Rheometer and Brookfield rotational viscosity) and dynamic modulus testing. The performance of Witczak dynamic modulus predictive model has been studied by Singh et al. (2012) for Oklahoma.

Simulation of the base layer material resilient modulus effects on MEPDG was studied by Xu et al. (2013). In performing the sensitivity analysis in MEPDG software, pavement rutting and fatigue cracking were considered. Monte-Carlo simulation was performed in the sensitivity analysis. Characterization and performance modeling of cement stabilized base layer in MEPDG was performed by Saxena et al. (2010) in another study.

Subgrade MR was studied for MEPDG in several studies by Hossain et al. (2011) and Khazanovich et al. (2006). MR data from 39 different counties throughout Oklahoma were evaluated by Hossain et al. (2013). Collected data of this study were randomly separated into two groups: data from 21 counties were used for developing the model and for evaluation; data from the other 18 counties were used for validation. Different stress based regression models were evaluated using statistical software. Detailed sampling and laboratory procedures to determine subgrade MR and different regression models were provided in the literature. Khazanovich et al. (2006) also studied the subgrade MR for Minnesota. They used two standard test methods for laboratory testing: NCHRP Project I-28 or Harmonized Test Methods for Laboratory Determination of MR for Flexible Pavement design and AASHTO T 307 for determining the MR of Soil. Sensitivity analysis was then performed with MEPDG to evaluate the MR for Minnesota subgrade. Moreover, falling weight deflectometer and dynamic cone penetrometer were used to determine the in-situ modulus in studies by Nazzal and Mohammad (2010), Flintsch et al. (2003), Chen et al. (2001), and Ksaibati et al. (2000). Determination of subgrade MR using bender elements in the laboratory was shown in a study by Baig and Nazarian (1995). Furthermore, there were limited studies on the effect of subgrade MR on the pavement permanent deformation (rutting) model in MEPDG (Graves and Mahaboub, 2006).

Pavement material characteristics for different pavement layers are prone to change with different temperature and moisture content. Evaluation of MEPDG seasonal adjustment factors for the unbound layer’s moduli has been shown in a single study (Nassiri and Bayat, 2013). In South Carolina, limited data on the seasonal variation of subgrade strength was obtained by Chu (1972). In that study, field tests were performed at select sites in South Carolina to examine subgrade moisture variations under existing pavements. The study recommended a complete moisture variation study below South Carolina pavements in connection with pavement performance and design. Several studies have been performed to determine the seasonal variation of the subgrade MR (Ceratti, 2004; Khoury and Zaman, 2004; Heydinger, 2003; and Guan et al., 1998). However, none of these studies has shown the effects of seasonal variation of subgrade MR in the MEPDG. Ceratti (2004) performed both laboratory tests and in situ tests to determine the seasonal variation of subgrade soil MR in Southern Brazil. Laboratory testing was carried out to establish the relationship between water content and soil suction. The MR was found for soil specimens submitted to drying, wetting, or wetting-after-drying paths. Jet-filled tensiometers were used to determine soil suction in different pavement test sections. A traffic simulator was also used in this study to measure the deflection. A relation between MR, moisture variation and soil suction for subgrade soils was developed by Khoury and Zaman in 2004. A new laboratory procedure for wetting and drying specimens was introduced in their study. Heydinger

(1998) evaluated the seasonal variation of subgrade soil for Ohio as part of LTPP instrumentation project seasonal monitoring program (SMP).

RESULTS AND DISCUSSION

Material properties are required to determine pavement responses in ME design. To predict the states of stress, strain and displacement within the pavement structure due to the wheel loading, pavement material properties are needed. Previous studies showed that pavement material characteristics have a significant influence on MEPDG (Hossain et al., 2011; Xu et al., 2013; Singh et al., 2013; El-Badawy, 2012; Khazanovich et al., 2006; and Saxena et al., 2010).

One particular study by Orobio and Zaniewski (2011) examined each of the pavement material characteristics and determined the sampling based sensitivity analysis of the MEPDG applied to material input. They studied 30 parameters and 11 of those showed significant effects on rutting. Fig. 2 shows the effects of those 11 material inputs on pavement rutting. The bar indicates the regression coefficients for different material properties. Five of the 11 significant parameters have negative regression coefficients, and the other six significant parameters showed positive relations. The pavement structures contained three asphalt concrete layers of 2 in., 3 in. and 10 in. respectively at the top of one asphalt treated base layer of 3 in. and the subgrade. A positive regression coefficient indicates that MEPDG rutting increases with increasing input values; whereas, a negative regression coefficient indicates that, as the parameter increases, pavement rutting decreases. The study found that the MR of subgrade had the largest effect on the rutting predicted from MEPDG. Baus and Stires (2010) also performed a sensitivity analysis and reported similar findings regarding material inputs. They found that subgrade MR had a significant influence on IRI, total rutting, alligator cracking and longitudinal cracking for pavements in different counties. They recommended a comprehensive subgrade investigation to determine modulus for South Carolina.

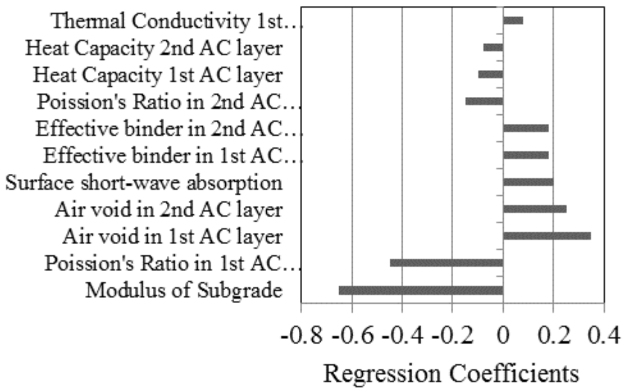

Figure 2 Effects of Material Input (Orobio and Zaniewski, 2011)

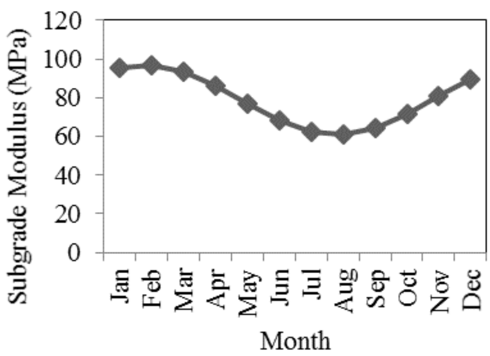

Figure 3 Seasonal Variation of Subgrade MR (Heydinger, 2003)

Several studies evaluated the effects of subgrade modulus on MEPDG; however, there were no studies on the effect of the seasonal variation of subgrade modulus on MEPDG. Only a few studies on the seasonal variation of subgrade MR were performed (Ceratti et al., 2004, Heydinger, 2003, and Guan et al., 1998). The study by Heydinger (2003) examined the effect of seasonal changes on subgrade soils. Fig. 3 indicates that there is a significant difference in subgrade MR in summer and winter. Furthermore, none of the literatures reviewed in this study have shown how pavement rutting in MEPDG is affected by seasonal variation of subgrade strength.

Based on a review of the literatures summarized herein, the influence of different material inputs in different mechanistic pavement design outputs requires more study. As shown in the concept map in Figure 1 there have been limited studies on MEPDG local calibration and no studies in South Carolina. Furthermore, there is a lack of a complete study to relate the seasonal and moisture variation of subgrade MR to pavement rutting in MEPDG; therefore, the seasonal variation of the in-situ subgrade modulus needs to be studied. Research on subgrade seasonal variation will lead to an improved understanding of the behaviour of subgrade soil in different temperature and moisture conditions. This is important because South Carolina soil characteristics are different than other states and the MR of South Carolina soils will affect the MEPDG.

CONCLUSIONS

The MEPDG local calibration has been performed for a number of other states (Tarefder and Rodriguez-Ruiz, 2013; Jadoun and Kim, 2012; and Banerjee et al., 2009); however, it has not yet been performed for South Carolina. With increasing pavement age, pavement distresses (i.e. rutting, alligator cracking) increase at a different rate depending on the pavement materials, climate and traffic condition. Results from different literatures that studied MEPDG local calibration have shown that local material, local climate and local traffic all have significant effects on pavement permanent deformation or rutting. Among different local material properties, the subgrade MR has been shown to have the most significant impact on pavement rutting with regression coefficients of -0.65 (Orobio and Zaniewski, 2011). Moreover, the subgrade MR varies in different weather conditions with changing temperature and water content (Heydinger, 2003). However, the effect of seasonal variation of subgrade modulus on pavement rutting in MEPDG has not yet been studied. Therefore, to fill in these information gaps, research is currently being conducted to study pavement rutting for South Carolina materials, climate and traffic conditions. The methodology of this study is to determine the subgrade MR by in-situ and laboratory tests and to study the effects of subgrade MR seasonal variation on pavement rutting.

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THE ROLE AND INTEGRATION OF NON-MOTORIZED TRANSPORT FOR EFFICIENT PUBLIC TRANSPORT SYSTEM IN DHAKA

Md. Mazharul Hoque[1*], S.M.A. Bin Al Islam[1] & Debashis Saha[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka – 1000, Bangladesh.*
**Corresponding Author, <dirarc@gmail.com >*

ABSTRACT

The transport system in Dhaka has been adversely affecting the general commuters and the community at large. Road traffic congestion continues to remain a major problem and indeed is deteriorating rapidly resulting in massive socio-economic losses. The greater challenge thus for transportation professionals is to develop a system of urban transport that meet the basic mobility needs for all urban dwellers at desirable safety and avoiding the unacceptable level of congestion and its consequent overwhelming adverse environmental effects. Augmentation of efficient public transport system is seen to contribute specifically for mitigating the congestion problem. But the role of public transport to solve the existing traffic congestion efficiently is still in question especially due to its poor accessibility and service quality. The quality of public transport is determined not only by the quality of the main transport mode, but also by the availability of before (access) and after (egress) modes. Access and egress are the weakest links in a public transport chain. Access to affordable public transport services, from the city peripheries with inadequate access to public transport is indeed a vital issue for the urban population.

Non-Motorized Transport especially walking, bicycles and rickshaws have been very effective means of transport particularly at the short distances and could provide sustainable door-to-door service to the bus based commuters as well in Dhaka. The energy efficient modes of public transport and NMT thus can be complementary. Integration of NMT with public transport can cancel out the negatives of both systems and provide efficient and sustainable door-to-door service to the bus based commuters. Considering social acceptability, economic response, fuel free eco-friendly characteristics, the role of non-motorized transport in sustaining the traffic and mobility needs of citizens is becoming increasing important.

This paper investigates a better approach may be to integrate public transport and non-motorized vehicles as complementary forces in meeting the comprehensive demand of Dhaka's transport. With the backdrop and given the international significance of the problem, this paper will delineate a preliminary framework for the future place of the NMT. This paper further discusses the challenges and opportunities for its sustainable co-existence in a mixed mode transport stream that best meets the network performance needs of Dhaka.

Keywords: Public Transport, Non-Motorized Transport, Integration, Accessibility, Traffic Congestion, Dhaka.

INTRODUCTION

Historically non-motorized transport (NMT) especially walking, bicycling and rickshaw has been playing a crucial role in the transport system of Dhaka. NMT has a high spatial penetration rate, virtually every location can be reached by walking, bicycles, rickshaws etc. and is a fast and efficient means of transport particularly at the short distances. Rickshaw and walking are the predominant modes for low and middle income (about 97%) communities of Dhaka (DHUTS, 2010). Every day rickshaws make seven million passenger trips, covering eleven million miles. That is double what the

London underground does (http://www.sos-arsenic.net).Considering social acceptability, economic response, fuel free eco-friendly characteristics, the role of non-motorized transport in sustaining the traffic and mobility needs of citizens is becoming increasingly important. However, the planned provision and incorporation of non-motorized transport (NMT) modes in the motorized traffic stream are often neglected.

The public transport service is mainly dominated by buses and minibuses, the cheapest mode available as mass transit, and is constrained by poor service conditions: long waiting, delay on plying and long walking distance from the residence/work place to bus stoppages are some of the problems that users confront daily. This situation has resulted in deterioration in accessibility, level of service, safety, comfort and operational efficiency. Access to affordable and good quality public transport services is critical for the urban population, especially low-income communities located in the city outskirts with inadequate access to public transport. There is an ever increasing urgency of mitigating this complex urban transport problem by increasing the efficiency of the entire public transport system systematically through enhanced overall coordination among various modes. NMT can be helpful in this context. Such modal integration can cancel out the negatives of both systems and provide efficient and door-to-door service to the bus based commuters.

This paper aims at to discuss the prospects and role of NMT and investigates the benefits of integration of public transport and non-motorized vehicles as complementary forces in meeting the increasing demand of Dhaka’s transport. This paper further discusses the challenges and opportunities for its sustainable co-existence in a mixed mode transport stream that best meets the network performance needs of Dhaka.

1. NMT: THE GLOBAL CONTEXT

Modal Share of NMT in various cities: NMT is a major mode of urban transport system in many developing cities. NMT is not only important for the entire trip (from origin to destination), but also as a feeder mode to public transport. Although bicycle trips to and from public transport stops are only a small proportion of the total number of NMT trips, it is true that NMT has a role to play when it comes to the promotion of public transport. Its potential as feeder system is very high. Modal share for Public Transport (Bus, Train), NMT (Rickshaw/Pedi cab, walk, Bicycle), Private transport-Car, Motorbike, Others (Taxi, Auto rickshaw etc.) of various cities is shown in Figure 1 which indicates a varied range of NMT prevalence in various developing cities around the globe. For instance, in Bangalore the modal share of NMT is almost 30% whereas in Colombo the share of NMT is only 15%.This Figure also shows that the share of public transport is higher in most of the developing cities like Bangalore, Colombo, Hong Kong etc. (40-60 %) and in the developed cities like Melbourne private transport carry the major share (about 50 %). In the case of Dhaka modal share of NMT is the highest, accounting for nearly 60 percent of the total travel demand which reflects their enormous mobility contribution.

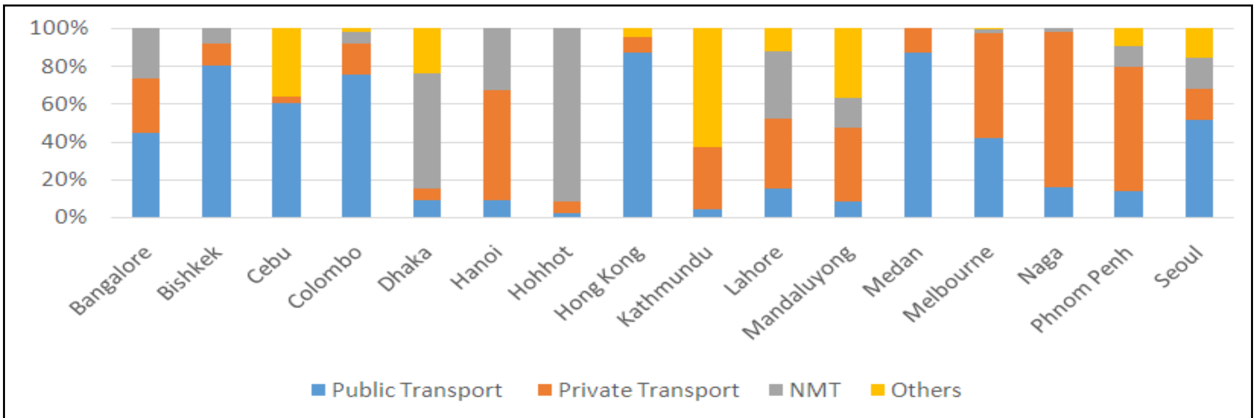

Figure.1- Modal Share of various cities (Source- derived from Transportation Statistics Transportation Information Sources by TDM encyclopedia)

Environment friendly mode: NMT is a core component of environmentally, economically and socially sustainable transport systems. The International Energy Agency (IEA) reports that greenhouse gas reductions in transport will come from three main sources, one of which is modal shifts, e.g. in urban short-distance travel made by NMT. In Bogota, for example, the cumulative CO_2emission has been reduced significantly because of NMT users leaving their car at home during the period 2000 to 2007. This noteworthy rapid total reduction is mainly due to the cycling modal share rising from 0.2% at the beginning of the project to 4% in 2007 (Kim et. al. 2010).

Modes of travel have varying effects on emissions of CO_2 and other greenhouse gases that cause climate change. According to ADB (2011) passenger cars and scooters are the least efficient means of travel whenconsideringCO_2emissions, while walking and bicycling put negligible CO_2 into the atmosphere. Buses and trucks are by and large are significant contributor in the case of SOx and NOx emissions. Auto rickshaws also contribute significantly in the case of SOx and COx emission. By contrast, the NMVs including rickshaw have almost negligible impact on climate change and thereby are most desirable ecologically**.**

Road space efficiency: NMT need less than a third of the road space that is used by a private vehicle, and a pedestrian needs only a sixth of that space (Kim et. al. 2010). Depending on vehicle size, occupancy or loading, and speed, the use of space can vary greatly for different modes of travel. This means that the potential volumes of passengers vary greatly by mode along a corridor. Clearly, the cycles and pedestrians are more spatially efficient mode than cars as they carry 14,000 and 9,000 people per hour respectively on 3.5 m wide lane in the city compared to only 2,000 people per hour carried by the car. Dense urban centers cannot effectively be served by the car, since not enough people can be delivered to the center (ADB, 2011).

In Dhaka, rickshaws transported almost 54% of vehicular passenger in return for 40% of road space they required, while cars, taking up 39% of the road space, transported less than 9% of the passengers (DTCB, 2005). Following rapid urbanization and growing prosperity, new roads are too often constructed with the purpose of accommodating the growing number of motorized vehicles. Efficiently NMT linked transport systems could potentially achieve the desired amount of sustained accessibility to revitalize and increase public transport capacity and thus relieving traffic congestion.

2. PROSPECTS OF NMT IN CONTEXT OF DHAKA:

Role of NMT in Dhaka's Transport System: As demonstrated earlier, *pedestrians* clearly form by far the single largest group of road users in terms of total catered number of trips in urban areas of Bangladesh. This is particularly (about 65%) for short trips to one mile (Hoque et al 2005). Walking as primary mode of travel is dominant in all age groups and income classed. These characteristics will continue to grow in the future as a large part of the urban population would hardly afford any kind of motorized transport of their own. Yet it is the motorist not pedestrian who normally receives the attention and greater share of priority. *Rickshaws* are the most significant mode of transport in Dhaka. Rickshaws account for 34% of all person trips in Dhaka increasing from 19% in 1998 (Louis Berger Group and BCL, 2005). The average length of rickshaw trips is 2.34 km and 61% of all rickshaw trips are made by people in the middle income levels (Tk12, 500 – 55,000 per month). It can be noted that almost 40% of the loaded rickshaws are being used by women and children, or people with goods and another 30% of users are students (Louis Berger Group and BCL, 2005). *Bicycles* comprised only around 2% of all vehicles and about 2% of the households own a bicycle. But in our recent survey an increased use of bicycle has been observed. With due attention, it can be the most demanding mode of transport in future Dhaka, particularly to the poor segment of the travelers. Expanded use of bicycles is crucial for the mobility of the urban poor.

Potential sector of employment: Rickshaw industry generates huge amount of non-skilled employment and maintains income of some of the most vulnerable urban dwellers. It is estimated that there are around two million rickshaw pullers across Bangladesh and that around 14% of the Bangladeshi population relies indirectly on rickshaw pulling for their livelihoods (their families, manufactures, garage owners, painters, repair men) . In Dhaka alone, 20% of the population relies on pulling or indirectly, which amounts to about 2.5 million people (Wipperman & Sowula 2007). It cannot be overstated how crucial the rickshaw is to employment and the socio-economic structure of Bangladesh, especially amongst the poorest sections of society. This must be an important consideration in transport planning for the future of Dhaka. Rickshaws contribute 34% of the value added from the transport sector to GDP (Gallagher, 1992).

Door to door accessibility: NMT has a high spatial penetration rate (virtually every location can be reached by bike or walking), NMT can be used throughout the day and is a fast and efficient means of transport particularly at short distance.

Available to everyone: Unlike cars and motor cycle which are private modes rickshaws are private transport available to all. Rickshaw typically serves 30-60 passengers per day while private car typically serve 2-6 passengers. 56% of walk trips are made by low income households compared to only 16% of those in the high income households and rickshaw is the dominant mode for both low and middle income households (STP, 2005a). For low income people, with extremely limited transportation choices, NMT is the most appropriate and efficient form of transport (World Bank,1998).

Serves routes where no other public transport available: NMT is also able to access neighborhood with narrow streets which are unsuitable for large motor vehicles. Private modes, including cycle and walking, simplify the operational integration since the modes are flexible and exclusively controlled by the user, enabling the passenger to be at the public transport station whenever it is necessary. Morethan50%of road is inaccessible to large sized vehicles particularly to public transports. In Dhaka city out of 1286 km road about 821km of road is found to be accessible (if road width is equal andmorethan4.5m) to motorized vehicles (Sohel, S. et.al.2010).

Highly resilient at times of crisis: Often the only transport available during hartals is rickshaw and cycle. Also, they are not vulnerable to oil crises, petrol or CNG shortages, or other socio-economic breakdowns. So in day of crisis city dwellers satisfy their needs of mobility through non-motorized vehicles.

3. Challenges and Shortcomings of NMT in Dhaka

Rickshaw Restriction Initiatives in Dhaka: Despite increased emphasize on the importance of accessible NMVs for people living in developing countries and the benefits of non-motorized transport in the face of climate change, the World Bank in 2001 suggested banning or movement segregation of motorized and NMT on major corridors for traffic flow improvement (Hummel 2008). In line with these guidelines as well as the recommendations of the Dhaka Urban Transport (DUTP) (1998) and the National Land Transport Policy (2004) adopted by the Government of Bangladesh (GoB), Dhaka City Corporation (the city government) planned for phasing out of cycle-rickshaws from eight major roads of Dhaka in order to make way for motorized transport (Manchetti 2005).Increasing limitations on rickshaws in Dhaka are likely to have considerable hardship to the poorest and most vulnerable segments of society, reducing the mobility of the middle class (particularly women, children, and the elderly), and contributing to air pollution and motorization.

Safety Concerns: Pedestrians are clearly the most vulnerable road user group in Dhaka. Pedestrian fatalities have significantly increased from 57 percent in 1998 to 66 percent in 2006 in urban areas. Indeed, the share of pedestrian fatalities in Dhaka is much higher, 86 percent, and there has been a sustained increase of pedestrian fatalities, as revealed in Figure 2 (Hoque et al., 2010).

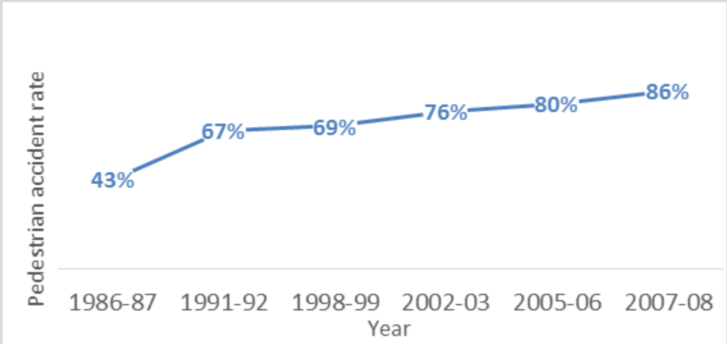

Figure2: Increasing Trends of Pedestrian Accident in Dhaka City

Cycle rickshaws and bicycles cater for substantial amount of urban travel and hence they are facing serious safety threats as well. Around 800 fatalities have been attributed to cycle rickshaws and bicycles in urban areas of Bangladesh during 1998-2006, which represented nearly 12 percent of total urban road deaths. On average 30 bicycle deaths and 60 cycle rickshaw deaths each year as reported by the police (Hoque et al., 2010).

Road accidents disproportionately affect the poor and their consequences plunge household into acute poverty. Poor people are forced to use non-standard and unsafe vehicles. NMV operators particularly rickshaw pullers are being unaware of traffic safety are putting lives at high risk.

Policies towards Transport Services at Operation in Dhaka City: As pointed earlier, NMTs are traditionally being discouraged compared to motorized transport. Although most of the trips (around 70%) are made by non-motorized transport, only 0.22% of the total investment has been allocated for the development of pedestrian facilities, rickshaw and bicycle facilities (STP 2005a).The comparative policy standing of government towards various modes operating in Dhaka is presented in Table 1.

Table.1: Government Policy towards Transport Services at Operation in Dhaka City (Source: Derived from STP (2005) and Critical Analysis and Rahman, M.M. Et.al. 2008).

Travel Mode	**Type**		**Government Policy**		
	Motorized	Non-motorized	Supportive^	Restrictive *	Negative**
Passenger					
Metro Train	●		●		
Bus	●		●		
Micro-bus	●		●		
CNG Auto rickshaw	●			●	
NMPT (Rickshaw)		●			●
Taxi	●	●		●	
Car/Jeep	●		●		
Motorbike	●		●		
Bicycle		●			●
Freight					
Truck	●		●		
Rickshaw van		●			●
Pushcart		●			●

^ Infrastructure design, finance and regulatory support;
* Restricted in terms of total fleet number
** Non-cooperation regarding infrastructure design, finance and regulatory support.

Shortcomings of NMT infrastructure in Dhaka: An extensive survey has been conducted by the authors to investigate the existing conditions and shortcomings of NMT infrastructure in Dhaka. Striking findings of the survey are:

- **Absence and/or poor footpath conditions:** Conditions of the most of the footpaths in Dhaka City are not pedestrian friendly; 44% of the streets do not have any footpaths; nearly 40% of the pedestrians could not walk where they wanted to (WBB Trust, 2011). In our survey it was found that in most of the cases footpaths were narrow, do not fit the standard requirements and was illegally encroached. Moreover, frequent garbage disposal, open sewer and water clogging in footpath lead to the deterioration of operational efficiency of footpaths.

- **Inadequate and unsafe pedestrian crossing facilities:** Non-standard crossings, absence of designated crossing facilities and encroachment of crossings etc. are seriously affecting the safer movement of pedestrians.

Figure3: Conditions of NMT infrastructure in Dhaka

- **Low usage of Bicycles:** The factors that are responsible for low usage of bicycle include infrastructure, cost, safety, status and culture (STP, 2005a).

- **Operational problems of Rickshaw:** Facilities are becoming increasingly limited resulting in serious congestion and bottlenecks. Fair disputes and inadequate facilities are also affecting rickshaw operations. The problem characteristics that are found from our recent survey among 220 rickshaw pullers and 378 passengers are given below:

Figure 4: Survey results on rickshaw pullers and passengers.

4. Some Ways of Creating NMT Friendly Facilities:

Conventional traffic/transport studies focused on vehicular movement rather than NMT. While large investment was made to improve vehicular traffic flow, except in a few cities, minimal budget has been allocated to improve the convenience/safety of NMT. The importance of pedestrians and NMVs in Dhaka has largely been neglected in planning for mobility improvement. Mechanized trips, however, also involve walking as feeder or transfer and there is an acute need to improve NMT facilities and safety considerations. To better cater the needs of NMT some of the policies and actions that deserve particular attention are described below:

- ***Balanced share of NMT***: The share of NMT investments in road and transport infrastructure projects should be balanced to meet the full range of people's mobility needs. Roads should facilitate the mobility of people and goods, without preference for some users over others.
- ***Strengthening institutional and professional capacities:*** Promoting further education and specialized training to develop a competent cadre of professional capacity to plan and design an intermodal transport system considering the context of NMT in order to provide a high level of accessibility, equity and environmental sustainability.
- ***Road design rationale:*** Roads should be designed so that the needs and requirements of vulnerable road users such as pedestrians and cyclists are protected from high speed impact including building NMT safety principles in the design concepts.
- ***Monitoring and evaluation of road performance:*** Indicators should be developed that can monitor and evaluate roads for their contribution to sustainable development in terms of environment, safety and accessibility.

5. Integration of NMT with Public Transport:

Augmentation of public transport is a must for achieving sustainability and our government is considering BRT and MRT as possible options. A modern BRT system for Dhaka will contribute to a forward looking image of the city. But it must be pre-planned to be efficient otherwise in the long run it will be ineffective like public bus service. Serious thoughts should be given towards improving supporting modes as well to utilize BRT, MRT otherwise it will lose its appeal. Because the users like to travel door to door but transport service like BRT, MRTs don't provide this facility.

The quality of public transport is determined not only by the quality of the main transport mode, but also by the before (access) and after (egress) modes. Access and egress are the weakest links in a public transport chain. The interconnectivity of the different modes also becomes important in order to realize a trip and determine the availability and convenience of public transport (Krygsman, 2004). Initiatives aimed at improving access and egress hold potential to significantly reduce public transport trip time and are inexpensive options compared to the expensive infrastructure and vehicle enhancement alternatives frequently considered.

Systematic modal integration has several benefits with regard to public transport which are crucially important for Dhaka. In cities such as Dhaka where non-motorized transport (walking, rickshaw, and bicycle) is currently a major component of the transport system, NMT is woven deeply into the urban fabric and can make an effective and functional contribution towards a sustainable transport system environmentally and economically. It has a high spatial penetration rate; virtually every location can be reached by walking, bicycles, rickshaws etc. and is a fast and efficient means of transport particularly at the short distances. It is necessary for Dhaka to devise a functional and indigenous integration mechanism that is contextual to the traffic characteristics and travel demand while meditating the majority of the transport users. Public transport and NMT can be complementary. Integration of NMT with public transport can cancel out the negatives of both systems and provide efficient and sustainable door-to-door service to the bus based commuters. An effective integration of NMT and public transport require the facilities that are given below:

Promoting Pedestrian Infrastructure: Pedestrian facilities refer to walkways, sidewalks, paths, and trails that are to be exclusively used by pedestrians only. Pedestrian movement should be carefully related to public transport network. Pedestrian ways should be attractive and convenient to use. Continuity should be maintained. Pedestrian facility should be provided to serve pedestrians better and encourage people to walk from choice rather than necessity. Desirable elements are:

- *Proper, unobstructed footpath*
- *Better and safe access to the stations*
- *Pedestrian Crossing Facilities*

Rickshaw improvements: As rickshaw is the dominant mode of travel in Dhaka's transport system, it will play a significant role in the effective modal integration in Dhaka. Increased consideration should be given to the planning and implementation of proper facilities for rickshaws.

- *Modernizing Rickshaw:* Existing model of rickshaw should be modified. More modernized, re-engineered, light weight or high technology vehicle should be considered.
- *Dedicated Space for Rickshaw:* Segregation of rickshaw to provide ample space and opportunity for increased speeds for rickshaw and other vehicles as well.
- *Rickshaw Stands:* Establishing formal rickshaw stations to integrate rickshaw with other modes and provide parking place for rickshaws.

- *Route Maps:* Developing rickshaw route maps and posted fare information to promote a systematic way of operation and a uniform fare rate to avoid the misunderstanding between passengers and rickshaw pullers.
- *Training:* Professionalized training to train the rickshaw driver to drive more safely.

Bicycle Facilities:

Bicycle as a mode of transportation should be provided with separate lanes and crossings, support local industries for manufacturing bicycle to make bicycle affordable. Increased consideration should be given to the planning and implementation of proper facilities for bicycle. Some of these options for promoting bicycle as a transport mode is given below:

- *Popularizing bicycle:* Most of the cities in the world placed emphasis on bicycles for integrating with public transport. Increasing use of bicycles could be possible through popularizing it among the users.
- *Developing a Community Bicycle/Pedestrian Network Plan:* A properly developed bicycle/pedestrian network is an integral part of efficient modal integration which can be supported through expanded facilities for bicyclists.
- *Bicycle scheme:* There can be two types of bicycle scheme (GTZ, Netherlands, 2010). These schemes are developed to make bicycle available to everyone even to those who do not have the capability to afford it. These two schemes are public bicycle scheme and bicycle rent scheme.

6. Conclusions:

Non-motorized transport (NMT) plays a dominant role as an affordable main mode of transport in Dhaka.NMT especially walking, bicycles and rickshaws have been very effective means of transport particularly at the short distances and could provide sustainable door-to-door service to the bus based commuters as well in Dhaka. It is likely that NMT will play a growing and greater role in urban transport system of Bangladesh, particularly in major cities. Promoting public transport networks and strengthening the ability of NMT to support those networks can address an efficient, environment friendly sustainable transport system. Integration of NMT with public transport can cancel out the negatives of both systems and provide efficient and sustainable door-to-door service to the bus based commuters. Considering social acceptability, economic response, fuel free eco-friendly characteristics, the role of non-motorized transport in sustaining the traffic and mobility needs of citizens is becoming increasingly important. The potentialities of non-motorized transport towards mitigating the complex transport problems in Dhaka are discussed in the paper. The context, expected benefits and systemic integration of non-motorized transport towards achieving the efficient public transport system in Dhaka are also discussed in the paper.

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SAFETY INVESTIGATION AND ASSESSMENT OF HIGH RISK ROAD SECTIONS IN BANGLADESH

Md. Mazharul Hoque[1*], Kamol Chandra Roy[1] & Md. Imran Shah[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka – 1000, Bangladesh.*
**Corresponding Author, <dirarc@gmail.com>*

ABSTRACT

Road safety has emerged as one of the most serious and growing global issues in recent times. Each year around 1.24 million people die in road traffic accident throughout the world. Around 91% of the world's fatalities on the roads occur in low-income and middle-income countries like Bangladesh where there is an alarming rise in road traffic accidents. Road accidents claim on average of 12,000 lives annually together with about 35,000 injuries in Bangladesh. About 70 percent of road accident fatalities occurred in rural areas including rural sections of national and regional highways. Accidents are highly clustered at some specified locations. Studies revealed that about 45 percent of national highway accidents occurred in only 2 to 5 percent length of its total network resulting in a total of 280 to 350 locations which are identified as hazardous locations. Pedestrians are the most vulnerable road user groups in Bangladesh. Pedestrians accounted for 49 percent of all reported fatalities. Around 52 percent pedestrian casualties occur while walking along the roadside in rural areas. Heavy vehicles, specially buses and trucks are mostly involved in fatal accidents. The road environmental factors are particularly prevalent with major roadway defects in design and layout, shoulders, road sides, bridge and its approaches, delineation devices and lack of access controls and others. According to the International Road Assessment Programme (iRAP) the highways in Bangladesh are mostly rated 2-stars or less (out of possible 5-stars) for vehicle occupants, pedestrians, motorcyclists and bicyclists indicating a relatively high level of risk of deaths and injuries.

The purpose of this paper is to make an investigation and assessment of selected high risk road sections in Bangladesh. Risk assessment has been made by combined utilization of iRAP star ratings and accident data. It was found that high incidence of accidents are closely related to the inbuilt roads and roadside hazards. Detailed field observations of the selected road sections were carried out to evaluate the extent of hazards and risk conditions associated with the road environment, road users and road vehicle. The results of the accident analysis and field observations clearly demonstrated significant road infrastructure deficiencies that require urgent corrective measures. The paper outlines the findings of the assessment and identifies site specific cost effective countermeasures to provide a safer road environment for both road users and vehicles.

Keywords: High Risk Roads, Safety Hazards, Risk Assessment, Safety Investigation.

1. Introduction

Road traffic accidents(RTA) are taking heavy toll of human lives and personal injuries worldwide by killing around 1.24 million people each year and injuring 50 million (WHO, 2013). Like all other low and middle income countries, Bangladesh is also experiencing a serious road safety crisis. Each year, there are at least 3,000 fatalities and 3,000 grievous and simple injuries from around 3,500 police reported accidents on Bangladesh roads (The Daily Star, 2012).The annual economic wastage occasioned by traffic accidents is

Figure 1: Road Accident Scene in Bangladesh

estimated to be in the order of 1.6 percent of the GDP (WHO, 2013). Almost 70 percent of road accident fatalities occurred in rural areas including rural sections of national and regional highways. Figure 1 shows a typical road accident scene in Bangladesh. Nearly 80 percent of road traffic fatalities are attributed to Vulnerable Road Users (VRUs) - pedestrians, bicyclists, motor cyclists and users of informal and unsafe motorized and non-motorized transport. Pedestrians accounted for 49 percent of all reported fatalities. Around 52 percent pedestrian casualties occur while walking along the roadside in rural areas (Shah, 2014). It is the poor that are most seriously affected with consequences of plunging poor households into acute poverty. Indeed the tragic premature, healthy and costly lives, permanent disabilities and property damages are exacerbating poverty reduction efforts particularly in rural areas. According to the International Road Assessment Programme (iRAP) the highways in Bangladesh are mostly rated 2-stars or less (out of possible 5-stars) for vehicle occupants, pedestrians, motorcyclists and bicyclists indicating a relatively high level of risk of deaths and injuries.


Figure 2: Typical high risk road section

The purpose of this paper is to make an investigation and assessment of selected high risk road sections in Bangladesh. The paper outlines the findings of the assessment and identifies site specific cost effective countermeasures to provide a safer road environment for both road users and vehicles.

2. The Context of High Risk Roads

High-risk rural roads are essentially lengths of road with a higher than ‘average’ crash risk, and by implication are roads where targeted safety improvements are most likely to reduce trauma (NZ Transport Agency, 2011). High risk road exhibits high rate of crash, fatality, injury than a normal rate specified by local or regional authority. About 70 percent of the fatalities occurred in the rural areas of Bangladesh and nearly 44 percent occurred on the national highways. Accidents are highly clustered at some specified locations. Studies revealed that about 45 percent of national highway accidents occurred in only 2 to 5 percent length of its total network resulting in a total of 280 to 350 locations identified as hazardous having section lengths varying between 0.1-0.5 km each location. (Hoque et.al. 2011).

The striking features of high risk roads can be summarized as follows (High-risk rural roads guide, 2011) :

- A rural road where the fatal and serious crash rate (personal risk) or crash density (collective risk) is classified as high or medium–high compared with other roads.
- A high or medium–high collective risk and/or high or medium–high personal risk (as defined by KiwiRAP risk maps)
- A rural road that has engineering features that have the potential for fatal or serious injury crashes to occur as determined by the KiwiRAP star rating or road protection score (RPS), eg a road with 1 or 2 stars or an RPS greater than 10.
- An equivalent process such as the Road Infrastructure Safety Assessment (RISA) where personal risk is greater than 2.5.

High risk roads are likely to show a large number of such hazardous locations or sections. It can also be assessed by International Road Assessment Program (iRAP). It thoroughly assesses the infrastructural deficiencies of the roads and produce star ratings, 5 star roads are the safest and 1 star roads are poorest. So the roads having 1 star or 2 star rating are high risk roads. Figure 2 shows a typical high risk road section in Bangladesh.

3. Severity of Road Accidents in Bangladesh

A detailed analysis of police reported accident data during the period 1998-2012 revealed the following striking characteristics of the accidents in high risk roads of Bangladesh.

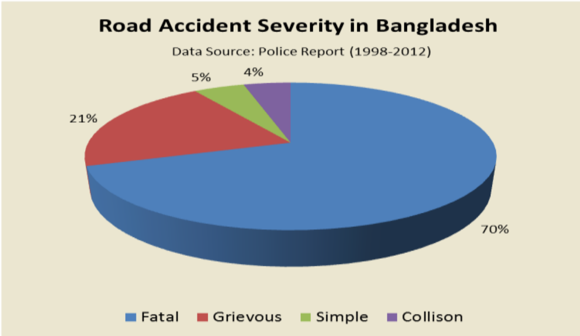

Figure 3: Road Accident Severity in Bangladesh

- There were 49847 no of accidents during the period of 1998-2012 among which 34886 were fatal accidents. Figure 3 shows the accident severity distribution in roads of Bangladesh. It is clear that fatal accidents are most common in the roads. Here significant numbers of grievous accidents are also occurred.

- About 75 percent of the fatalities and 63 percent of accidents occurred in rural areas. Rural accidents are more severe than the accidents in urban areas. Figure 4 shows that national highways hold the highest number of the total accident.

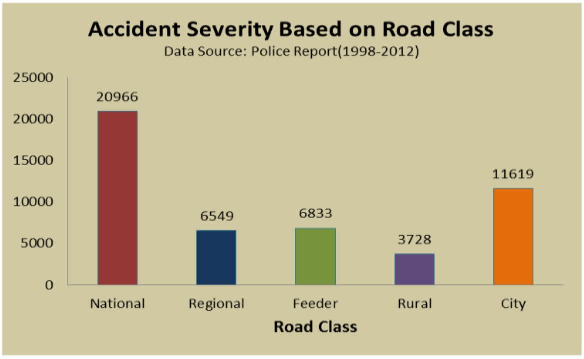

Figure 4: Accident Severity Based on Road Class

- Hit-pedestrian is the most frequent type of accident which is nearly 45 percent of total accidents and 54 percent of all fatal accidents. This accident type is followed by rear end (15%), head on (15%) and overturning accidents (8%). These four accident type groups accounted for about 83 percent of all accidents and 87 percent of all fatal accidents.

- Half of the total fatalities are pedestrians. The children under 16 years of age accounted for about 21 percent of total fatalities. About one-third of the total pedestrian fatalities are children. Around 76 percent of the road deaths are in the age groups of 5 - 45 years which have significant economic and societal impacts.

The principal contributing factors to accidents are adverse roadway and roadside environment, poor detailed design of junctions and road sections, excess speeding, overloading, dangerous overtaking, reckless driving, carelessness of road users, failure to obey mandatory traffic regulations, variety of vehicle characteristics and defects in vehicles and conflicting use of roads. The road environmental factors are particularly prevalent. Other issues of concern are: defective and road unworthy motor vehicles; driver incompetency; Inadequacy in police inspection and law enforcement; poor road user behavior and safety education; and institutional weakness. (Hoque et. al. 2010)

4. Accident Investigation and Safety Assessment of Highway Sections

Selected hazardous locations were studied by conducting on site visits and in-depth investigations. The specific road sections and road environmental hazards and the conflicts of roadside friction were also identified. Data from these site visits together with information from accident data and iRAP star rating were assembled for assessing the safety risk and needed corrective measures. In the study (Shah, 2014 and Roy, 2014) two hazardous locations of Dhaka-Sylhet Highway (N2) and Dhaka-

Aricha Highway (N5) were investigated to identify the risk issues that causing high number of accidents there.

4.1. Hazardous Road Locations: Case 1 on N5 Highway

4.1.1 Accident Characteristics

The Golora Bus Stand is located on the Dhaka-Aricha National Highway (N5) which is 57.6 kilometer far from Dhaka. This bus stand area is one of the most accident prone locations of the N-5 with a section length of 200 meters. In this road section road traffic collisions have been historically concentrated. A total of 18 accidents were reported in the accident data in the period of 1998-2010. But the actual number is higher than the reported accident. Pedestrians and motorcyclists are the most vulnerable groups. The predominant accident types are pedestrian hit and rear end location and are 33% and 22% respectively of the total reported accidents (Roy, 2014).

Among the accidents there were 13 fatal types and 7 non-fatal types. Analysis of pedestrian accidents based on pedestrian activity reveals that there were 5 accidents while pedestrians walking along the road edge which accounted for 5 pedestrian deaths. Again there were 2 accidents while pedestrians crossing the road which accounted for 4 pedestrian deaths. The data implies that pedestrians are at high risk in this road section. Table 1 shows the types of vehicle involved in accidents at this particular site. It shows that buses and trucks are the most predominant types of vehicles involved in the accidents, 32% and 23% respectively.

Table 4: Vehicles Involved in Road Traffic Accidents at Golora Bus Stand

Vehicle Type	Truck	Bus	Mini Bus	Car	Cycle	Motor Cycle	Other	Total
No. of Accident	5	7	3	1	2	1	3	22
Percent	23%	32%	13%	5%	10%	4%	13%	100%

4.1.2 Risk Assessment by iRAP and Accident Data

Figure 5 represents the iRAP star rating conditions and actual accident data by road kilometer post (Km 40.1- Km 47.3) for the period 1998-2010. The data includes all types of reported accidents with hit pedestrian as the predominant type.

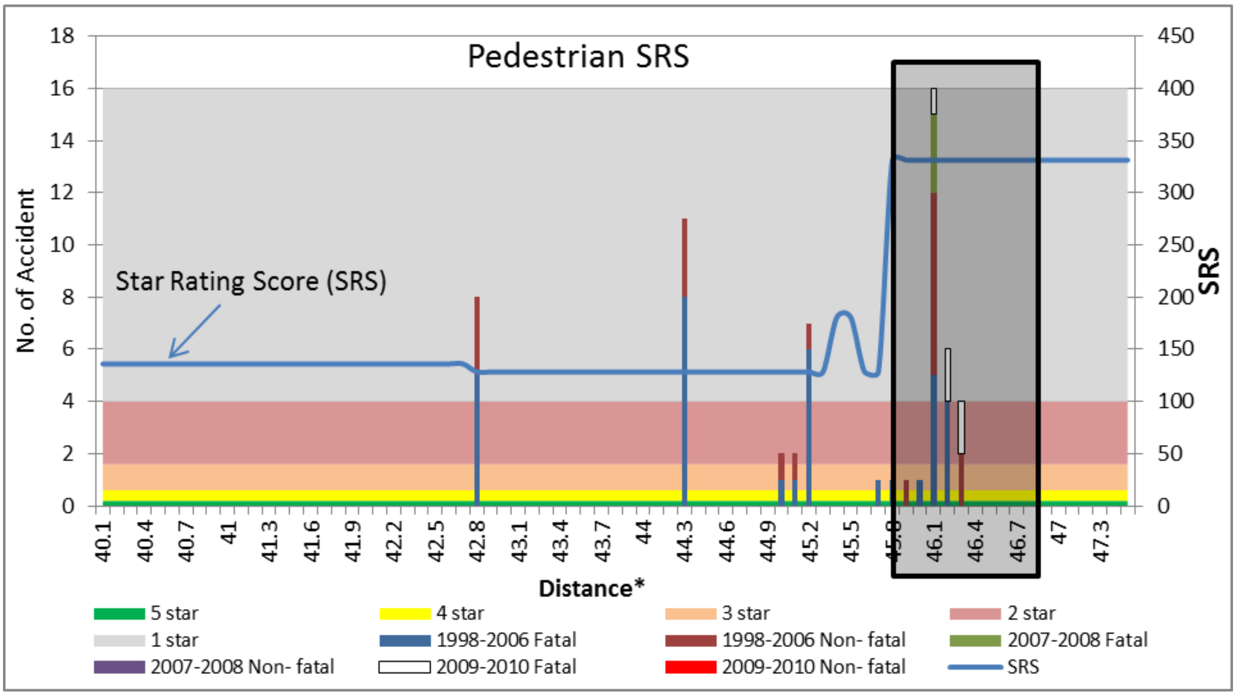

Figure 5: Combined Graph of iRAP Star Rating and Accident Data

*40.1 kilometre = 51.4 kilometre from Dhaka

The star ratings for pedestrians are presented in five categories (viz. 1star,2star,3star,4star,5star) as depicted in the figure. Clearly the entire section is characterized as 1 star for pedestrians, demonstrating very risk for pedestrians. At the location of Golora Bus Stand both accident data and star rating scores are extremely high compared to adjoining locations. From the figure it is also seen that Star Rating Score has suddenly increased at this location. The in-built risk at this location is mainly attributed to bus stops, haphazard roadside developments and road infrastructure hazards. The drivers and pedestrians are unable to cope up with this sudden change of infrastructure risk which leads this location to be a very high accident prone zone.

4.2. Hazardous Road Locations: Case 2 on N2 National Highway

4.2.1 Accident Characteristics

The other hazardous road section is located on Dhaka Sylhet Highway(N2) at kilometer post 92.0 to 94.0 km from Dhaka and is one of the most accident prone locations in N2 highway. The area is known as Khariala road which is at a distance of 2 km to 7 km from Ashuganj. There are both residential and commercial activities that produce traffic demand in this road section. According to the accident database it was found that a total of 36 reported fatal accidents occurred at this section in the period of 4 years (2007-2010). Among these hit pedestrian and rear end collision type accidents are predominant and are 41% and 28% respectively. Most of the accidents are fatal type and are causing serious injuries. The local people were interviewed to know about the accident type and frequency in this section. They confirmed that accidents occur frequently in this section and the predominant types are pedestrians. They also informed that former finance minister of Bangladesh was fatally injured in a serious car accident at this spot. The accident data and type clearly indicate immediate need of proper safety countermeasures to save valuable lives.

4.2.2 Risk Assessment by iRAP and Accident Data

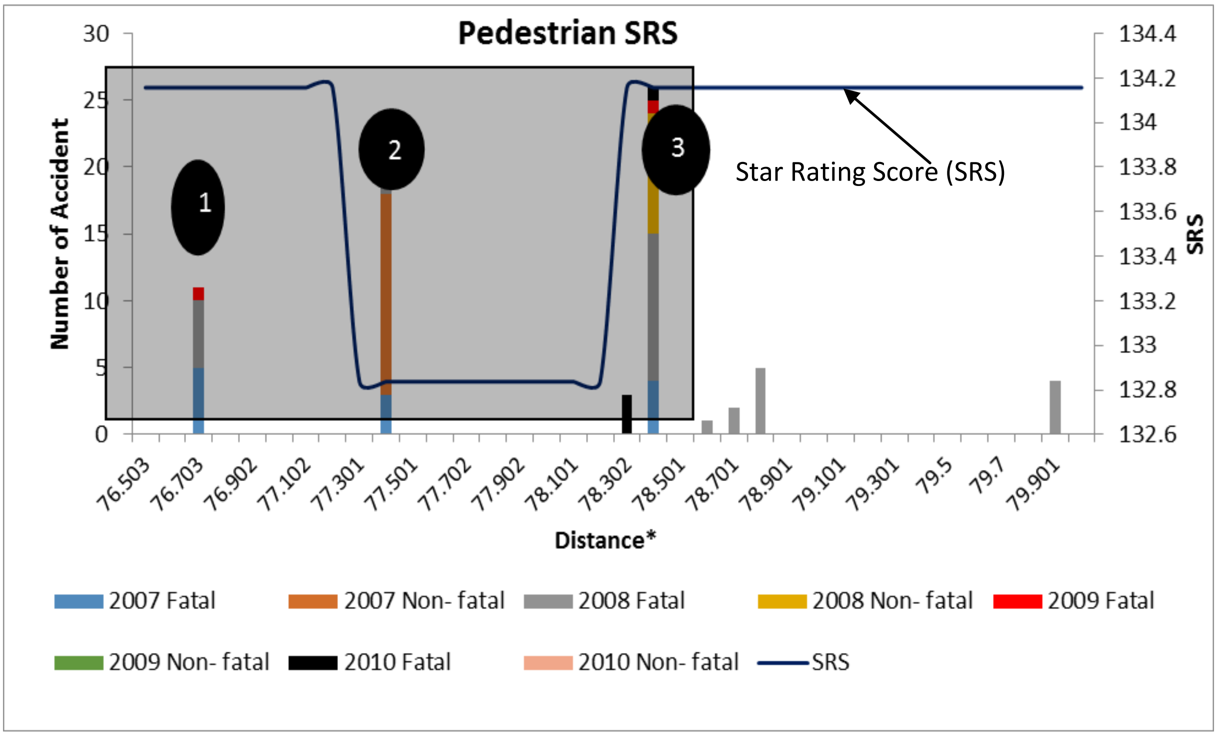

Figure 6: Combined Graph of iRAP Star Rating and Accident Data

*76.503 kilometer = 91.803 kilometer from Dhaka

Figure 6 represents the iRAP star rating condition and actual accident data by road kilometer post (Km 76.503- Km 79.901) for the period 2007-2010. Accident data includes all types of reported accidents with predominant hit pedestrian accidents. The star ratings represent risk for pedestrians. From the combined graph of accident and Star Rating Score it is shown that there are 3 black spots (highlighted as 1, 2, and 3) in this section where the road traffic accidents occurred very frequently. This section is rated as 1 Star for pedestrian (Star Rating Scores between 132 and 134) which indicates a very risky road for pedestrians. Similar characteristics of sudden change in Star Rating Score are observed where accident rate is higher (see highlighted portion 2, 3).

5. On Scene Investigation of the Study Areas

In this section a detailed field investigation of the study areas has been briefly described. The road sections were investigated during peak hour of the day when high volume of traffic passes through it. The investigation identified all the road infrastructure safety deficiencies and non-road related factors such as pedestrian behavior that are causing the roads to be high risk roads. Typical road traffic and road user behavioral hazards are depicted in this section.

Golora (Kamta) Bus Stand

An extensive field survey has been done to find out the on-site conditions of this hazardous road section. Some of the features that are found during our investigation are given below:

- Roadside of this road section is a busy place with a lot of shops, hotels, tea stalls, rickshaws and tempo workshops including high pedestrian activities.

- In our survey it was found that in most of the cases no footpath is available for the pedestrian to use but a shoulder of at most two feet wide that is encroached by parked vehicle, wooden stock, Moreover, frequent garbage disposal and water clogging in footpath lead to the deterioration of operational efficiency of pedestrian walkways. These features forced pedestrians to walk into the road which cause a serious safety hazard for pedestrian.

- At one entrance there is a moderate curve and from the other side there is a bridge which situated at upper level than the bus stops causing interference in the visibility. Only one speed hump at the curve side and thus traffics coming from the bridge side may be very dangerous to the other traffic and the pedestrians.

- Some faded zebra crossing were observed but most of the pedestrian don't seem to use it. No sign or signal for pedestrians was found. The lighting condition of the road was very poor as well.

- Though overtaking marking is provided but drivers very often ignore that. There was a speed limit of 60km/hr marked near the bus stand. The spot speed of some typical heavy vehicles like trucks and buses were measured and it was found that most the vehicles were running at a speed of 70-75km/hr.

Figure 7: Hazardous Road Condition Near Golora Bus Stand

Khariala Road Section near Ashuganj

In this selected road section of Dhaka -Sylhet Highway (N2) Dhaka the features that are found by field investigation have been given below:

- The pedestrian generating activities here are mainly rice mills, residential houses, paddy fields etc. The road section could be said a death trap for pedestrians. There were random movements of pedestrians although the volume is very low. No facilities for pedestrians were observed. Pedestrian uses shoulder of width 2 feet for walking purpose.

- The road section is pretty straight and quality of road surface is good. There were low number of cracks, potholes etc. on that section.

- Heavy vehicles often overtake with a high speed as much as 70 km/hr leading to the risk of fatal accidents.

- The mixing of NMT with the through traffic also causes a serious risk of accidents. It was alarming that children were playing near the road and fully exposed to through traffic which is a major risk.

Figure 8: Safety Hazards on N2 Highway

6. Measures for Site Specific Safety Improvements

In this section site specific countermeasures are recommended for the observed infrastructure deficiencies at the selected locations surveyed in N2 and N5 Highways to improve safety of the road users.

6.1 Countermeasure for the location observed at N2

Defects: No dedicated side walk or cross walk facilities for pedestrians and absence of regulatory/warning signs or signals, Illegal Parking and no delineation marking.

Preventive Measure:

- Adequate shoulders and their proper maintenance of shoulders will help pedestrians to use for safe walking along the roadway as there is no dedicated roadway space for the pedestrians or the NMVs.

- Clearly defined central line and edges line are of critical importance for the safe and efficient operation of traffic and are vital in enabling drivers to look at and path selection on road ways.

Figure 9: A Risky Section on N2 Highway

- Designated pedestrian crossing facilities and refuges and proper parking provision.

- Compliance of speed limit signs and their strict enforcement.

- Some sort of speed calming devices (rumble strips or jiggle bars) may be placed to control over-speeding vehicles.

Defects: No marking, poor pedestrian walking facility and Poor Shoulder

Preventive Measure:

- Shoulder widening, shoulder sealing and their proper maintenance to help pedestrians to use for safe walking along the roadway as there is no dedicated roadway space for the pedestrians or the NMVs.

- Clearly defined central line and edges line to enable drivers to path selection on road ways.

Figure 10: A Typical Section of N2 Highway

Defects: road side hazards, no centerline, edge line and lane use marking, risky roadside parking

Preventive measure:

- Removal of roadside hazards
- Proper parking provision
- Clearly defined central line and edges line
- Median barriers or separation, flexible posts, central hatching to prevent head on crash.
- Lane use marking and turning lane, remove unnecessary speed hump, proper signs and speed limit to prevent rear end crash.

6.2 Counter-measure for Golora Bus Stand at N5

Defects: Parking on pedestrian walk-way and very bad condition of road-side shoulder.

Figure 11: Roadway Environmental Hazards at Golora Bus Stand

Preventive measure: Proper parking location and adequate shoulder width for pedestrian sidewalk and its proper maintenance.

Defects: Heterogeneous traffic flow condition, very rare use of zebra- crossing, lack of proper signing and speed calming device.

Preventive measure:

- Designated pedestrian crossing facilities and refuges.
- Compliance of speed limit signs and their strict enforcement and speed calming device at proper location.

Figure 12: Pedestrian Safety Deficiencies on Highway Sections

7. Discussions and Conclusions

Road traffic accidents result from failures in the interaction of humans, vehicles and the road environment - the elements which produce the road traffic system. The combination of these various elements to produce road accidents means that road safety itself has to be tackled in a multi-functional manner in order to break the chains of events that lead to accidents and the eventual injuries of road users and property damages. There is now a growing assertion that the road environment is a most important determinant of accident frequency. Indeed it is the road and road component which the traffic engineer and the planner can most directly affect.

This paper has highlighted striking road accident characteristics and outlined typical road and road environmental safety hazards identified through selected case studies on national highways. International Road Assessment Programme (iRAP) methodology and the actual accident data were used to assess the severity and risk of the sections. iRAP assessment incorporated the geometric and other roadway condition attributes which revealed that the roadway sections are most vulnerable for the road users, predominantly for pedestrians due to lack of facilities and poor roadway conditions. The actual accident data also demonstrated the same level of severity scenario with large number of fatal and non-fatal accidents. In this study it was found that crash rates are higher where Star Rating Score suddenly increased than a section having uniform Star Rating Score. This indicates the importance of uniform Star Rating (at least 3 star) throughout the road. On-scene in-depth study of accident locations also revealed that factors relating to road environment are significant in road accidents and the road design features are indeed associated with particular accident types and hazards.

The problem characteristics dictate priorities to be placed on corrective measures relating to road design and environmental improvements to eliminate the hazards, obstacles and bottlenecks as well to channel orderly behavior of road users. Large reductions in accidents are potentially possible and indeed urgently needed include treatments and provision of roadway shoulders, self-enforcing speed reducing measures, special facilities for pedestrians and other NMV traffic, treatment of roadside hazards, special off-road bus stops facilities, installation of delineation devices and others. Given that pedestrian accidents and conflicts are the biggest problem in terms of vulnerability, it is very urgent to provide physically separated spaces in the form of segregated footways. Importantly, the principles of forgiving roadway design and clear recovery area should always and strictly be incorporated. It is time and very urgent to ensure that the road and its environment are put into safety through both reactive (identification and treatment of hazardous locations) and proactive approaches (introduction of road safety audit, inspection and assessment programs).

Indeed, the systematic accident investigation in this regard can lead to the implementation of most cost-effective countermeasures aimed at specific accident types and/or specific road user groups. Corrections and fixing of existing road safety hazards demonstrated in the paper are indeed an enormous challenge to road engineering professionals in Bangladesh. Shortage of safety expertise, research and requisite funding are now significant constraints. It is therefore a priority for Bangladesh to address such safety issues with significant institutional improvements through sharing knowledge and experience of effective road safety programs worldwide. Much more research and studies require for enhancing our understanding of relationships between road design features and accident characteristics.

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FEASIBILITY OF CONCRETE PAVEMENT IN BANGLADESH

Rahman. Md Towhid Ur[1*] & Zakaria. M [1]

[1]*Department of Civil Engineering, BUET, Dhaka, 1000, Bangladesh.*
**Corresponding Author, <ko2towhid@gmail.com>*

ABSTRACT

In Bangladesh most of the Roads constructed by RHD, LGED or DCC are Flexible Pavement. As the traffic demand rises with days the roadways get distressed and lose serviceability. To restore the condition- extensive maintenance and rehabilitation works are performed by the authority which are both very frequent and costly. In this paper it has been shown how PCC or Rigid Pavement can be a better alternative to Flexible Pavement for long term planning.

A study of Life Cycle Cost Analysis (LCCA) method is adopted to calculate the costing for 20yr analysis period for 1km length of both type of pavements considering all standard maintenance and rehabilitation works. The construction costs and Maintenance and Rehabilitation Costs (M&R) are calculated as per RHD Schedule of Rates (June, 2011). Discount Rate of 5% and predicted Monitory Inflation Rate have been considered for 20yr analysis period. Finally the total Life Cycle Cost has been calculated and compared for both Hot Mix Asphalt (HMA) and Portland Cement Concrete (PCC) types of Pavement. Finally a graphical representation of the Life Cycle Costs has been shown.

It is Flexible Pavements costs 3 times more to Rigid Pavement in 20yr analysis period for 1km length of each type of pavements and after 7 years of lifetime the costing for HMA Pavement exceeds the costing of Rigid Pavement.

Keywords: Concrete Pavement, HMA Pavement, Life Cycle Cost Analysis, Economic Study, Bangladesh

Introduction

As a general practice in Bangladesh, Flexible Pavements are constructed because of its low construction cost. But in some studies it has been observed that in the long run Rigid Pavement or PCC pavements are more cost effective than Flexible Asphalt Pavements. This study aims at assessing the feasibility of Concrete Pavement in Bangladesh.

Methodology

- *Data Collection*

First of all the location of the study was selected. Then the AADT data and geographic condition survey report were collected from Roads and Highways Department. The Roadway sections were then designed for two types of pavements.

The Schedule of Rates (2011) was used to calculate the construction cost for 1km segment of both Flexible and Rigid pavement. The standard maintenance and rehabilitation procedures were considered to be performed within the 20yr study period. The present rate of cost for these procedures were followed to calculate the yearly Maintenance & Rehabilitation costs.

- *Design and Analysis*

As per collected data and rate schedule the roadway sections were designed. The cost of construction was calculated for 1km long segment of both types of pavement. The present value of money as per Schedule of Rate by Roads and Highways Department (Ministry of Communication, Government of the People's Republic of Bangladesh, Edition: June 2011). Afterwards the M&R costing for corresponding type of pavement was calculated. For 20yr projection year to year values were calculated and cumulated to get the total investment through 20yrs time for Flexible and Rigid pavements.

As inflation rate, discount rate and interest rate are important parameter for Life Cycle Cost Analysis, so the effect of these parameters were demonstrated and analyzed for effectiveness of the study. The residual value, the total M&R over 20yr time and total life cycle cost was calculated.

Assumptions

Some assumptions were made during this study. They are stated below:

Economic Condition may change (Inflation Rate is included only).
Oil Price may change which affect all the petroleum based components specially asphalt.
Improved materials may be used which cost less and will change the design pattern of the pavements.
Improved maintenance and Rehabilitation techniques may be initialized to improve maintenance of the pavements
Changes in societal needs may occur.
New type of transport facility may be implemented.
Natural calamities and disasters may change the serviceability of the road.

Design Procedure

There are several design procedures for Flexible and Rigid pavement design. In this study the Flexible Pavement was designed as per Road and Highways Department Design Manual and Rigid Pavement was designed as per AASHTO Design Manual 1993.

- *Flexible Pavement*

Base Year Traffic Count

For both the geometric and pavement design of new roads, or the upgrading / widening of existing roads, traffic counts must be undertaken to establish the current Average Annual Daily Traffic (AADT) on the road. At least one whole day (24 hour) traffic count in both directions of flow should be undertaken on a typical weekday for sections of the road having more or less the same traffic volumes. For geometric design purposes the forecast traffic demand for the road in the design year should be estimated expressed in passenger car units (PCUs). In this cars, rickshaws and other light traffic may be ignored with only trucks and other commercial vehicles being considered.

Design Life and Traffic Growth Rate

For new roads and the full depth reconstruction of existing roads the following design standards are to be adopted:

Table 0-1: Design Life and Traffic Growth Rate

	Pavement Design Life	Traffic Growth Rate
National Highway	20yr	10% pa
Regional Highway	20yr	7% pa

Determining ESA over pavement life

A Standard Axle is taken to be 8,160 kg. Based on axle load studies previously undertaken in Bangladesh, the following equivalence factors have been determined:

Table 0-2: ESA Conversion Factors

Vehicle Category	Equivalence Factor
Large Truck	4.8
Medium Truck	4.62
Small Truck	1.0
Large Bus	1.0
Mini Bus	0.5

Cumulative ESA can be calculated from the equation stated below:

$$\text{Cumulative ESA} = \frac{(1+r)^n - 1}{r} \qquad \text{(Equation 0-1)}$$

Where,
r = annual traffic growth rate
n= design life in years
(Note: For national roads r=10% and n= 20yr, for regional roads r=7% and n=20yrs)

Determination of the thickness of the pavement layers

The estimated cumulative ESAs are then used to determine the various pavement layers from the following design chart:

Table 0-3: Pavement Layer thickness determining chart

mm	Surfacing (mm)		Roadbases (mm)* (Select one type)			Sub-bases (mm)** Subgrade CBR %		
Traffic ESA (mill)	Asphalt Wearing Course	Asphalt Base-Course	Cement-bound Granular	Granular Base Type I	Granular Base Type II	5	8 - 25	> 25
60 - 80	40	155	Refer to BRRL for design advice	N/A	N/A	300	150	0
40 - 60		140						
30 - 40		125						
25 - 30		110		250	300	250		
17 - 25		105						
15 - 17		95				200		
11 - 15		90						
9 - 11		80		200	250			
7 - 9		70						
6 - 7		65						
5 - 6		60						
4 - 5		55						
3 - 4		45		175	200	175		
< 3		35		150	175	150		

* CBR of granular base type I is min. 80%
* CBR of granular base type II is min. 50%
** CBR of sub-base material is 25%
N/A. = not applicable

Determination of improve subgrade thickness

For an improved subgrade in situ CBR should be 5%. If the CBR at subgrade is less than 5%, CBR is improved in the manner below:

Table 0-4: Determination of sub-grade thickness

CBR required	Compacted Thickness of additional layer to provide required CBR			
	CBR of Underlying layer			
	2%	3%	4%	5%
5%	250mm	150mm	100mm	-

○ *Rigid Pavement design*

There has been a widely used equation by AASHTO.

$$\log_{10}(W_{18}) = Z_R \times S_o + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624\times 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32p_t)\times \log_{10}\left[\frac{(S'_c)(C_d)(D^{0.75}-1.132)}{215.63(J)\left(D^{0.75}-\frac{18.42}{\left(E_c/k\right)^{0.25}}\right)}\right]$$

Equation 0-2: 1993 AASHTO Gide Basic Design Equation

Various variable values are input here and assumed slab thickness D is taken. By several trials the slab thickness is measured.

Maintenance and Rehabilitation works

- *Flexible Pavement*

Routine Maintenance works

There are several types of Flexible Pavement Routine maintenance works. Some of them are: Bituminous Surface Treatment (BST), Non-Structural Overlay, Slurry seals, Fog Seals, HMA patching, Crack seals etc are included.

Periodic Maintenance works

Periodic maintenance works include mainly overlay works. PCC and HMA overlay on HMA Pavement are the major periodic maintenance works done.

- *Rigid Pavement*

Routine maintenance works

Some routine maintenance are preformed to keep the running surface smooth and repair cracks to prevent moisture intrusion in subbase. Some are- Diamond Grinding, Joint Repair, Slab Stabilization, PCC joint and Crack sealing etc.

Periodic Maintenance works

Although rigid pavement demand a very few periodic maintenance works. Some may be named as- Stitching, Bonded Concrete overlay etc.

Life Cycle Cost Analysis (LCCA)

Life-cycle cost analysis (LCCA) is an essential component of modern roadway infrastructure design and system selection. LCCA embraces maintenance and rehabilitation costs, not just initial construction costs when evaluating pavement alternatives. The actual service life of initial pavement construction and rehabilitation treatment depends on a variety of factors including type and composition of the traffic, timeliness of maintenance treatments, and environmental factors such as temperature and precipitation. Future costs are discounted to today's BDT by using a discount rate derived from the combination of expected future inflation and interest rates. These inputs help determine the net present value of future costs. By comparing the total life-cycle cost of two or more pavement options, it is possible to make informed decisions on selecting the best pavement alternative for a particular application.

The required inputs include:
General inputs: 1.Analysis period. 2. Discount rate. 3. Site description/dimensions.
Specific inputs for all pavement types: 1. Unit costs. 2. Initial pavement layer thickness. 3. Maintenance and rehabilitation plan and quantities.

- *Calculation of Net Present Value*

The costs distributed over the pavement are typically translated into a Net Present Value (NPV). The NPV represents the total cost today that would be required, accounting for the interest and inflation expressed as the discount rate. The NPV of all activities are summed up to estimate the total maintenance and rehabilitation cost.

(Equation 0-3) $$\text{Total M\&R Cost} = \sum_i \frac{(M\&R\ Cost)}{(1+\text{Discount Rate})^{age}}$$

Where, Discount Rate (%) = Interest Rate (%) – Inflation Rate (%)

The discount rate used in the analysis represents the expected rate over the life of the project. In public sector projects, the discount rate depends primarily on the current economic environment, cost of borrowing, bank interest rates, market risk and opportunity costs to the public agency or government. In North America the discount rate is usually 3% to 5%. In this study the discount rate has been assumed 5%.

- *Pavement Residual Value*

The residual value is estimated by linear depreciation of the last capital activity cost. The prorated life method is used in the LCCA procedure to estimate the residual value. The recoverable cost is estimated by dividing the remaining life of the last rehabilitation treatment, by the expected life of the treatment.

Residual Value = Last Rehab Cost x [(Service Life - Activity Age) / Service Life]

To determine the salvage value, the last major rehabilitation activity is used.

- *Life Cycle Cost Calculation*

The total cost to construct and maintain each design option is the key focus of a LCCA. To accomplish this, the total sum of all costs, in equivalent NPV is required. The total cost is thus calculated as:

LCC = (Initial Cost + Total Discounted M&R Cost – Residual Value)

This value can then be used to benchmark other potential options and determine which is the most cost effective.

Design and Data Collection

In this study the author used RHD data and selected a 1Km section of Chittagong-Cox's Bazar- Teknaf Road. This is National Highway N1. From traffic data and roadway geometry the experimental sections of the roadway are selected. Rigid Pavement and Flexible Pavement sections are taken for cost analysis.

- *Design data*

Geometry data

From surveys the geometric data are collected by Roads and Highways Department (RHD) and the collected data are stated here:

Table 0-5: Geometry Data

Design Elements	Type/ Values
Road Standards	Type 2
Terrain	Plain

Speed data

The design speed was taken 80km/hr.

Horizontal control data

Minimum Curve Radius, Maximum Super Elevation and Maximum Transition data are collected for Horizontal control data.

Table 0-6: Horizontal Control Data

Design Elelment	Type/ Values
Minimum Curve Radius (m)	500
Maximum Super Elevation (m)	5%
Minimum Transition (m)	55

Vertical Control Data

Maximum Gradients, Minimum K value, Vertical Clearence Over Main Carrigeway and Vertical Underpass Values are tabulated below:

Table 0-7: Vertical Control Data

Design Element	Types/values
Maximum Gradients	4%
Maximum K value	35
Vertical Clearence over main carrigeway (m)	5.7
Vertical Clearence at Underpass (m)	4.5

Cross Section Element data

To measure the width, width at intersection, with at curvatures, lane width, horizontal clearance the tabulated data are collected:

Table 0-8: Cross Section Element Data

Design Elements	Type/Values
Carriageway Width (Each Direction) (m)	7.3

Traffic Lane Width (m)	3.65
Normal Cross Fall (%)	2.5%
Paved Shoulder Width (m)	2.0
Verge Width (m)	1.8
Verge Cross Fall (%)	5%
Inner Marginal Shoulder (m)	0.3
Width of Raised Central Median (m)	1.2
Embankment Slope	2H:1V

- *Traffic Data*

The Roadway designed here is RHD N1 roadway. This contains the part of Chittagong-Cox's Bazar- Teknaf Road. The general traffic data of N1 is stated below in a tabular form:

Table 0-9: Basic Data

Road No.	N1		
Road Name	Dhaka (Jatrabari)-Comilla (Mainamati)-Chittagong-Teknaf Road		
Class	National Highway	Starts at	Dhaka (Jatrabari)
Length	464.036 Km	Ends at	Teknaf

Table 0-10: Traffic Data for N1

Traffic (AADT)	10912 (Motorized: 10170, Non-Motorized: 742)
Average width	8.31 (m)
No. of bridges	331
No. of ferry ghats	0

- *Designed Pavement section*

Designed Flexible Pavement Section

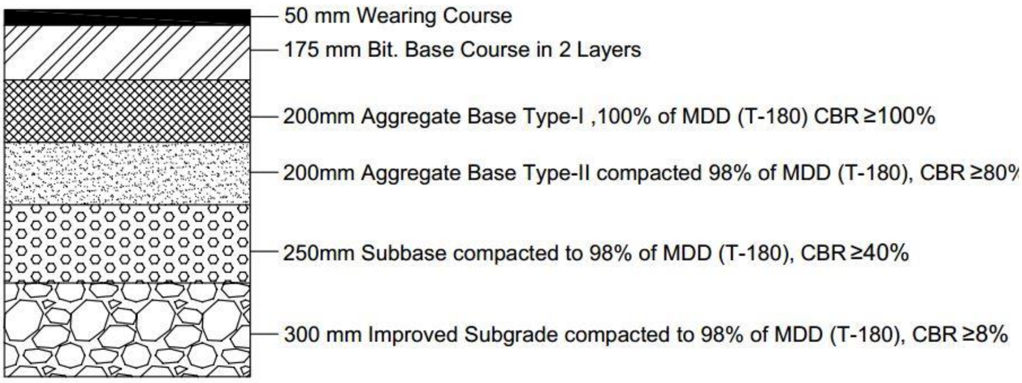

Figure 0-1: Designed Flexible Pavement Section

Designed Rigid Pavement Section

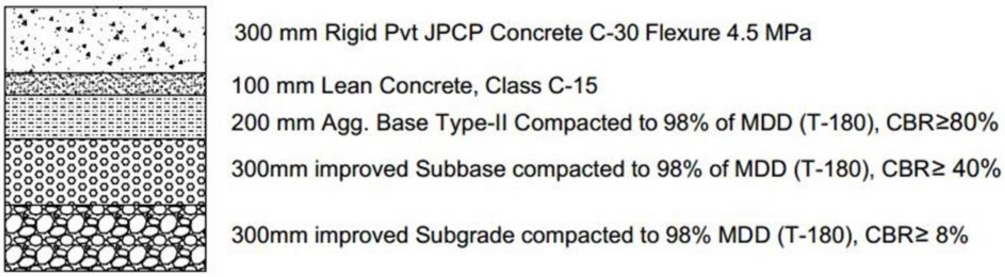

Figure 0-2: Designed Rigid Pavement Section

Data analysis

- *Construction Cost Calculation*

Flexible Pavement Construction Cost

The construction cost has been calculated including grade development and other costs. They are shown below:

Table 0-11: Construction Cost for 1km Flexible Pavement

SL	Layers	Thickness (mm)	Length (km)	Width (mm)	Unit cost(tk/m³)	Unit cost (tk/m²)	total cost (Tk)
1	50mm wearing course	50	1	27800		14267	396,623
2	Bit. Base course	175	1	27800		264	7,339
3	Aggregate base type-I	200	1	27800	5475		30,441,000
4	Aggregate base type-II	200	1	27800	4275		23,769,000
5	Subbase CBR≥40%	250	1	27800	3953		27,473,350
6	Improved Subgrade CBR≥8%	300	1	27800	861		475,380
Total Construction Cost							82,562,692

In words: Eight Cr. Twenty Five lac. Sixty Two thousand Six hundred and Ninety Two tk. only.

Rigid Pavement Construction Cost

Table 0-12: Construction Cost for1Km Rigid Pavement

SL	Layers	Thickness (mm)	Length (km)	Width (mm)	Unit cost(Tk/per	Total cost (Tk)

					cum)	
1	Concrete C-30	300	1	27800	12778	106,568,520
2	Lean Concrete	100	1	27800	12245	20,096,620
3	Base Type-II (CBR> 80%)	200	1	27800	4275	23,769,000
4	Subbase (CBR≥40%)	300	1	27800	3953	32,968,020
5	Subgrade (CBR≥8%)	300	1	27800	861	1,585
Total Cost						183,403,745

- *Maintenance Cost Calculation*

M&R Cost for Flexible Pavement

Maintenance cost per km of Flexible Pavement has been calculated below:

Type of work	Maintenance Work	Type of Unit	Cost per unit BDT	Total cost BDT
On Road	Pothole Repair	Per Cubic meter	12715.00	7,069,540
	Small Crack Repair	Per square meter	132.00	3,669,600
	Patch Repair by Bituminous Emulsion	Per square meter	60.00	1,668,000
	Total Cost for Routine Maintenance			12,407,140

Table 0-13: Maintenance Cost of per km Flexible Pavement

Rehabilitation Cost per km of Rigid Pavement has been calculated below:

Table 0-14: Rehabilitation Cost for per km of Flexible Pavement

Type	Maintenance Work	Unit	Cost/ unit	Total Cost
Resealing	Resealing	Per m²	86	2,390,800
	Single/Double Bituminous Surface Treatment	Per m²	300	8,340,000
Overlay	Spot Improvement	Per m²	300	8,340,000
	Bituminous Carpeting	Per m^3	13191	3,667,098
	Total Cost for Periodic Maintenance			22,737,898

M&R cost for Rigid Pavement

In general practice the usual cost per km of maintenance of rigid pavement is very insignificant. This is why in this study it has been assumed that the routine maintenance cost per km of Rigid pavement cost 1% of the construction cost and periodic maintenance cost 5% of the total cost of construction.

- *Life Cycle Cost Analysis*

This study focuses on the economic life cycle analysis of the two types of pavement. In this analysis the life cycle has been taken for 20years (From year 2014 to 2034).

There are some uncertainties that may occur during the analysis. Some are stated below:

- Economic Condition may change (Inflation Rate is included only).
- Oil Price may change which affect all the petroleum based components specially asphalt.
- Improved materials may be used which cost less and will change the design pattern of the pavements.
- Improved maintenance and Rehabilitation techniques may be initialized to improve maintenance of the pavements
- Changes in societal needs may occur.
- New type of transport facility may be implemented.
- Natural calamities and disasters may change the serviceability of the road.

During this study of Life Cycle analysis all these factors were excluded from the study. The assumption is “All factors remains the same”.

Monitory Inflation Rate

In LCCA monitory inflation rate is very important to be considered. In our country the unstable economic condition has been very inflation-prone. So before starting analysis this should be taken under consideration. For the prediction of future inflation rates the trend-line equation was developed to facilitate the future inflation pattern.

Plotting the previous year inflation rates of Bangladesh the below stated equation was developed:

$$y = -0.1487x + 304.45 \quad \text{(Equation 0-4: inflation rate)}$$

here, x= no of years starting from 2013
y= corresponding year inflation rate

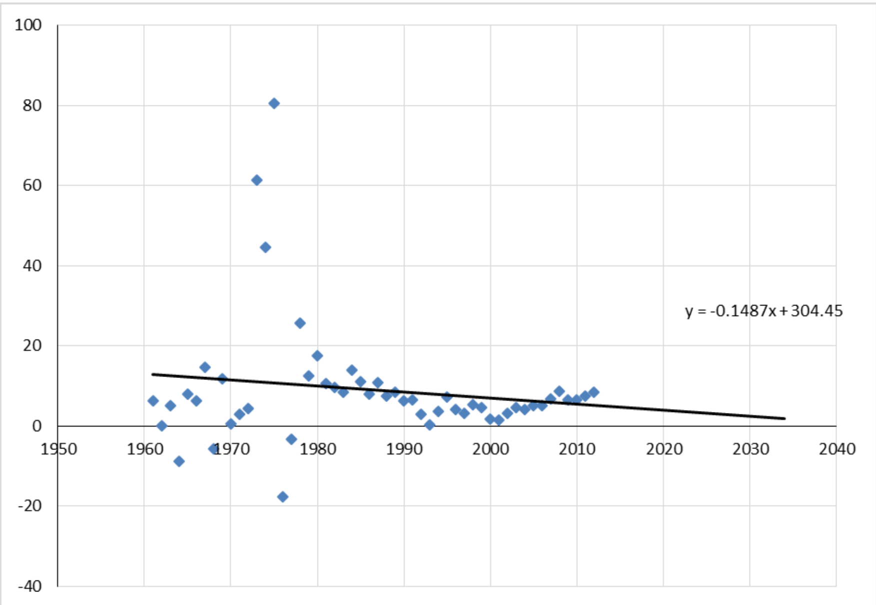

Figure 0-3: Inflation Rate Pattern: Year vs. Inflation Rate Graph

20yr Life Cycle Cost of 1Km Flexible Pavement

All the yearly and periodical M&R costs were summed up to the table below:

Table 0-15: 20yr Life Cycle Cost for 1Km Flexible Pavement

Head of cost	Amount BDT
Construction cost	8,25,62,692
20yr M&R Cost at NPV	61,68,32,227
Salvage Value	2,10,87,023
Life Cycle Cost for 1km Flexible Pavement	67,83,07,896

20yr Life Cycle Cost of 1Km Rigid Pavement

All the values were summed up below. The M&R Costs were taken to NPV taking corresponding inflation rate and at 5% discount rate.

Table 0-16: 20yr LCC of 1Km Rigid Pavement

Head of cost	Amount BDT
Construction cost	18,34,03,744
20yr M&R Cost at NPV	4,53,72,142
Salvage Value	66,02,534
Life Cycle Cost for 1km Flexible Pavement	22,21,73,352

- *Comparison between the two option based on the LCCA:*

Here is the sum up of the pervious calculations:

Table 0-17: Comparison between the costs of Life Cycle of Flexible and Rigid Pavement

Head	Flexible Pavement	Rigid Pavement
Construction cost	8,25,62,692	18,34,03,744
20yr M&R Cost at NPV	61,68,32,227	4,53,72,142
Salvage Value	2,10,87,023	66,02,534
Life Cycle Cost	67,83,07,896	22,21,73,352
Difference	45,61,34,544	

From the previous Calculations it is evident that the Life cycle cost of Flexible pavement is higher than the rigid pavement. The Difference in Life Cycle Cost between 1km Flexible and Rigid Pavement for 20yrs analysis period is = 45,61,34,544 BDT. In words: Forty Five Cr. Sixty one lac. Thirty Four thousand Five hundred and Forty Four tk. only. It is evident that 1km of Flexible Pavement costs 3times to 1km Rigid Pavement in 20yr Life Cycle.

- *Graphical representation of the Life Cycle Cost*

The yearly costs were plotted years along X-axis and costs along Y-axis.

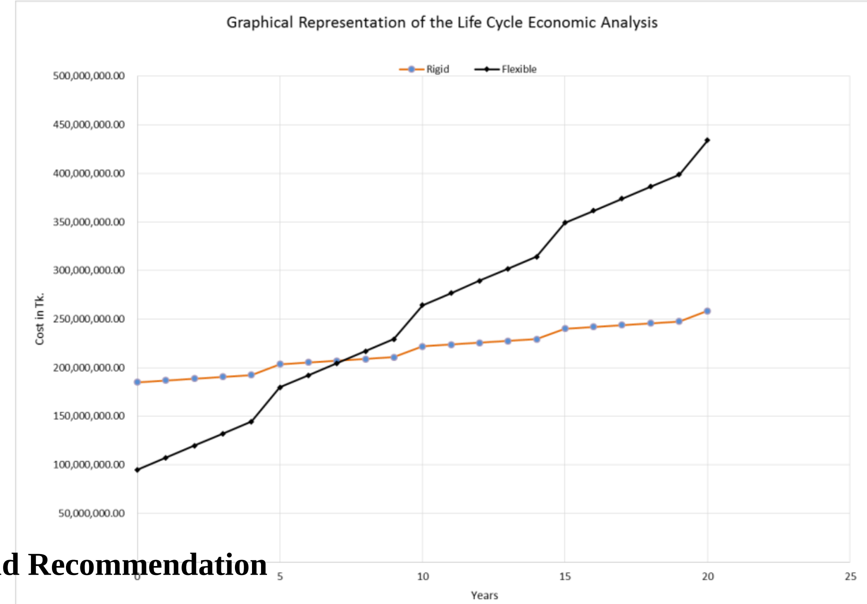

Figure 0-4: Graphical Representation of the Life Cycle Costs of 1km Flexible Pavement and 1km Rigid Pavement for 20yrs Analysis Period at NPV @5% Discount Rate

Findings and Recommendation

Based on the assumption and analysis made, findings and recommendations are as follows:

- *Findings*

The below stated findings were noted after the completion of the study:

- Major portion of construction cost of a Flexible Pavement is covered by Development of Subbase Course and Aggregate Base Development phases. The proportion is 33% and 37% of the total construction cost respectively.
- For Rigid Pavement the major cost is involved with PCC Slab construction. It contains 58% of the initial construction cost.
- Development of Improved and graded Base costs more for both type of pavement.
- Cost for Routine maintenance of flexible pavement equals to the amount of 15% of the initial investment.
- Cost for Periodic Maintenance of flexible pavement equals to the amount of 28% of the initial investment.
- In both types of M&R works the costing of flexible pavement is more than that of Rigid Pavement.
- NPV of the 20yr M&R cost of 1km Flexible Pavement at 5% discount rate equals 67,83,07,896.00 BDT.
- NPV of the 20yr M&R cost of 1km Rigid Pavement at 5% Discount Rate equals 22,21,73,352.00 BDT.
- The Residual Value or Salvage Value of the road types have a sudden hike immediate after Rehabilitation Work has been done.
- The Residual Value of Rigid Pavement is 1,44,84,489.00 BDT more than that of Flexible Pavement at the end of 20yr analysis period.

- Total Life cycle cost for 1 km long flexible pavement is triple to the LCC of Rigid Pavements.
- Total savings in 20yr life cycle cost for 1km Rigid pavement is 200% of the cost of 1km Flexible pavement 20yr life cycle cost.
- The value of savings is 45,61,34,544.00 BDT. For 1km Rigid Pavement.
- From the both trend line of costing when they are plotted together the author found that the both lines intersect at 7 years' time.

In summary this study shows that Rigid Pavement is more feasible than Flexible Pavement in long service life.

o *Recommendation*

This study has some recommendation for planners and designers. The economic analysis by LCCA method recommends:

- The future researchers may have a brief guideline to their research
- The policy makers should think about this better opportunity
- In this study there are many graphical relationships between considered variables. This graphs can be used for further reference
- The data used here are up to date
- As there have not been any research work on this topic in Bangladesh perspective, this study has a versatile scope to develop new ideas
- As the currency was used Local Bangladeshi Taka (BDT.) the calculations and costing are easily understandable to anyone related to transportation sector
- In this study there is a complete process of road section design to feasibility analysis. For further research this study can be treated as a very good reference.

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APPENDIX A

AASHTO Rigid Pavement Design Inputs:

Inputs:

- Loading: The predicted loading is simply the predicted number of 80 kN (18,000 lb.) ESALs that the pavement will experience over its design lifetime.
- Reliability. The reliability of the pavement design-performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period. In other words, there must be some assurance that a pavement will perform as intended given variability in such things as construction, environment and The 1993 AASHTO Guide equation requires a number of inputs related to loads, pavement structure and subgrade support. These inputs are:
- The predicted materials. The ZR and So variables account for reliability.
- PCC elastic modulus: If no value is known, the PCC elastic modulus (Ec) can be estimated from relationships such as the following:

$E_{c=} 57000\sqrt{f_c'}$

Where, E_c= Elastic Modulus of PCC
f_c'= Compressive Strength of PCC

- PCC modulus of rupture: The modulus of rupture (S_c') is typically obtained from a flexural strength test. Slab depth. The pavement structure is best characterized by slab depth (D).
- The number of ESALs a rigid pavement can carry over its lifetime is very sensitive to slab depth.
- Drainage coefficient: Rigid pavement is assigned a drainage coefficient (Cd) that represents the relative loss of strength due to its drainage characteristics and the total time it is exposed to near-saturation moisture conditions.
- Serviceable life: The difference in present serviceability index (PSI) between construction and end-of-life is the serviceability life.
- Load transfer coefficient (J Factor): This accounts for load transfer efficiency. Essentially, the lower the J Factor the better the load transfer. The J Factor for the AASHO Road Test was estimated to be 3.2
- Modulus of subgrade reaction: The modulus of subgrade reaction (k) is used to estimate the "support" of the PCC slab by the layers below. Usually, an effective k (k_{eff}) is calculated which reflects base, subbase and subgrade contributions as well as the loss of support that occurs over time due to erosion and stripping of the base, subbase and subgrade.

APPENDIX B

In this calculation all the cost per unit value was collected from RHD Schedule of Rates (June 2011). The components along with their Rates per unit (Volume or Area) are enlisted below:

1. 50mm Wearing course:
   Unit cost = 14267 BDT/sqm.
   Length of segment = 1km = 1000m
   Width (avg.) of the segment = 27800 mm
   Thickness of the Wearing Course = 50 mm
   Total Cost for 50mm Wearing Course = 396,623.00 (BDT)

2. 175mm Bituminous Base Course:
   Unit Cost = 264 BDT/sqm
   Length of segment = 1km = 1000m
   Width (avg.) of the segment = 27800 mm
   Thickness of the Bituminous Base Course = 175 mm
   Total Cost for 175mm Bituminous Base Course =7339.00 (BDT)

3. 200mm Aggregate Base Type-I:
   Unit Cost = 5475 BDT/cum
   Length of segment = 1km = 1000m
   Width (avg.) of the segment = 27800 mm

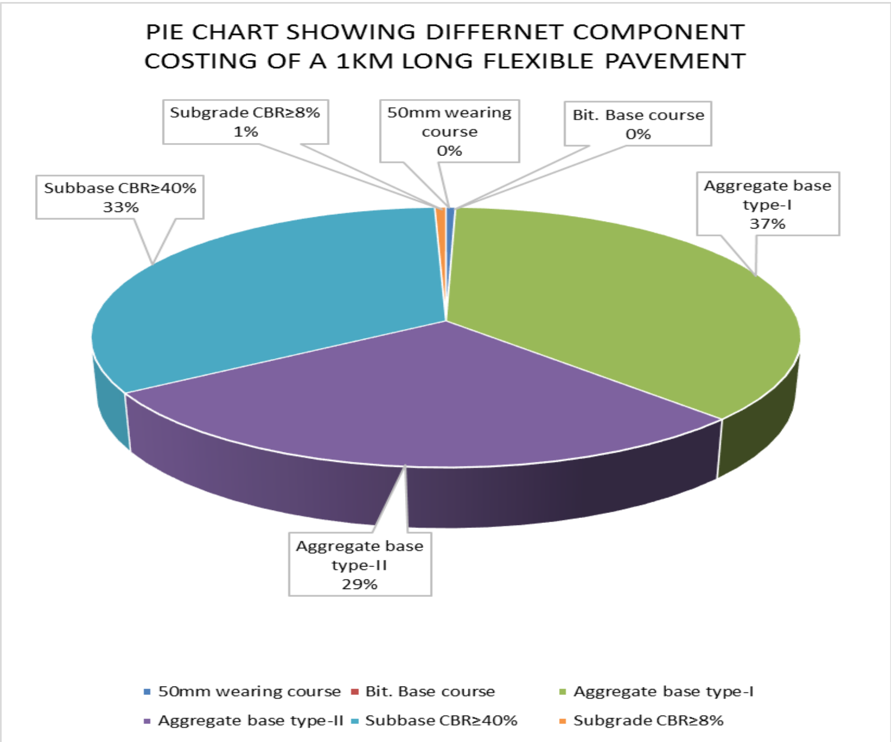
Thickness of the Aggregate Base = 200 mm
Total Cost for 200mm Aggregate Base Type-I = 30,441,000.00 (BDT)

4. 200mm Aggregate Base Type-II:
Unit Cost = 4275 BDT/cum
Length of segment = 1km = 1000m
Width (avg.) of the segment = 27800 mm
Thickness of the Aggregate Base Type-II = 200 mm
Total Cost for 200mm Aggregate Base Type-II = 23,769,000.00 (BDT)

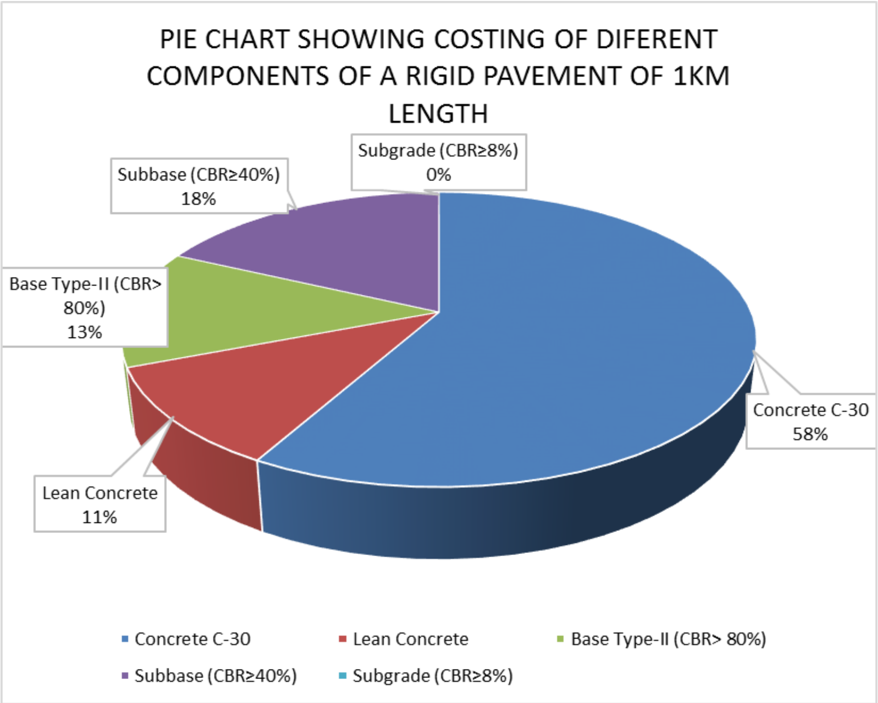
5. 250mm Subbase (CBR≥40%):
Unit Cost = 3953 BDT/cum
Length of segment = 1km = 1000m
Width (avg.) of the segment = 27800 mm
Thickness of the 250mm Subbase (CBR≥40%) = 250 mm
Total Cost for 250mm Subbase (CBR≥40%) = 27,473,350.00 (BDT)

6. 300mm Improved Subgrade (CBR≥8%):
Unit Cost = 57 BDT/sqm
Width (avg.) of the segment = 27800 mm
Thickness of the Improved Subgrade (CBR≥8%): = 300 mm
Total Cost for 300mm Improved Subgrade (CBR≥8%) = 475,380.00 (BDT)

APPENDIX C



APPENDIX D



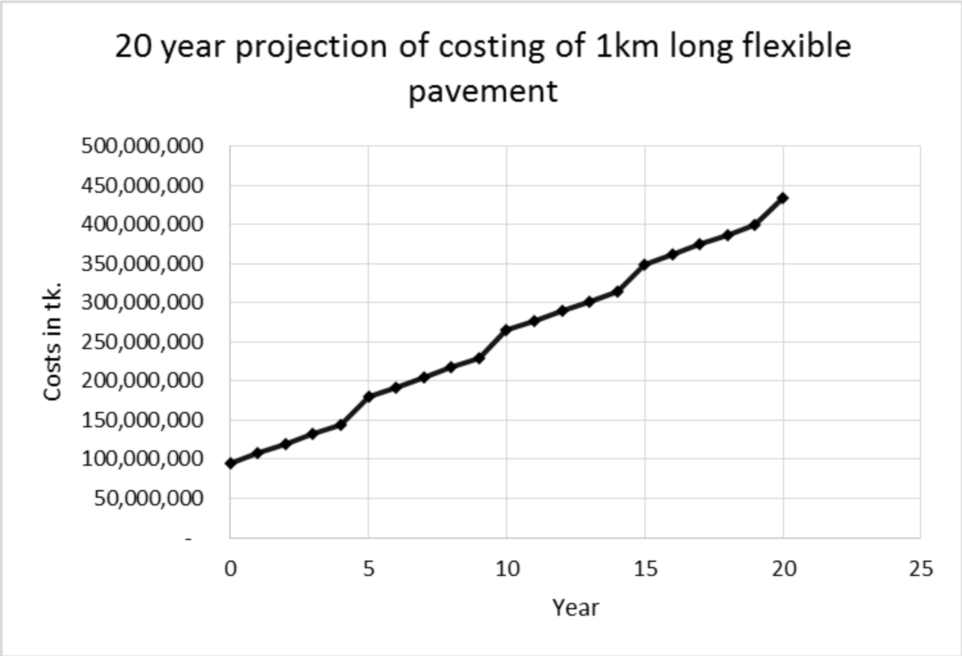
APPENDIX E

Table 4-5: Yearly Inflation Rates at Bangladesh along with trend line Eqn values

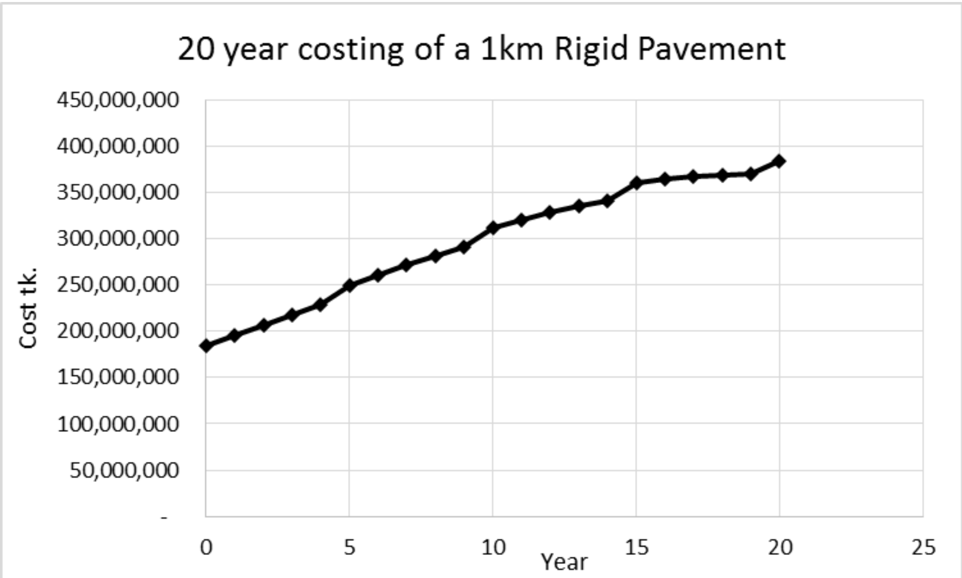
Year	Value from Statistics	Value from Equation
1961	6.257482	12.8493
1962	0.022224	12.7006
1963	5.164059	12.5519
1964	-8.74322	12.4032
1965	7.931705	12.2545
1966	6.296189	12.1058
1967	14.79196	11.9571
1968	-5.76958	11.8084
1969	11.82772	11.6597
1970	0.510309	11.511
1971	2.963255	11.3623
1972	4.40202	11.2136
1973	61.40578	11.0649
1974	44.54272	10.9162
1975	80.56976	10.7675
1976	-17.6304	10.6188
1977	-3.21016	10.4701
1978	25.61889	10.3214
1979	12.56451	10.1727
1980	17.55507	10.024
1981	10.52793	9.8753
1982	9.687499	9.7266
1983	8.515266	9.5779
1984	14.04688	9.4292

Year	Value from statistics	Value from Equation
1988	7.60067	8.8344
1989	8.500223	8.6857
1990	6.335597	8.537
1991	6.596235	8.3883
1992	2.97637	8.2396
1993	0.28697	8.0909
1994	3.771827	7.9422
1995	7.345332	7.7935
1996	4.234504	7.6448
1997	3.090097	7.4961
1998	5.274366	7.3474
1999	4.655731	7.1987
2000	1.859661	7.05
2001	1.585395	6.9013
2002	3.195375	6.7526
2003	4.52763	6.6039
2004	4.240429	6.4552
2005	5.074715	6.3065
2006	5.172374	6.1578
2007	6.78645	6.0091
2008	8.789101	5.8604
2009	6.520954	5.7117

APPENDIX F



APPENDIX G



ID: WRE 001

STUDY ON SOCIAL AND ENVIRONMENTAL IMPACT AND ADAPTATION TECHNIQUES TO FLOOD IN PATHARGHATA UPAZILLA, BARGUNA, BANGLADESH

Md. Hamidul Islam[1]*, Md. Mahmudul Hasan[2], Ruma Khanam[2], Md. Saiful Isalm[2] & Md. Shafiqul Islam[2]

[1] *Department of Building Engineering & Construction Management, KUET, Khulna-9203, Bangladesh.*
[2] *Faculty of Disaster Management, Patuakhali Science and Technology University, Dumki, Patuakhali-8602, Bangladesh.*
**Corresponding Author, <hamidcekuet@gmail.com>*

ABSTRACT

Bangladesh is facing various kinds of disasters every year due to its geographic location. Flood is one of them which mostly affect the southern part of our country.Patharghata is located in Barguna district of Barisal division in Bangladesh. The geographical coordinates of this area is at 23°38’35’’N 90°13’01’’E. Southern part of the study area is most vulnerable (60%) to flood because most of the river and the Bay of Bengal is on this side. Eastern and western part of the study area is medium vulnerable (25%) and northern part of the study area less vulnerable to flood risk because this part is the highest area of Patharghata(Tamima, 2009). Our study focuses on finding out the social and environmental impacts, adaptation techniques to minimize these impacts and to suggest management strategy to flood in the study area. From this study the major social impacts of flood are loss of agricultural crops, scarcity of pure drinking water, communication system goes ran out, breakout various water borne diseases, loss of economy, loss of fish hatchery. Major environmental impacts of flood are loss of biodiversity, air quality decreases, decrease water quality, flood plains are damaged. To minimize flood impacts structural adaptation measures are dam, sluice gate, embankment, sea wall, flood wall and non-structural adaptation measures are early warning system, well prepared response strategies, accuracy of forecasts.For better performance to control flood damage significant factors to be maintained such as greater investment in disaster risk reduction, warning methods and dissemination should be expended. And also reducing greenhouse gas emission, global warming problems should be under control, cultivating flood adaptive variety. If the problems are solved which are identified in the study area the flood impacts will be reduced at 80%.

Keywords: Impacts, Structural and non-structural adaptation, Advantage and disadvantage, Relief, Rehabilitation.

INTRODUCTION

Bangladesh is a deltaic country located at the lower part of the basins of the three mighty rivers: The Ganges, the Brahmaputra and the Meghna delta (Paul and Baky, 2010). The people are frequently threatened by natural and human-made catastrophes such as floods, cyclones, tornadoes (Anon, 2006) (Recovery et al., 2013). Flood is the major problem among them. Flooding can occur due to river overflow, surface runoff, heavy rains, when ocean waves come onshore, when snow melts too fast, or when dams or levees break (Anon, 2004). There are two types of floods which occur in Bangladesh: annual floods (locally is called barsha) that inundate up to 20% of the land area and low frequency floods of high magnitude that inundate more than 35% of the area (locally is called bonna). Also, different factors affects flood like huge population with its geometric growth day by day, process of urbanization, industrialization (Tamima, 2009). Flood, storm surge, river bank erosion, salinity intrusion and cyclone are frequently occurring in my study area. Flood and storm surge is the major problem at Patharghata, Barguna district. The objectives of this study are to identify the social and environmental impacts of flood, determine the adaptation techniques (structural and non-structural) to minimize flood impacts and suggest management strategy to flood.

Fig. 1: Floods in Bangladesh.

Fig. 2: Destruction of house due to flood.

Study Area:

Patharghata is an upazilla of Barguna district. It is located at 23°38’35’’N 90°13’01’’E. This area is mostly vulnerable to several floods in the rainy season. The extreme water of flood from The River Paira inundates the agricultural land, roads, houses and the entire locality. Southern part of the study area is most vulnerable (60%) to flood risk. Eastern and western part of the study area is medium vulnerable (25%) and northern part of the study area is less vulnerable to flood risk.

Fig. 2: Encircle portion is the study area (Patharghata upazilla).

MATERIALS AND METHODS

Data Collection:

Primary data collection:

Primary date collected by two methods which are observation method and questionnaire method. The observation methods are participated structured and controlled. The interview method of collecting data involves presentation of oral-verbal stimulating and reply in terms of oral-verbal personal response.

Secondary data collection:

This study concerned relevant secondary sources of data and information such as statistical records, survey records, written documents, encyclopaedia, Google earth, Wikipedia.

RESULTS AND DISCUSSIONS

Social and Environmental Impacts of Floods in Patharghata:

Social impact can be defined as the net effect of an activity on a community and well-being of individuals and family. Environmental impact can be defined as the possible adverse effects are caused by a development, industrial or infrastructural project or by the release of substances in the environment. The vulnerability assessment of Patharghata upazilla as shown in figure 3.

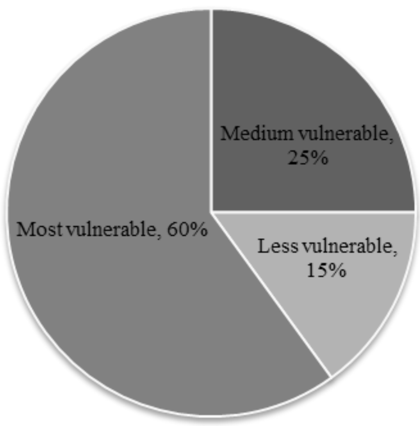


Fig. 3: Vulnerability of the study area due to floods (Source: Patharghata upazila office, 2014).

Social impacts:

Positive impacts:

- Increase soil fertility
- Increase fish production

Negative impacts:

- Scarcity of pure drinking water
- Outbreak various water born disease such as diarrhea, cholera, dysentery
- Communication system goes ran out
- Loss of economy
- Face water logging problems
- Students of this area cannot go to school due to flood

Environmental impacts:

- Loss of biodiversity
- Air quality decreases
- Decrease water quality
- Flood plains are damaged which are the areas of highest productivity

On farm impacts:

- Loss of agricultural crops, loss of top soil, loss of soil nutrients, soil erosion, livestock
- Loss of fish hatchery farmers
- Cultivable land degraded

Off farm impacts:

- Power supply failure
- Damage transportation systems

Study represent that 66% people suffer from social and environmental impacts. This is shown in Table 1.

Table 1: Percentage of people suffers from social and environmental impacts.

Variables	Types	Percentage of impact for each types	Total percentage of impact
Social impacts	Scarcity of pure drinking water	10%	35%
	Communication system goes ran out	10%	
	Outbreak various water born disease	5%	
	Loss of economy	10%	
Environmental impacts	Loss of biodiversity	5%	12%
	Air quality decreases	2%	
	Decrease water quality	5%	
On farm impacts	Loss of agricultural crops, loss of soil nutrients, soil erosion, livestock	10%	12%
	Loss of fish hatchery farmers	2%	

Off farm impacts	Power supply failure	2%	7%
	Damage transportation systems	5%	
Total percentage of impact			66%

Adaptation Measures:

Emergency responses to flood:

Immediate responses made by the government and international aids are given below:

- Food distribution
- Provide medical service
- Providing clean waters
- Providing cloths
- Vegetable seed distributions

Structural measures:

- Dams and Sluice gate
- Embankments, flood walls, sea walls
- Natural detention basin
- Channel and drainage improvement
- Diversion of flood waters
- Storm water management
- Coastline protection
- Shelterbelt Plantation and coast Afforestation

Non-structural measures:

Reducing the susceptibility to flood damages through:

- Integrated floodplain management
- Flood proofing and shelter
- Real time flood forecasting and warning

Minimizing the flood loss through:

- Disaster relief, flood fighting including public health measures
- Floodplain zoning and risk mapping
- Urban flood management
- Flood management modeling
- Flood emergence response
- Flood damage assessment
- Awareness building through public media
- Development of regional cooperation

Post rehabilitation and relief:

Relief and rehabilitation of the flood victims is done for making the community prepared on flood disaster and initiate income generating activity in the flood affected areas. There is need to find out ways to make sustainable life in the flood prone areas such as:

- Providing food and safe drinking water
- Construction of damaged infrastructure
- Development of vulnerable groups

The risk reduction options for better management:

- Greater investment in disaster risk reduction
- Warning methods and dissemination should be expended
- Global warming problems being under control
- Cultivate flood adaptable variety
- Proper use of ground water
- Manage proper drainage systems

The other regulation would include:

- Not permitting unrestricted new development in the hazard prone areas
- Anchoring and flood proofing structures to be built in known flood prone areas

Different structure that is available in the survey area:

Fig. 3(a): Sluice gate

Fig. 3(b): Culvert

Fig. 3(c): Embankment

Fig. 3(d): Dam

Fig. 3: Different structure that is available in the survey area.

Study recommended that if all structural and non-structural adaptation techniques are performs accurately then the flood impacts will be reduced at 80%. The impacts reduction strategy as shown in Table 2.

Table 2: Impact reduction strategy by structural and non-structural measures.

Variables	Types	Percentage of impact reduction for each types	Total percentage of impact reduction
Structural	Dam	10%	50%
	Sluice gate	5%	
	Embankment	15%	
	Sea wall	10%	
	Flood wall	10%	
Non-Structural	Early warning system	15%	30%
	Well Prepared response strategies	5%	
	Accuracy of forecasts	10%	
Total percentage of impact reduction			80%

CONCLUSION

Flood is a common phenomenon at Patharghata upazilla in Barguna district. It can damage both economically and environmentally. Flood has major impacts on human lives, health, livestock, water, agriculture and biodiversity (Paul and Baky, 2010). From this study the major social impacts of flood are loss of agricultural crops, scarcity of pure drinking water, communication system goes ran out, breakout various water borne diseases, loss of economy, loss of fish hatchery. Major environmental impacts of flood are loss of biodiversity, air quality decreases, decrease water quality, flood plains are damage. To control the flood the structural adaptation measures are dam, sluice gate, embankment, sea wall, flood wall and non-structural adaptation measures are early warning system, well prepared response strategies, accuracy of forecasts. Study represents some limitation such as lack of awareness, inaccuracy in forecasting system and repair activity not done properly. To reduce impacts of flood it is recommended that the risk reduction options such as greater investment in disaster risk reduction, warning methods and dissemination should be expended. And also reducing greenhouse gas emission, global warming problems should be under control, cultivating flood adaptive variety.

ACKNOWLEDGMENTS

First of all authors would like to express the deepest satisfaction to the almighty Allah for giving the opportunity to carry out the study. Also cordial thanks faculty members of faculty of disaster management, PSTU and Patharghata upazilla parishad for their valuable information.

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ID: WRE 002

HYDROLOGICAL MODELING FOR THE SEMI UNGAUGED BRAHMAPUTRA RIVER BASIN USING SWAT MODEL

S. Paul[1*], A.S. Islam[1], M. A. Hasan[1], G. M.T. Islam[1] & S. K. Bala[1]

[1] *Institute of Water and Flood Management, BUET, Dhaka.*
**Corresponding Author, <paulwre710@gmail.com >*

Bangladesh is a delta formed as a result of sedimentation by the Ganges, the Brahmaputra and the Meghna rivers and its distributaries and tributaries. Among these three major rivers, the Brahmaputra carries the highest annual flow to Bangladesh from China, India and Bhutan (about 67%). In order to assess the water availability and predict floods in Bangladesh, it is necessary to establish a hydrologic model over the Brahmaputra basin. In this study, a physical based model SWAT has been set up over the Brahmaputra basin. The model has been calibrated and validated using the observed daily flow data from 1998 to 2007. Rainfall data from three gridded global standard data products, namely, TRMM, APHRDOTIE and GPCP, have been used to simulate the model. Discharge data of Bahadurabad gage station of the Brahmaputra river have been used for model evaluation as flow data in the upper part of the catchment inside India are not available. It has been found that TRMM data are more accurate than APHRODITE and GPCP. Using TRMM data, the coefficient of determination (R2) and Nash-Sutcliffe efficiencies for calibration are found 0.83 and 0.73 respectively, whereas those values for validation are 0.81 and 0.72, respectively. It is notable that as we are using gridded rainfall data, the peak flow during the floods in 1998 and 2007 are underestimated by the model.

Keywords: Brahmaputra river basin, SWAT, sensitivity, uncertainty, Watershed

INTRODUCTION

The Ganges-Brahmaputra-Meghna (GBM) River system plays an important role in China, Bhutan, India, Nepal and Bangladesh. The GBM basin is the third largest freshwater outlet to the world's ocean (Chowdhuryet al., 2004). The 67% of the total annual water flow of Bangladesh is contributed by the Brahmaputra River (Immerzeel, 2008). This river basin is the main source of water in Bangladesh. Assessment of stream flows through this river can play a vital role for the water management of the country. However, estimation of water scarcity or water availability depends on the understanding of the hydrological system which is the main governing backbone of all kinds of water movement and water pollution(Jha,2011). Watershed analyses and hydrological modelling are important tools for management of many natural resources like, land and water. Thus, for proper planning and efficient utilization of the land and water resources, it is necessary to understand the hydrological cycle and estimate the hydrological parameters. But, setup and validation of a semi ungauged catchment are very difficult tasks. Calibration of model, sensitivity and uncertainty analysis can help to evaluate the ability of the model to sufficiently predict streamflow. However, over-parameterization is a well-known problem in such a distributed model. For this reasons, the main focus on this paper is to setup an appropriate hydrological model for the large Brahmaputra river basin.

A relatively recent modelling tool developed by the U.S. Department of Agriculture (USDA) called SWAT (Soil and Water Assessment Tool) has proven very successful in the watershed assessment of hydrology and water quality. The wide range of SWAT applications underscores that usefulness of model as a robust tool to deal with variety of watershed problems (Jha,2011).SWAT is a basin-scale, continuous-time model that operates on a daily time step and is designed to predict the impact of management on water, sediment, and agricultural chemical yields in ungauged watersheds. The model is physically based, computationally efficient, and capable of continuous simulation over long time periods. Major model components include weather, hydrology, soil temperature, plant growth, nutrients, pesticides, and land management. In SWAT, a watershed is divided into multiple sub-watersheds, which are then further subdivided into Hydrologic Response Units (HRUs) that consist of homogeneous land use, management, and soil characteristics. The HRUs represent percentages of the sub-watershed area and are not identified spatially within a SWAT simulation. Alternatively, a watershed can be subdivided into only sub-watersheds that are characterized by dominant land use, soil type, and management. The water balance of each HRU in the watershed is represented by four storage volumes: snow, soil profile (0 to2 meters), shallow aquifer (typically 2 to 20 meters), and deep aquifer (more than 20 meters). Flow, sediment, nutrient, and pesticide loadings from each HRU in a sub-watershed are summed, and the resulting loads are routed through channels, ponds, and/or reservoirs to the watershed outlet (Neitsch et al., 2002).

SWAT has been used extensively for assessment of water quantity and quality all over the world. Srinivasan and Arnold (1994) used the SWAT model to simulate water transport in the upper portion of the Seco Creek basin (114 km^2) in Texas. Monthly simulated streamflow data from SWAT were compared to monthly measured streamflow data for a 20-month period. The authors reported that there were no general tendencies to over or under predict surface runoff during certain seasons of the year. Simulated values compared well with measured values, with the average monthly predicted flows 12% lower than measured flows, and a Nash-Sutcliffe $R^2$of 0.86.SWAT model are found very useful to study changes of flow of the semi-ungauged Ganges due to climate change (Narsimlu et al,2013 and Gain et al,2011). Hence, it has been expected that SWAT can be able to generate flows for the Brahmaputra river basin which is unique in hydro-morphological nature. Over this basin, major precipitation occurs during the monsoon season. Staring from June till September, about 80 per cent or more of the annual rainfall occurs. These concentrated amount of precipitations are the main causes of water availability in Bangladesh. However, years of extreme precipitation, floods could inundate 70 per cent of the country and the physical damage could be very serious for the economy of Bangladesh. Therefore, setting up and calibrating SWAT model for the Brahmaputra river can assist many impact studies including the change of low flows or floods due to climate change which can act assist policy makers and planners.

METHODOLOGY AND DATA

Several types of data are required as input for SWAT to develop model using the ArcSWAT interference. Topographic data have been obtained from the Shuttle Radar Topographic Mission (SRTM) with a spatial resolution of 90m. The sub-basin parameters, such as slope gradient, slope length of the terrain, and the stream network characteristics (channel slope, length and width) are derived from the analysis of Digital Terrain Model. The DEM has been masked for the Brahmaputra basin area as shown in Figure 2.Land use maps are required for delineating of the HRU of the model. A 300m resolution land use from 2009 to 2010 are collected from Europe Space Agency GLOBCOVER(http://postel.mediasfrance.org/en/PROJECTS/Preoperational-GMES/GLOBCOVER/).This data has been reclassified to match the SWAT Land classes.

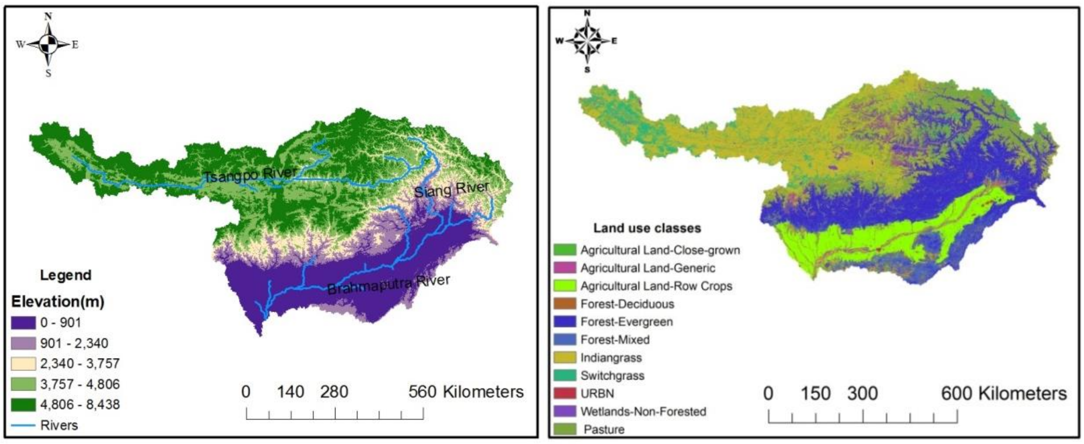

Fig 1: Digital Elevation map and land use map for Brahmaputra Basin.

Soil map of the study area was collected and extracted from the FAO digital soil map of the world (http://www.fao.org/geonetwork/). A ‘‘usersoil’’ database table was created for the study area from the available interpretations and lookup tables. SWAT requires daily or sub-daily observed meteorological data such as rainfall, maximum and minimum temperature, relative humidity, solar radiation and wind speed etc. Climatic data has been obtained from TRMM (Tropical Rainfall Measuring Mission),GPCP (Global Precipitation climatology Project) and APRHODITE (Asian Precipitation - Highly-Resolved Observational Data Integration Towards Evaluation).River discharge data are collected from Bangladesh Water Development Board (BWDB) for Bahadurabad station (Station No. SW 46.9L) on the Brahmaputra river. Flow data of this station are used for model calibration and validation. Watersheds have been delineated through automatic watershed delineation techniques. A threshold for minimum sub-basin area has been selected for the stream network and outlet calculation. As Bahadurabad station on the downstream of the Brahmaputra basin have been selected as the watershed outlet, this delineation resulted in a watershed of area 510452.91 km^2 and a total of 39 sub-basins. The land use and soil maps as derived from above mentioned methods have been imported and linked with the respective database table to create appropriate lookup tables. Multiple HRU option has been selected which generates 304 HRUs for the whole watershed**.** A period of 5 years (1998–2002) has selected for calibration and 5 years (2003–2007) for validation based on the availability of Aphrodite and GPCP. But for TRMM data sets, a period of 5 years (2000-2004) has been selected for calibration and 5 years (2005-2009) for validations. In addition, 1 year has been kept as warm-up period for both calibration and validation. A warm-up period allows the model to get a fully operational hydrological cycle and thus helps to stabilize the model. The main methods used in modeling the hydrologic processes in SWAT were curve number method for runoff estimating, Penman-Monteith method for PET and Muskingum method for channel routing**.**

RESULTS AND DISCUSSIONS

SWAT has been simulated using TRMM, APRHODITE and GPCP gridded rainfall data sets for the Brahmaputra river basin. Each simulation(experiment) independently was calibrated and validated against one discharge station. The calibration has been done by manually adjusting parameters until a good match have obtained between calculated and observed flows for each rainfall estimator. Then same adjusting parameters have been used for other rainfall estimators. Critical parameters have been identified during calibration were the curve number (CN2), baseflow recession constant (ALPHA_BF), soil water holding capacity(AWC),Minimum threshold depth of water required in shallow aquifer for ground-water flow to occur(GWQMN), Groundwater re-evaporation coefficient that controls the upward movement of water from shallow aquifer to root zone in proportion to evaporative demand (GWREVAP). Simulated discharge from GPCP and APRHODITE satellite data are underestimating peak flow of the flood in model calibration as shown in Fig 2. It has been found that satellite based gridded rainfall data is normally not able to capture high intensity rainfall observed by the typical single point raingauge. However, simulated discharges from TRMM datasets are overestimated for some years of model calibration. The patterns of the simulated flow hydrographs at Bahdurabad using these three satellite datasets are similar. During the period, where model is validated, simulated peak discharge using APRHODITE data underestimated observed peak discharge more than other two datasets i.e., GPCP and TRMM datasets. However, during the 1998 and 2007 floods, peak flows from the model derived by three datasets are heavily underestimated though using TRMM rainfall provides better results as shown in Fig. 3.A summary of the statistical analysis and comparison of model errors is given in Table 1.Nash index and correlation of determination R^2all confirmed that hydrological model driven by the precipitation data from TRMM data has performed better for both calibration and validation than those from GPCP and APRHODITE. Using TRMM data, the R^2 values were found as 0.83 and 0.81 and with Nash index are found as 0.73 and 0.72for calibration and validation of the model, respectively. Another statistical indicator, RSR(RMSE-observations standard deviation ratio) is calculated as the ratio of the RMSE (root mean square error) and standard deviation of measured data. The lower RSR, the lower the RMSE, and the better are the model simulation performances. RSR for simulated discharge using TRMM data e performed very well (values are less than 0.5) for both calibration and validation period. RSR from other datasets have shown values greater than 0.5 for the validation period. A RSR value less than 0.5 is reasonable accepted as per Moriasi et al. (2002). Also, the percent bias (PBIAS) has been satisfied and it has shown less than ± 25% (Moriasi et al, 2002). Here, TRMM data shows more accuracy for the both calibration and validation as PBIAS values are less than ± 25%. A positive value of PBIAS indicates underestimation whereas negative value indicates overestimation by the model. Although for the model calibration, discharge simulated by APRHODITE and GPCP data have good PBIAS values (less than ± 25%),results are not acceptable for the model validation.

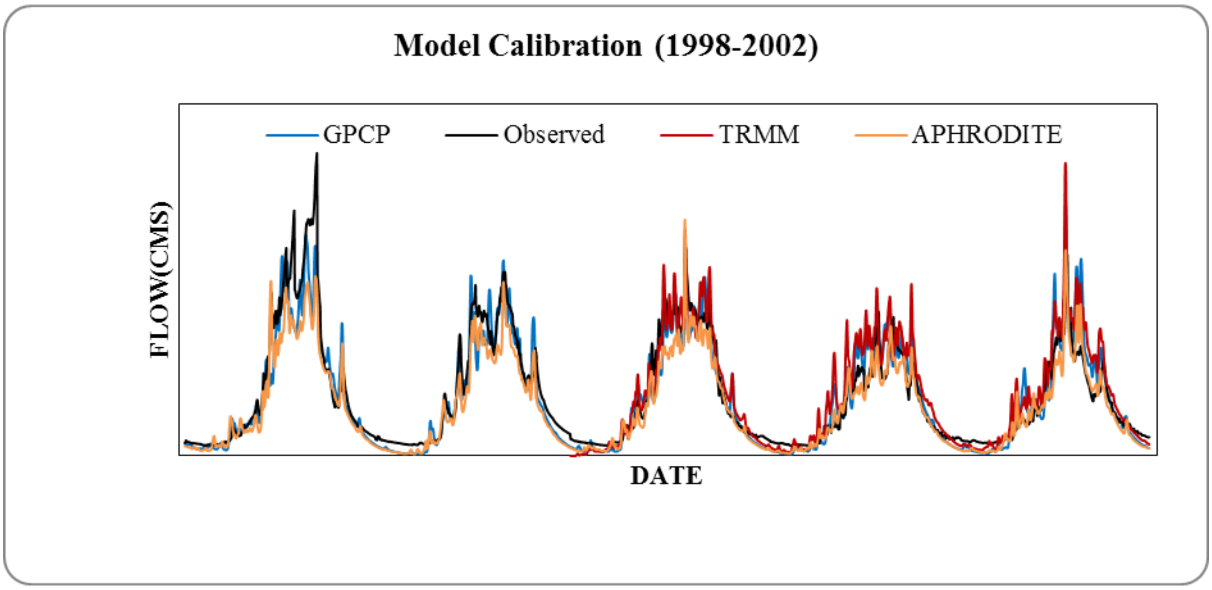

Fig 2: Model hydrographs for calibration period generated by gridded rainfall data products.

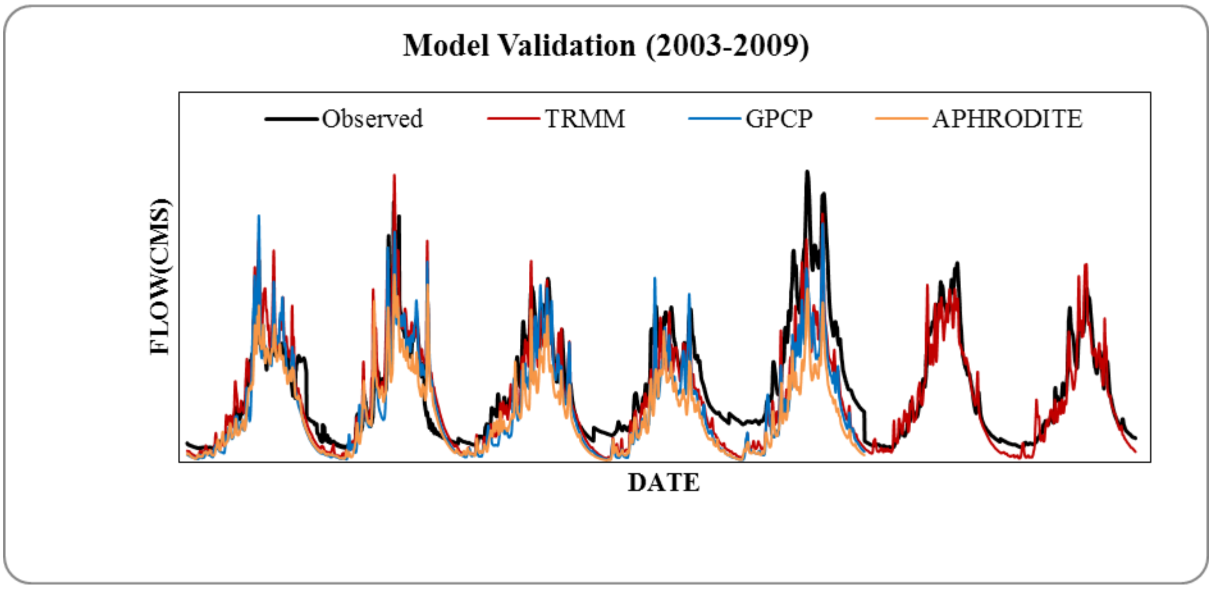

Fig3: Model hydrograph for validation period generated by gridded rainfall data products.

Table 1: Statistical parameter of calibration and validation period for different climate source

	Aphrodite		GPCP		TRMM	
Time period	1998-2002	2003-2007	1998-2002	2003-2007	2000-2004	2005-2009
R^2	0.91	0.71	0.85	0.72	0.83	0.81
NS	0.86	0.42	0.86	0.58	0.73	0.72
RSR	0.31	0.71	0.31	0.61	0.28	0.27
PBIAS	17	36	5	26	-14	21

CONCLUSIONS

Brahmaputra basin, like other watersheds in many parts of the world, is poorly gauged or ungauged. Streamflow data are not available. In this study, different satellite based gridded rainfall data products have been considered as precipitation source. Different parameters of the governing equations of the model are adjusted and fine-tuned to calibrate the model. The results of the study indicated that SWAT performed watershed simulations reasonably well using different sources of precipitation with parameterizing methods. Visual inspection from hydrographs and statistical indicators all have shown that the simulation performance of SWAT is better when using TRMM data. It can be seen that the accuracy of precipitation input determines the accuracy of model results. However, there are still some difficulties to predict peak flow in the flood year.

ACKNOWLEDGMENTS

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APPLICATION OF MATHEMATICAL MODEL FOR DREDGING RESPONSE IN IDEALIZED TEST CHANNELS

M. Musfequzzaman[1*], F. Noor[1] & M. A.Matin[2]

[1] Institute of Water Modelling, Dhaka-1206, Bangladesh.
[2] Department of Water Resources Engineering, BUET, Dhaka-1000, Bangladesh.
*Corresponding Author, <mzn@iwmbd.org>

Dredging in a river is certainly a human intervention need to be analyzed for better planning and decision processes. Intensive assessment of river response due to dredging is needed for this analysis though it is quite esoteric task as many variables are involved. In this study, two idealized test channels i.e. straight and meandering channels are set-up for model development and the results are explored for understanding the theoretical response of river due to dredging. Both the idealized test channels are 15 km long and 1 km wide. To setup these morphological models, MIKE21C, an advanced two-dimensional mathematical modeling software developed by DHI, has been applied and numbers of simulations have been conducted for different dredging conditions to fulfil the study objectives. From analyses, it has been found that with the increasing of dredging depth, dimensionless velocity increases along the dredged channel and decreases along the bank respectively. Moreover, for the straight channel, the bed is getting deeper along the dredged channel and siltation occurred near the bank. On the other hand, for meandering channel, the dredged channel is shifted towards the outer bend and siltation occurred at the inner bend.

Keywords: Dredging, river response, mathematical modeling, idealized channel, dimensionless velocity

INTRODUCTION

Getting idea on river response due to dredging is a complex task. To overcome this complexity two-dimensional morphological model has been a useful tool. River response is quite cryptic topics in river engineering. Many qualitative and quantitative theories are there to make it logical to us. Especially in case of dredging, in what way the river behaves itself due to this. Rivers always try to achieve a stable state of equilibrium throughout it reaches over a period of time. Change in a river due to any interference is a time depending morphological process in nature (Vries, 1993). Many researchers like Lane (1955), Schumm (1969), Bettess and White (1983) and Sarker and Thorne (2006) have developed numerous experimental works, analytical studies, numerical modelling and conceptual model in the field of river response. However the necessary dredging related study yet to be carried out. On this context an attempt has been taken to understand the river response due to dredging theoretically in idealized test channel using 2D morphological model.

METHODOLOGY

Idealized test models are mainly developed for theoretical justification. Two types of idealized channels with different dredging depth have been selected as a model to understand the behaviour of any real river due to dredging. In this context, one straight channel and one meandering channel are set up in such a way so that the water level and velocity passes through the idealized channels are exactly same as passes through a single channel of the river Jamuna. Parameters of idealized channels are also kept same as the model of Jamuna River (Musfequzzaman, 2012). In this study Models are simulated for different dredging depth. Then the velocities at different locations for varying dredging depth of the channels are taken to analyse the river response. This analysis is made universal by determining the velocity as dimensionless. Finally, various relationships have been come out from the velocity versus dredging depth curve.

Description of the Model Setup

To evaluate the response due to dredging, two-dimensional morphological model has been setup using MIKE 21C software. Model setup included grid generation, bathymetry preparation, boundary setup, parameter setup etc.

A 15 km long curvilinear grid has been generated to represent the straight channel and meandering channel. Figure 1 shows the computational grid and bathymetry of the straight and meandering channel. The bathymetry of those channel have been made by keeping the bed slope 7 cm per kilometre (FAP 24, 1996). A constant boundary has been applied at the u/s boundary condition by analyzing the conveyance properties. The cross sectional area of Jamuna River near Sirajganj Hardpoint has found 20419.6 m^2(Musfequzzaman, 2012). On the other hand the areas for those idealized channels are 12000m^2. It is worth mentioning here that almost 70% water of total discharge of Jamuna River passes along the Sirajganj Hardpoint during peak time (IWM 2012). That means for average flood event (2005), about 65987*0.7 = 46191 m^3/s discharge has passed through the Jamuna River during peak time along the channel near Sirajganj Hardpoint. Comparing the conveyance area of Jamuna River with the idealized channel, discharge has been found 27145m^3/s which has been used as u/s boundary for the idealized channels. Sediment boundary is selected for the corresponding flow discharge from sediment rating curve of Jamuna River (FAP 24, 1996).

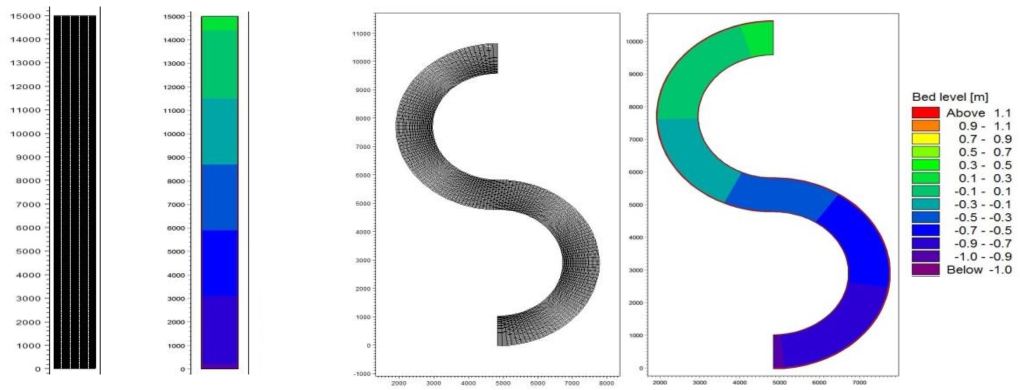


Fig 1: Computational grid and generated bathymetry for the straight and meandering channel

Parameters are the important key for model setup which are called tuning properties. In this study, the following parameters have been used. These tuned parameters are exactly the same parameters used in a model of the Jamuna River (Musfequzzaman, 2012). The alluvial resistance factor and exponent are used as 20 and 0.5, respectively. In all the simulations, the value of eddy viscosity is 1. The sediment transport of equation of VanRijn has been used in this model.

Determination of Critical Velocity

Critical velocity is the velocity needed to transport sediment load (Bull, 2012). For non cohesive soil, assuming a steady uniform flow, the basic stability criterion can be expressed as (BUET, 2008) -

$$\tau_{cr} = \Psi_{cr}(\rho_s - \rho_w).g.D > \tau_o \quad (1)$$

where,

τ_{cr}= Critical shear stress, τ_o= shear stress exerted along the bank/bed boundaries, ψ_{cr}= dimensionless critical Shields shear stress parameter for specific material, D= mean grain size, G = acceleration due to gravity, ρ_s = density of soil, ρ_w = density of water.

The value of Ψ_{cr} depends on particle shape, velocity profile, etc. For fine sediments with D< 4 mm, the Shields parameter can be written as:

$$\psi_{cr} = \frac{\tau_c}{(\gamma_s - \gamma_w)D} = \frac{U_*^2}{\Delta_m g.D} \quad (2)$$

where,

$$U_* = \sqrt{g \cdot R \cdot S} = \frac{u \cdot \sqrt{g}}{C} \quad (3)$$

where,

Δ_m (-) = $(\rho_s-\rho_w)/\rho_w$, i.e.,relative density of submerged material, U*= shear velocity, R=hydraulic radius or water depth, S = slope of energy line, u= mean velocity, C = Chezy coefficient and h = depth of flow.

For practical cases the following formulas can be used to estimate the critical velocity (U_c) causing subsoil erosion:

$$U_c = (\sqrt{g \cdot \Delta \cdot D_{50}}\sqrt{\psi_{cr}}) * 5.75 * \log\frac{6h}{D_{50}} \quad \text{(finer sediment)} \quad (4)$$

Determination of Dimensionless Instantaneous Velocity

Incipient motion is important in the study of sediment transport, as this motion of sediment particles in alluvial rivers is the key force for the mobility. It represents the difference between bed stability and bed mobility (Matin, 1994). Due to the stochastic nature of sediment movement along an alluvial bed, it is difficult to define precisely at what flow condition a sediment particle will begin to move (Yang, 2006). Consequently, it depends more or less on an investigator's definition of incipient motion.

The initiation of motion of a particle could be identified by comparing the instantaneous velocity with the critical velocity. Therefore if the ratio between instantaneous velocity and the critical velocity is higher than unity, the particle would initiate its motion. In this study, the instantaneous velocity along the structures has been extracted from the model simulation for varying dredging depth and finally dimensionless velocity has been calculated at the expected location. The dimensionless velocity corresponding to the relative dredging depth for straight and meandering channels are shown in Table 1.

Table 1: Dimensionless velocities corresponding to the relative dredging depth for straight and meandering channel (as a sample calculation)

Location	Dredg. depth, D_d	Avg water depth at dredging location	Dimensionless dredg. depth	along dredging location	Near bank location	along dredging location		Near bank location	
				Velocity	Velocity	Critical velocity, U_c	V/V_{cr}	Critical velocity, U_c	V/V_{cr}
	m	m	-	m/s	m/s	m/s	-	m/s	-
Straight channel	0	11.92	0	2.40	2.30	2.27	1.05	2.19	1.05
	0.5	13.30	0.037	2.51	2.24	2.29	1.09	2.18	1.03
	1	14.59	0.068	2.61	2.22	2.31	1.13	2.18	1.02

RESULTS AND DISCUSSIONS

In this study to understand the river response due to dredging various morphological outputs such as channel layout, bed scour and quantitative velocity coming from the model simulations are analysed. The results of these analyses are presented in the following section.

Channel Layout

The simulated channel layouts of varying dredging depth of the straight and meandering channel are presented in Figure 2 and Figure 3, respectively. It has been observed that, in case of a straight channel the dredged channel become deeper for higher dredging depth and silted along the bank (Figure 2). On the other hand it has been observed that, for the meandering channel the dredged channel shifted towards the outer bend by silting the inner bend (Figure 3). Here, dredged channel also become deeper with the increasing of the dredging depth (Figure 3).

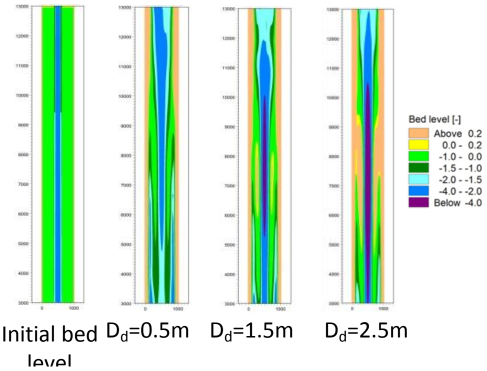


Fig 2: Simulated bed level of straight channel for different dredging depth

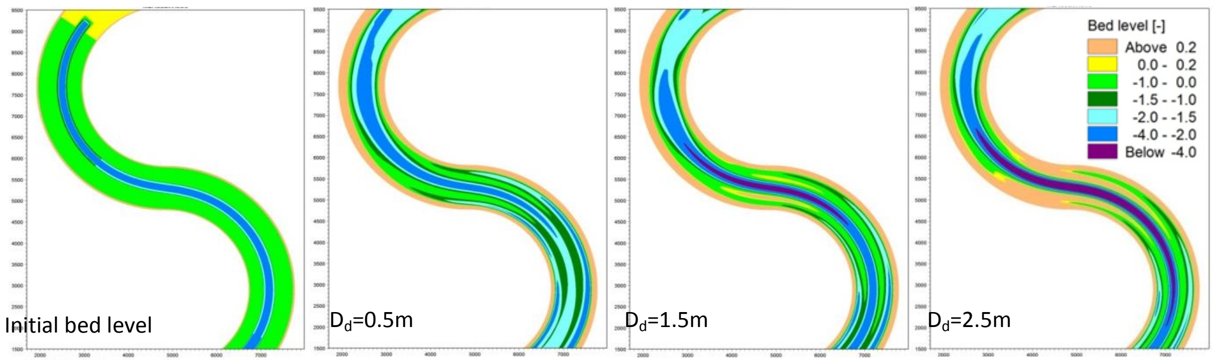
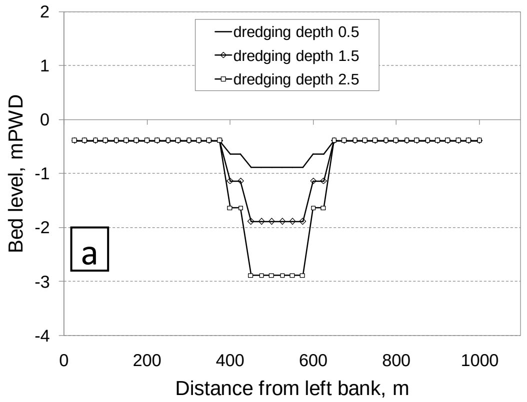


Fig 3: Simulated bed level of meandering channel for different dredging depth

Bed Scour

The simulated cross sections of varying dredging depth of the straight and meandering channesl are presented in Figure 4 and Figure 5, respectively. In case of a straight channel, it is seen that as the dredging depth increases, the dredged channel become deeper and siltation is occurred near the bank, (Figure 4). Again in case of a meander channel, it is seen that as the dredging depth increases, the dredged channel is slightly shifted towards the outer bend and siltation is occurred at the inner bend, shown in Figure 5.



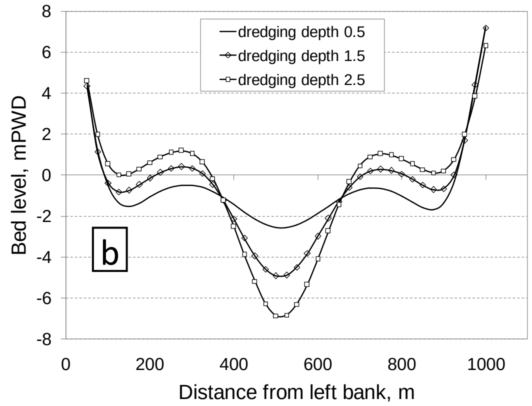


Fig 4: Simulated cross section of straight channel at a) initial and b) end condition for different dredging depth

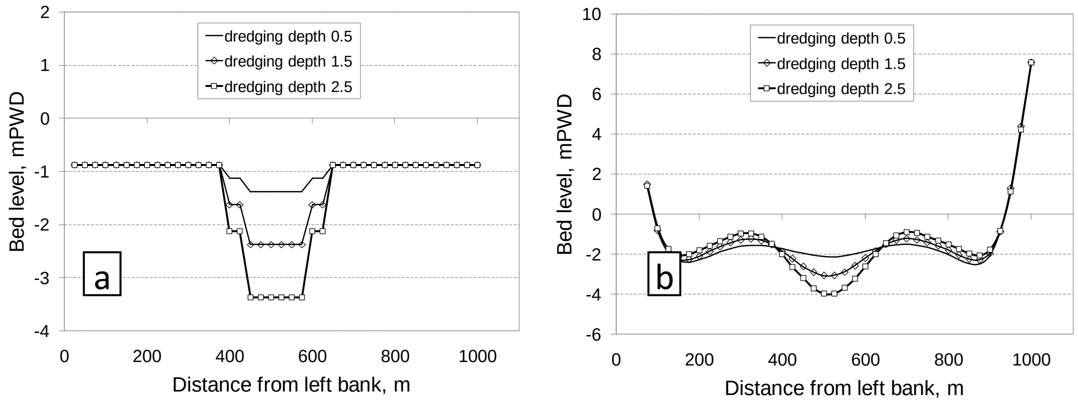


Fig 5: Simulated cross section of meandering channel at a) initial and b) end condition for different dredging depth

Relationship between Dimensionless Velocity and Relative Dredging Depth

Experimentally, two different types of morphological models i.e. straight channel, meandering channel have been simulated incorporating the dredging alignment. To determine the dimensionless velocity and relative dredging depth, average water depth and average velocity have been extracted from these models and finally, relationship between dimensionless velocity and dimensionless dredging depth has been plotted (Figure 6 and Figure 7).

For Straight Channel

From Figure 6a, it is seen that along the dredged channel dimensionless velocity increases with the increase of relative dredging depth. As the velocities become higher than critical velocity, the dredging zone is more erosive than the non dredged condition. It is worth mentioning here that for the first 1m increment of relative dredging depth in the straight channel, the increasing rate of velocity is about 7%. Again for the next 1m increment of relative dredging depth, the increasing rate of velocity is found 4%.

From Figure 6b, it is seen that near the bank, dimensionless velocity decreases with the increase of relative dredging depth. As the velocities become lower than critical velocity and their ratio become less than 1 after certain dredging depth, the both of the banks become non erosive.

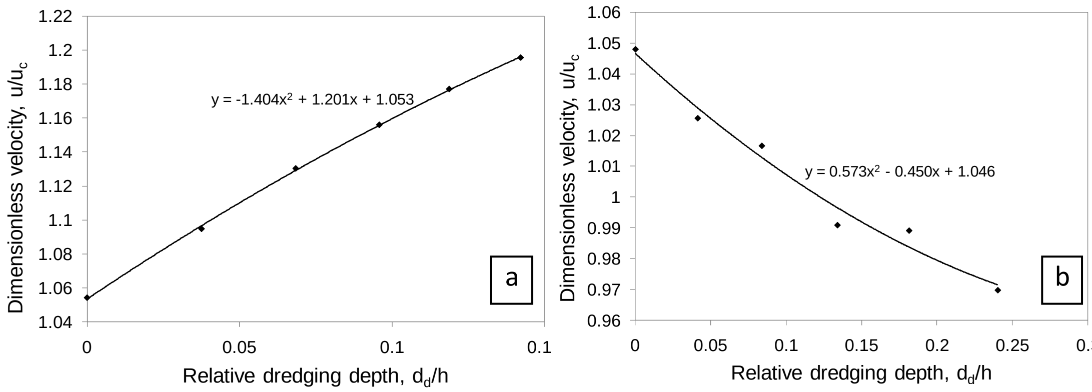


Fig 6: Dimensionless velocity versus relative dredging depth curve for straight channel a) along the dredged location and b) near bank location.

For Meandering Channel

For the meandering channel, results have been extracted for three different locations; one is along the dredged channel and the other two are at the outer bend and at the inner bend of meander channel. From Figure 7a, it is seen that along the dredged channel the dimensionless velocity increases with the increase of relative dredging depth. Here, also the dredging zone is more erosive than the non dredged condition as the velocities become higher than critical velocity. It is worth mentioning here that for the first 1m increment of relative dredging depth in the meandering channel, the increasing rate of velocity is about 9%. Again for the next 1m increment of relative dredging depth, the increasing rate of velocity is found 6%.

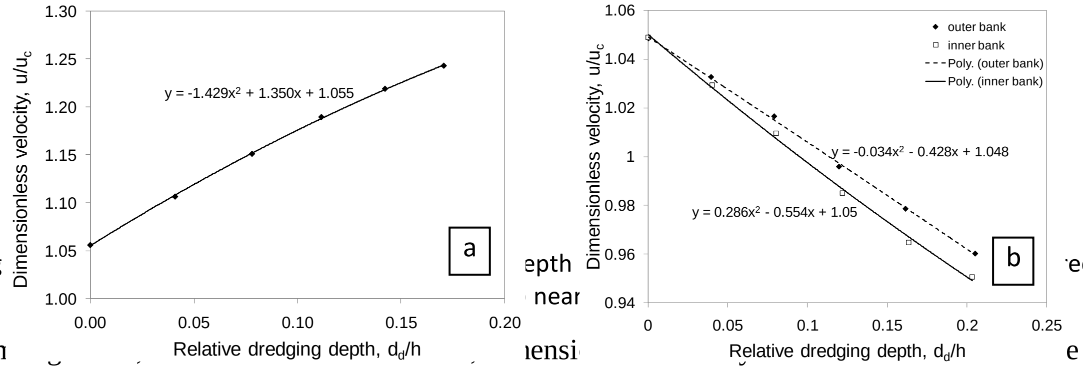


Fig ... epth ... near ... edged

Fron ... iensi ... e of relative dredging depth. As the velocities become lower than critical velocity and their ratio become less than 1 after certain dredging depth hence the bank would behave like a non erosive zone. But there is a slight difference of velocity between the outer bank and inner bank. As the flow attracted towards the outer bank, the velocity at the outer bank is higher than the inner bank.

Summary Representation of River Response in Idealized Test Channels

Various outcomes regarding to river response due to dredging in idealized channels coming from different simulations have been summarized in the following Table 2.

Table 2: River response due to dredging

Channel and location		Dependent variable	Independent variable
		Velocity	**Dredging depth**
Straight Channel	Along the dredged channel	+	+
	Near the bank	-	+
Meandering Channel	Along the dredged channel	+	+
	At the inner bank	-	+
	At the outer bank	-	+

\+ means increasing - means decreasing

CONCLUSION

The following conclusion can be drawn after summarizing the present study. In idealized test channels it has been found that with the increasing of dredging depth, dimensionless velocity increases along the dredged channel and decreases near the bank location. Moreover, for the straight channel, the bed is getting deeper along the dredged channel and siltation occurred near the bank. On the other hand, for meandering channel, the dredged channel is shifted towards the outer bend and siltation occurred at the inner bend.

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ID: WRE 008

STUDY ON THE EFFECTIVENESS OF BIO-SHIELD ALONG KUAKATA BEACH OF BANGLADESH IN REDUCING THE STORM SURGE ENERGY USING BOB MODEL

Md. Arifur Rahman[1*], Md. Mostafa Ali[1] & Md. Ataur Rahman[2]

[1]*Dept. of Water Resources Engineering, BUET, Dhaka-1000, Bangladesh*
[2] *Dept. of Civil Engineering, Presidency University, Dhaka-1000, Bangladesh.*
**Corresponding Author, <arifur@wre.buet.ac.bd >*

In this study,a two-dimensional numerical model named Bay of Bengal (BoB) model is used to simulate the effect of coastal bio-shield in reducing the energy of storm surge SIDR along the Kuakata beach of Bangladesh. Four simulations were performed for four different conditions. Surge levels and current speeds were measured at three specific locations (west, middle and east part of Kuakata beach) on the Kuakata beach (polder-48). The surge level and current speed are compared with the respective base conditions to find out the reduction of surge height and current speed diminished by the 200 m wide bio-shield barrier. From the simulation results it is found that due to the presence of bio-shield at the foreshore, the surge heights are reduced by 1-2 cm and the surge speeds are reduced by 59% to 70% compared to no bio-shield situation. Therefore, the existence of a bio-shield at the foreshore has strong effect in reducing the speed of a storm surge.

Keywords: Bio-shield, Storm surge, SIDR, Current speed, Wave height, Numerical modeling, BoB model.

INTRODUCTION

Bangladesh has a coastal belt of about 710 km long from Raimongal River to the west and Teknaf to the Southeast. It has an area of about 144,000 square kilometres and a population of more than 150 million, of which 28% of the total population lives in the coastal region (Paul, 2009). Almost the entire coastal belt of Bangladesh is exposed to the potential danger of cyclones with associated storm- surges. During the years of 1582 to 2013, Bangladesh has been hit by 75 severe cyclones, 35 of which were accompanied by storm surges (Paul, 2009). A cyclone in November 1970 hit the southern districts of Bangladesh forcing a 9 m high storm surge and killing approximately 300,000 people (Haider et al., 1991). The cyclone of 1991 caused more than 145,000 lives (Haider et al., 1991). In 2007, about 3406 people were reportedly killed in the coastal areas of Bangladesh by the super cyclone SIDR (Paul, 2009). In 2009, about 330 people were died by cyclone Aila in the southern coastal region of Bangladesh which hit the country with a wind speed of 110 km/h and 3.0 m of surge. Bangladesh is one of the top vulnerable countries to the climate change (Ali, 1999). Due to climate change and sea-level rise, the country is likely to be affected by more intense cyclonic events in the foreseeable future. According to the 5th IPCC report (IPCC, 2013) a global mean sea level rise (SLR) would be 74 cm up to 2100. Again according to the 4th IPCC report (IPCC, 2007) the intensity and frequency of the cyclonic storm surge will be increased in the near future in the Bay of Bengal. The cyclones and storm surges of high magnitude associated with future climate change effect can be faced by green belt bio-shield protection along the coastal areas to save the hinterland against future severe natural disasters (Rahman and Rahman, 2013). Tanaka (2009, 2006) developed a different approach to numerically model the storm surge from Cyclone SIDR, which made landfall in Bangladesh in 2007. He modeled the passage of short period wind waves (wave period 1 or 2 minutes) and a longer period storm surge (wave period 1 or 2 hours) through trees; both types of waves were modeled separately and in combination. The modeled vegetation characteristics were based on the non-mangrove tree species *Casuarina* *equisetifolia*; trees were modeled as cylinders, 10 m high and 16 cm in diameter, with 0.35 trees per m^2 in a triangular arrangement. The underlying topography and vegetation measurements matched those seen in transects in Mathbaria, Bangladesh, and the model results were compared with observations of how this area was affected by Cyclone SIDR. In this study, a two-dimensional numerical model named Bay of Bengal (BoB) model is used to simulate the effect of coastal bio-shield in reducing the energy of storm surge SIDR along the Kuakata beach of Bangladesh.

METHODOLOGY

A numerical model named MIKE21 developed by DHI (Danish Hydraulic Institute) is used to develop the Bay of Bengal (BoB model) model by Institute of Water Modeling. This model has been applied to the Kuakata beach of Bangladesh to simulate the effectiveness of coastal bio-shield against the current speed and surge level of super cyclone SIDR happened in 2007. MIKE 21 has a number of modules for different purpose and each module has different sets of equations. In this study, hydrodynamic and cyclone module of MIKE 21 have been used.

Governing Equation in Hydrodynamic Module:

The governing equations used in MIKE21 in solving hydraulic problems in coastal areas are:

Conservation of Mass Equation:

$$\frac{\partial \varepsilon}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=0 \qquad \text{..........(1)}$$

Conservation of momentum equation:

The momentum equation in the x-direction is given by:

$$\frac{\partial p}{\partial t}+\frac{\partial \left(p^2/h\right)}{\partial x}+\frac{\partial \left(pq/h\right)}{\partial y}+gh\frac{\partial \varepsilon}{\partial x}+\frac{f}{2}\frac{\sqrt{p^2+q^2}}{h^2}p-\frac{1}{\rho}\frac{\partial \tau_{xy}h}{\partial y}-\Omega q-\frac{\rho_a}{\rho}C_w W W_x+\frac{h}{\rho}\frac{\partial p_a}{\partial x}=0 \qquad \text{......(2)}$$

The momentum equation in the y-direction is given by:

$$\frac{\partial q}{\partial t}+\frac{\partial \left(q^2/h\right)}{\partial y}+\frac{\partial \left(pq/h\right)}{\partial x}+gh\frac{\partial \varepsilon}{\partial y}+\frac{f}{2}\frac{\sqrt{p^2+q^2}}{h^2}q-\frac{1}{\rho}\frac{\partial \tau_{yx}h}{\partial x}+\Omega p-\frac{\rho_a}{\rho}C_w W W_y+\frac{h}{\rho}\frac{\partial p_a}{\partial y}=0 \qquad \text{.......(3)}$$

Where,

p and *q* = flux in x and y directions respectively ($m^3/s/m$); t = time (s), x and y (m) are Cartesian Co ordinate (s); h =water depth (m); g =acceleration due to gravity (9.81 m^2/s); ε =Sea surface elevation (m).

ρ_w & ρ_a =Air and water density respectively (kg /m^3); C_w =wind friction factor = 0.0008 + 0.000065W in accordance with Wu (1982); W =wind speed (m/s); Ω=Carioles' parameter; P_a =Atmospheric pressure (kg/m/s^2).

Governing Equation in Cyclone Module:

The Cyclone module of MIKE21 has been used to generate the pressure and wind distributions all over the Bay of Bengal. Cyclone Model is normally described by relatively few parameters relate to pressure field, which is imposed to the water surface and a wind field which is acting as a drag force on the water body through a wind shear stress description. The wind fields consist of a rotational and a translational component. At a distance *R* from the centre of the cyclone the rotational wind speed V_r is given as

$$V_r = V_m \left(\frac{R}{R_m}\right)^7 \exp\left(7\left(1-\frac{R}{R_m}\right)\right) \quad \text{for} \quad R<R_m \qquad (4)$$

$$V_r = V_m \exp\left(C\left(1-\frac{R}{R_m}\right)\right) \quad \text{for} \quad R>R_m \qquad (5)$$

Where *R* and R_m are in km and *C* is given as

$$C = 0.0025 R_m + 0.05 \qquad (6)$$

C determines the shape of wind distribution for $R>R_m$.

And translational component V_t is given as

$$V_t = -0.5 V_f (-\cos\theta) \qquad (7)$$

Where, θ is the angle between the radial arm and the line of maximum winds. The total wind speed is $V = V_r + V_t$

And finally, the pressure at particular location is given as

$$P = P_c + (P_n - P_c)\exp\left(\frac{-R_m}{R}\right) \qquad (8)$$

Where *P* is pressure at radius *R*, P_c is central pressure, P_n is neutral pressure.

The two-dimensional numerical model known as BoB model has benn applied to the Kuakata beach of Patuakhali district. Four simulations are done for various scenarios to study the behavior of coastal bio-shields for the reduction of storm surge and cyclone energy along the Kuakata beach considering future climate change effects. The numerical modeling run conditions are given in the Table 1.

Table 1: Numerical Modeling Run Conditions

Run No	Width of bio-shield, (m)	Distance from embankment, (m)	Climate change Effect	Cyclonic Wind Speed (km/h)
1	200	200	Sea level rise 74 cm for the year 2100	210 (For super cyclone SIDR in the year 2007)
2	200	200	No sea level rise	
3	No bio-shield and no sea level rise			
4	No bio-shield and considering 74 cm of SLR			

STUDY AREA

Bangladesh is one of the most vulnerable countries to several natural disasters and every year natural calamities upset people's lives in some part of the country, especially in the southern coastal areas of Bangladesh. The major disasters concerned here are the occurrences of cyclones and storm surges.

People of coastal area of Bangladesh are the main victim of these cyclonic storm surges. Presently, the Kuakata beach of Patuakhali district shown in Fig. 1 is very much vulnerable to these natural disasters and it is the most attractive place to the visitors as well. That's why this beach is selected for the study.

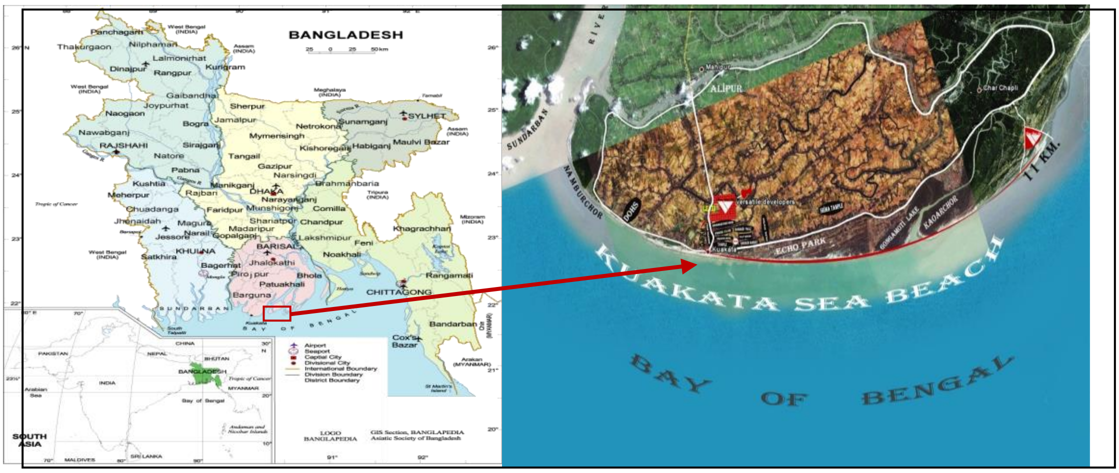

Fig. 1: Location of Kuakata beach on the Bangladesh map

RESULTS DISCUSSION

A two-dimensional numerical model named Bay of Bengal model developed by Institute of Water Modeling (IWM) was used with the help of IWM to study the effect of coastal bio-shield in reducing the energy of storm surge SIDR on the Kuakata beach of Bangladesh. In this numerical modeling four simulations were performed for four different conditions (i.e. no bio-shield and no SLR; no bio-shield and considering 74 cm of SLR; 200 m width of bio-shield at 200 m away from the polder-48 and no SLR; bio-shield of 200 m width at 200 m away from the polder-48 with 74 cm SLR). Surge levels and current speeds are measured at three specific locations on the kuakata beach (polder-48). The Fig.2 shows the measuring points on the map. The surge level and current speed are compared with the respective base conditions (i.e. no bio-shield and no SLR; no bio-shield and considering 74 cm of SLR) to find out the reduction of surge height and current speed diminished by the 200 m bio-shield barrier. The results of the numerical simulations are tabulated in the Table 2, Table 3 and Table 4.

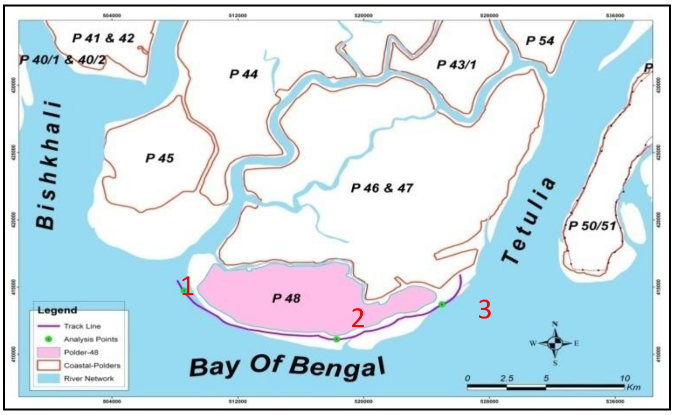

Fig. 2: Three specific measuring points on the kuakata beach (Polder-48)

From the model results it is pretty obvious that the surge height and current speed or thrust of storm surge SIDR reduces by 1 cm and 0.23 m/s or 70 % at point-1 for considering only 200 m width of bio-shield and no SLR. Again, when 74 cm of sea level rise (SLR) is considered the surge height and current speed also reduce by 2 cm and 0.24 m/s or 68 % by the 200 m bio-shield barrier. At point-2, the surge height again decreases by 1 cm and the current speed also reduces by 0.38 m/s or 63 % for considering only 200 m width of bio-shield and no SLR. Moreover, when 74 cm of sea level rise is considered the surge height decreases by 2 cm and current speed also reduces by 0.36 m/s or 58 %. Finally, at point-3 the surge height and current speed reduce by 2 cm and 0.48 m/s or 62 % respectively by 200 m width of bio-shield and no SLR. And again, the surge height and current speed reduce by 3 cm and 0.46 m/s or 59 % respectively for considering 74 cm of sea level rise.

Table 2: Comparison of surge level and current speed at measuring point-1

Item description	Surge level (mPWD)	Current Speed(m/s)	Change in surge height (cm)	Change in current Speed	
No bio-shield and no SLR	1.99	0.33	-	m/s	%
Bio-shield of 200 m width at 200 m distance from polder-48 and no SLR	1.98	0.10	- 1	-0.23	70
No bio-shield and considering 74 cm of SLR	2.70	0.35	-	-	-
Bio-shield of 200 m width at 200 m distance from polder-48 and considering 74 cm of SLR	2.68	0.11	- 2	-0.24	68

Table 3: Comparison of surge level and current speed at measuring point-2

Item description	Surge level (mPWD)	Current Speed(m/s)	Change in surge height (cm)	Change in current Speed	
No bio-shield and no SLR	2.93	0.60	-	m/s	%
Bio-shield of 200 m width at 200 m distance from polder-48 and no SLR	2.92	0.22	- 1	-0.38	63
No bio-shield and considering 74 cm of SLR	3.63	0.62	-	-	-
Bio-shield of 200 m width at 200 m distance from polder-48 and considering 74 cm of SLR	3.61	0.26	- 2	-0.36	58

Table 4: Comparison of surge level and current speed at measuring point-3

Item description	Surge level (mPWD)	Current Speed(m/s)	Change in surge height (cm)	Change in current Speed	
No bio-shield and no SLR	4.74	0.77	-	m/s	%
Bio-shield of 200 m width at 200 m distance from polder-48 and no SLR	4.72	0.29	- 2	-0.48	62
No bio-shield and considering 74 cm of SLR	5.45	0.78	-	-	-

Bio-shield of 200 m width at 200 m distance from polder-48 and considering 74 cm of SLR	5.42	0.32	- 3	- 0.46	59

The results of numerical simulations that have been performed on the Kuakata beach (polder-48) of Bnagladesh shows that a 200 m width of bio-shield barrier placed at 200 m distance from the polder-48 is capable of reducing up to 0.48 m/s or 62% of current speed when SLR does not come into consideration and 0.46 m/s or 59 % for the consideration of 74 cm of SLR. It has also effects on surge height reduction, that is the bio-shield barrier can reduce the surge height up to 2 cm and 3 cm for without and with considering SLR respectively. The reduction in current speed and surge level would be more prominent if the width and location of bio-shields barrier would vary as two fold or three fold which is found in others studies.

CONCLUSION

A two-dimensional numerical model named Bay of Bengal model has been used to simulate the effect of coastal bio-shield in reducing the energy of storm surge SIDR along the Kuakata beach of Bangladesh. Four simulations were performed for four different conditions. Surge levels and current speeds are measured at three specific locations (west, middle and east part of Kuakata beach) on the Kuakata beach. Simulation results prevails that the existence of a bio-shield at the foreshore has strong effect in attenuating the surge height and also in reducing the current speed of a storm surge. From this study, it is suggested that dense bio-shields barrier should be preserved as wide as possible not less than 200m wide in front of the coastal sea facing polders to reduce the effect of high frequency cyclonic storm surges for protecting the hinterland.

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ESTIMATION OF DOMINANT DISCHARGE IN THE GUMTI RIVER

M. Ahammad[1*] & M. Musfequzzaman[1]

[1] *Institute of Water Modelling, Dhaka-1206, Bangladesh.*
**Corresponding Author, <muneer.buet@gmail.com>*

ABSTRACT

Various methods have been proposed by different investigators for the choice of representative discharge which is very important in studying the river characteristics. This representative discharge is also referred to as dominant discharge in the literature and may be defined as that hypothetical discharge which would produce the same result in terms of average channel dimensions as the actual varying discharge. Moreover, dominant discharge produces maximum morphological activities for a regime channel. Estimation of the frequency of dominant discharge in the rivers is necessary for flood plain management. The determination of the dominant discharge is also very important for flood mitigation and estimation of flood damage. As a case study, different concepts (flow duration concept, bankfull discharge concept, bed generative discharge concept, meander wavelength concept) are adopted here for the computation of dominant discharge of the Gumti which is a hilly river having a strong current. In this study the frequency of dominant discharge of the Gumti River at Comilla is investigated and the dominant discharge is also determined. The flood discharge at a particular station (Station ID-110) for 30 years have been analysed by collecting the data from BWDB. The flood frequency analysis is used to find the return period of the dominant discharge in the Gumti River. It is found that the dominant discharge in the Gumti River at the above station has return period of around 1.005 years on partial series analysis. Finally, the dominant discharge for the Gumti River is found around 7000 cusec.

Keywords: Dominant discharge, Bankfull stage, Gumti River, Rating curve, Sediment load transport.

INTRODUCTION

The dominant discharge is the flow doing most geomorphic work and it is, therefore, the channel forming discharge. Determining the dominant discharge is very important for sedimentation problem in the river and it is also very useful for the riverine stabilization and fish habitats. Despite the importance of the dominant discharge, it is not yet completely determined for the existing rivers. Inglis (1949) defined that the dominant discharge in a natural stream is a discharge, representative of a whole range of discharges that pass through the channel and that forms the channel morphology. The dominant discharge is usually defined as; i) the most effective discharge for sediment transport. Benson and Thomas (1966) defined that the dominant discharge is the discharge which transports the most sediment transport in suspended load. Pickup and Warner (1976) defined the dominant discharge as the discharge that transports the sediment particles as the bed load. Andrew (1980) defined the dominant discharge as the discharge which transports the most sediment particles as total load. ii) The natural bankfull discharge or the discharge in a river which just fills the main channel and not overbanking the flood plains. iii) The dominant discharge is also defined as the discharge or a flood of fixed frequency such as 1-2 years flood and iv) it is defined as the discharge which exhibits the best statistical correlation with various channel morphological characteristics.

Dominant discharge was also studied by many researchers; for instance, Williams (1978) found that the dominant discharge is a bankfull discharge of approximately 1.5 years. Keshavarzi and Erskine (1995) and Erskine and Keshavarzy (1996) investigated that the dominant discharge on South Creek in New South Wales, in Australia has an average recurrence interval (ARI) of 1.89 to 2.40 years on the partial series. Valentine et al. (2001) studied regime theory and the stability of straight channels with bankfull.

For the rivers of Bangladesh, Hossain (1992) computed dominant discharge of the Ganges and Jamuna by bed generative discharge concept using the observed sediment discharge at Hardinge Bridge and Bahadurabad.

In this study, total sediment load transport was primarily used to estimate the dominant discharge. Here the frequency of dominant discharge of the Gumti River in Comilla was determined using 30 years of recorded flood discharge. The stage discharge curve and meander wavelength method were also used to find the dominant discharge.

METHODOLOGY

As a case study, different concepts (flow duration concept, bankfull discharge concept, bed generative discharge concept, meander wavelength concept) are adopted here for the computation of dominant discharge of the Gumti River. This river originates from Dumur in the North-eastern hilly region of Tripura state of India. From its source it flows about 150 km along a meandering course through the hills, turns west and enters Bangladesh near Katak Bazar (Comilla Sadar), shown in Figure 1. Then it takes a meandering course again and passes through the northern side of Comilla town. The Gumti is about 135 km long within Bangladesh. It is a hilly river having a strong current in which flow varies from 100 to 20,000 cusec at Comilla. Flash floods are common phenomena of this river and it occurs at regular intervals.

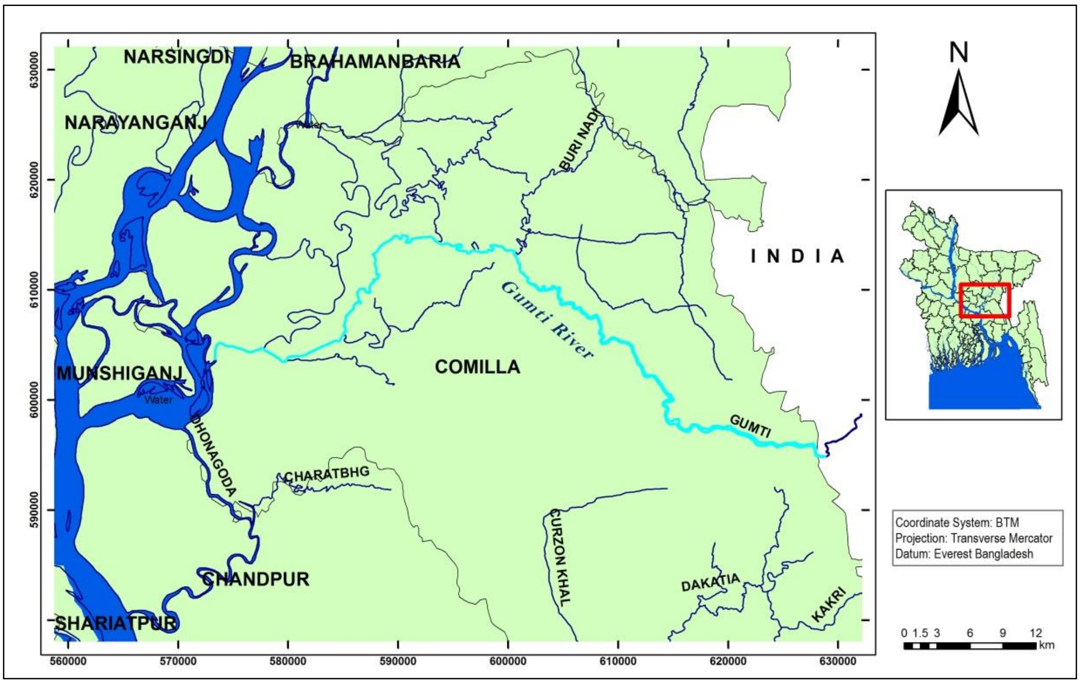

Fig 1: The Gumti River in Bangladesh

To estimate the dominant discharge, firstly a graphical relationship was established between discharge and the frequency of discharge. Then another graphical relationship was developed between flow discharge and sediment load discharge at the same station. Finally the two relationships were used to develop a third histogram which relates discharge and product of sediment load and frequency. The dominant discharge corresponds to largest volume of sediment discharge which can be read directly from the histogram.

The computed dominant discharge was compared to the discharge at bankfull stage using rating curve of that station. The Dury's method (1965) of dominant discharge using meander wavelength was also used. Meander wavelengths were measured from the satellite images by ArcGIS-10 software. To check the return period of dominant discharge, flood frequency analysis was applied to the 30 flood data at Comilla station. Water level and discharge data (1965-1994) and sediment data (1986-1993) of the above station were collected for this study from Bangladesh Water Development Board (BWDB).

RESULTS AND DISCUSSIONS

Total load transport

Figure 2 to 4 show the steps involved in the estimation of dominant discharge as described earlier.

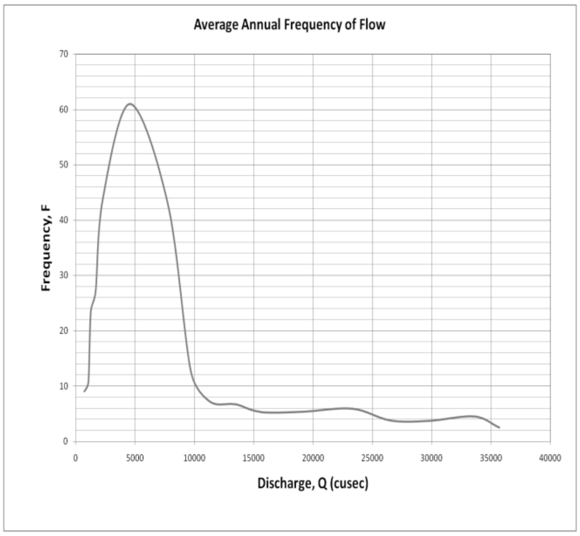

Fig 2: Discharge-Frequency relation of the Gumti at Comilla

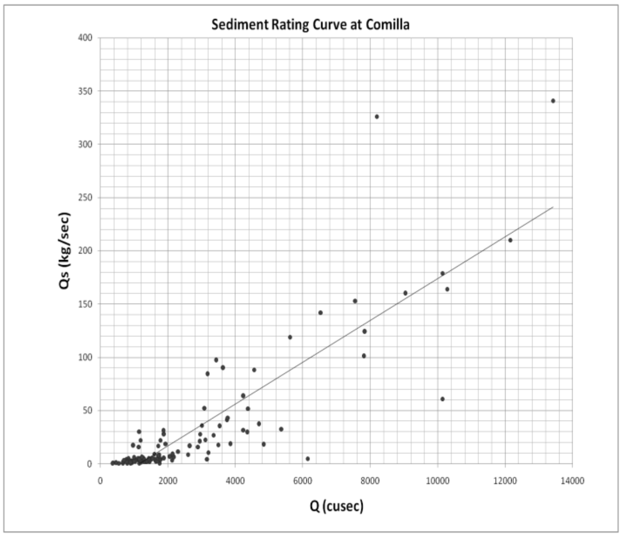

Fig 3: Relationship between discharge and total sediment load

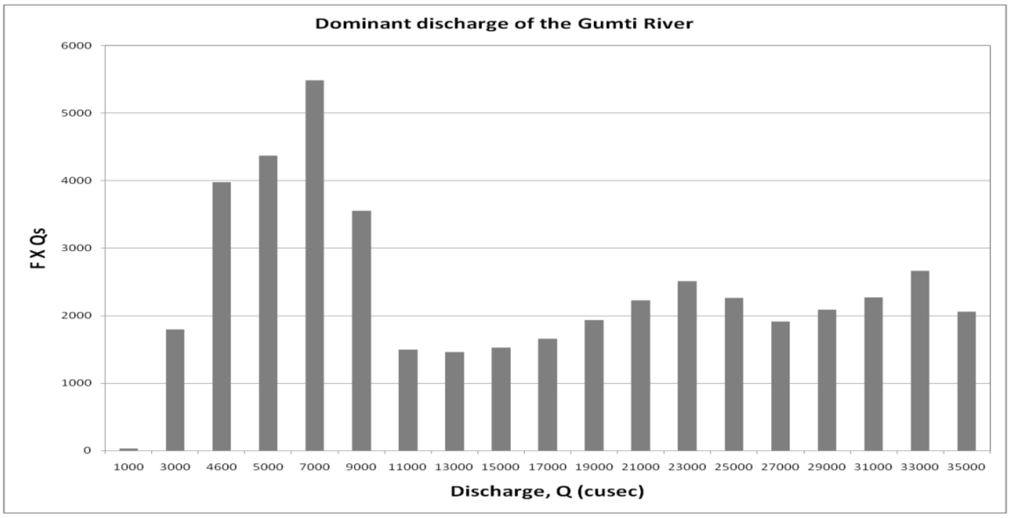

Fig 4: Dominant Discharge of the Gumti River at Comilla

From the histogram of Figure 4, the mode is identified which corresponds to the dominant discharge. According this concept, the bed generative discharge is found to be around 7000 cusec.

Stage-discharge curve

As mentioned previously, a definition of dominant discharge was equal to the bankfull discharge and in the stage discharge curve it is the point at which the rating curve exhibits an abrupt flattening in slope. When discharge increases beyond the effective flow, water begins to spill over the bank tops at more and more locations. The effective discharge should be compared to the bankfull discharge. This can be accomplished as well by identifying the bankfull stage during stream reconnaissance of the project reach and calculating the corresponding discharge.

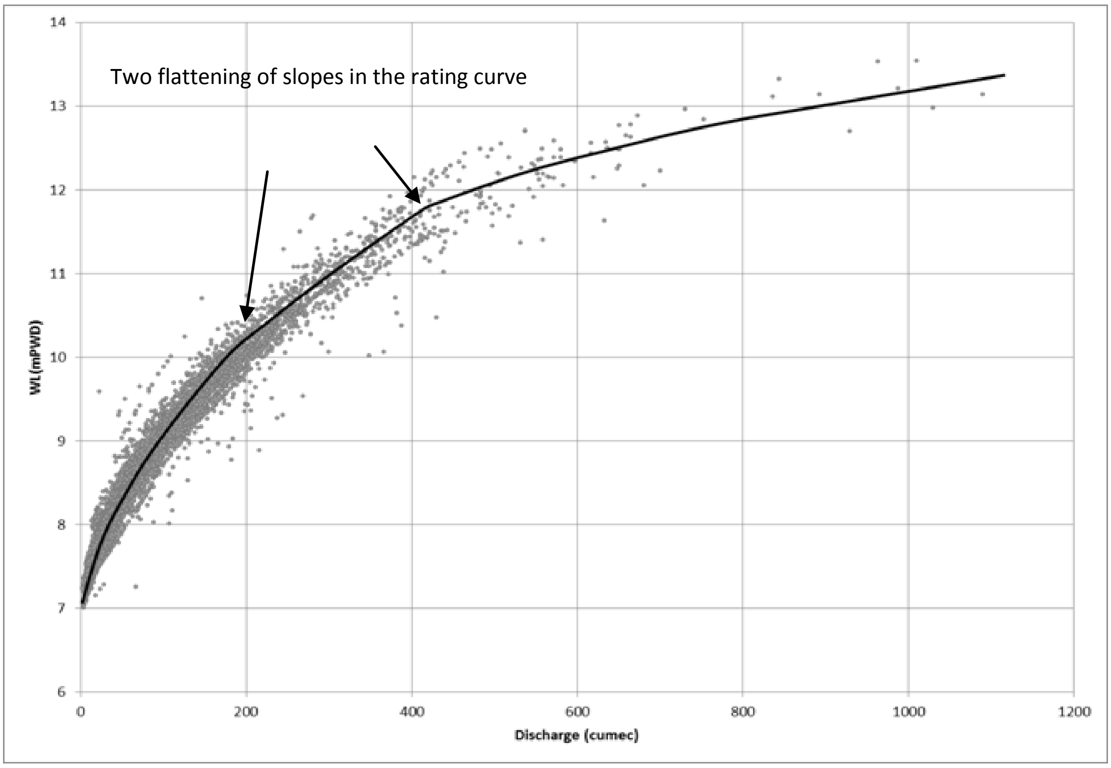

Fig 5: Rating curve for the Gumti River at Comilla

It is to be mentioned that, the left bank of the river section at Comilla is around 2 m higher than that of right bank. Because of this reason, stage-discharge relation at this station represents a 3-stage rating curve. However, for the consideration of dominant discharge, discharge corresponds to the lower bank level is taken here rather than the discharge corresponds to average bank level. From the rating curve of Gumti River at Comilla station, the flow for bankfull level was found to be around 200 m^3/s or 7000 cusec, shown in Figure 5.

Meander wavelength

Meander wavelength varies with the square root of bankfull discharge and any empirical relationship between wavelength (L) and bankfull discharge (Q_{bf}) may be statistical rather than causal (Dury, 1965). Wolman and Leopold (1957) concluded that bed width is determined directly by discharge, whereas wavelength depends directly on width and thus only indirectly on discharge. Then;

$$L = K.q^b \qquad (1)$$

Where L is meander wavelength, q is dominant discharge, K is coefficient and b is the exponent. The above parameters are in FPS unit.

Inglis (1949) found following relationship:

$$L = 36q^{0.5} \qquad (2)$$

Dury (1965) used a very large data set and found that

$$L = 30q^{0.5} \quad (3)$$

The above relationship was applied to the meander wavelength in the Gumti River. The numbers of selected bends were 28 and the average meander wavelength (L) was 2590.614 ft (Table 1). Therefore, bankfull discharge calculated by the Dury's method (equation 3) is found to be 7456.98 cusec. This discharge agrees closely with the discharge which was determined from stage discharge curve.

Table1 - Bend number along with their meander wavelength

Bend No	Wavelength(ft)	Bend No	Wavelength(ft)	Bend No	Wavelength(ft)	Bend No	Wavelength(ft)
1	5638.32	8	1918.8	15	2689.6	22	4569.04
2	3365.28	9	1354.64	16	2574.8	23	4496.88
3	3030.72	10	2643.68	17	2079.52	24	2407.52
4	2236.96	11	1403.84	18	1813.84	25	2030.32
5	2076.24	12	1256.24	19	1853.2	26	3972.08
6	2017.2	13	1623.6	20	1764.64	27	3873.68
7	2486.24	14	2168.08	21	2948.72	28	2243.52

Flood frequency analysis

Wolman and Leopold [13] recommended that bankfull discharge has an Average Recurrence Interval (ARI) of 1-2 years ($Q_{bf} = Q_{1\text{-}2years}$) on the annual series. Dury (1965) suggested that bankfull discharge is a discharge with ARI of 1.58 years or $Q_{bf} = 0.97Q_{1.58}$.

30 largest floods were selected for the flood frequency analysis in this study. Here the following probability distributions were used to determine the best probability frequency of the data.

1. Gumbel distribution function
2. Log-Normal distribution function
3. Log-Pearson distribution function

With the comparison of the Chi-square value, it was found that the best fitted frequency distribution to the data was Log-Pearson as it gives lowest value in Chi-square test. Therefore, the annual exceedence probability (AEP) of dominant discharge in Gumti River was found using Log-Pearson distribution.

Table 2 - Results of flood frequency analysis

Type of Distribution	Chi-square Value
Gumbel distribution function	3.1852
Log-Normal distribution unction	3.1852
Log-Pearson distribution function	2.8148

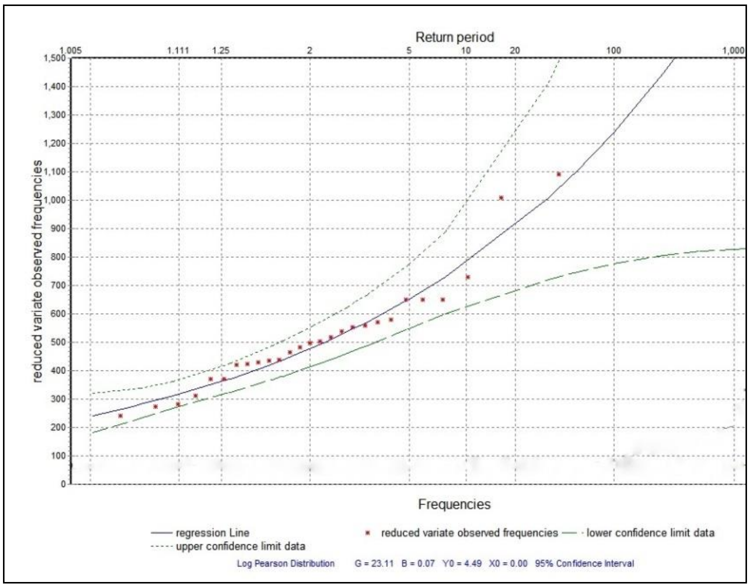

Fig 6: Flood frequency analysis

From the previous analysis, the value of bankfull discharge is found to be around 7000 cusec or 200 m^3/s. This agrees closely with the bankfull discharge and characteristic discharge which is determined from meander wavelength. Therefore the ARI of dominant discharge for the Gumti River was found from the Log-Pearson distribution and it was 1.005 years.

CONCLUSION

To predict the morphological changes of the alluvial river, the computation of the changes due to widely varying discharge is difficult. Although the use of a single discharge for the computation of the changes in the entire river regime may be questionable, but still, dominant discharge offers the advantage to correlate the average channel characteristics and help designing channel protection works. In this work, the bed generative discharge along with rating curve methods were used to determine dominant and bankfull discharge in the Gumti River. It was found that the average value of dominant discharge for the Gumti River at Comilla station may be taken to be 7000 cusec which has an ARI of 1.005 year.

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ID: WRE 011

NON-REVENUE WATER PREDICTION FOR KHULSHI AREA IN CHITTAGONG CITY

Md. Apel Mahmud[1*] & Aysha Akter[1]
[1]Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong – 4349, Bangladesh.
*Corresponding Author, <apl_047@yahoo.com>

ABSTRACT

Non-RevenueWaters (NRWs) is a common issue in a Water Distribution System (WDS) which pose challenges to urban areas in developing countries and the estimated NRWs were reported as 30% to 50% . In Bangladesh, WDS of some parts in Chittagong city area are facing problems due to NRWs and sometimes the amount even found up to 35%. Recently there was a pilot study conducted by Chittagong Water Supply and Sewerage Authority (CWASA) for their records. As the existing pipeline passed underneath the busy roads and also for relatively less noise, the acoustic leak detector was used for the pilot study normally during mid night. Thus, due to high cost involvement the existing survey was inadequate to provide frequent study based dataset. So, there is an urgent need to find a convenient field survey to supplement the existing dataset through acquiring information on influencing factors i.e. demand-supply relationship and the physical parameters of a WDS. This paper deals with NRWs prediction in Khulshi WDS and thus associated with relevant physical parameters of pipes (diameter, length, materials) using Global Positioning System (GPS) to establish a statistical relationship by carrying out weekly field study for NRWs. Methodologies involved with identifying about 170 meters in the study area for water flow measurement to compare with historical data collected by earlier pilot project. This is envisaged considering an intensive field survey, this paper would establish a relationship among NRWs and the relevant factors and the acquired relationship will be used for future decision making.

Keywords: Water Distribution System (WDS); Non-RevenueWaters(NRWs); Water Flow; Metering; Consumption

INTRODUCTION

In a water distribution system (WDS) it is a severe problem in developing countries and pose challenge for water utility companies to manage non-revenue waters (NRWs) while as a combination of poor infrastructures and poor operational practices is common (Islam and Babel, 2013). The difference between system input volume and billed authorised consumption is NRWs consists of: unbilled metered consumption and water losses. The estimating water losses in the world is around 30% (Feldman, 2009) is the major part of NRWs that leads the losses of distributed water through apparent loss such as unauthorized consumption or metering in accuracies and real loss such as leakage on transmission, distribution mains, service connection, or overflows in tanks. (Fig 1) (Lambert and Hirner, 2000).

Transient flow simulations with real-time at pipe inlet and outlet was developed for a single pipeline by Liggett and Li-Chung (1994) and later on practiced for leak detection by Silva et al. (1996). Leakage were modelled as an orifice by pressure dependent function for an effective optimization of leakage levels through allowing optimised number and location of control valves, as well as their opening adjustments. Thus leakage also studied considering as orifice by differen researches (Greyvenstein and Zyl, 2005). For leak detection through pressure control, parameters of different type, number and location of valves were used by Araujo et al. (2006). Thus also the roughness, number and location of valves, coefficient of head loss for economic and technically viable were considered during optimization model studies (Araujo et al., 2006, Mashford et al., 2009). The World Bank recommends that NRW should be less than 25% of the total water produced, while as the reported NRWs as 35% for developing countries (Islam and Babel, 2013). In Bangladesh, water distribution systems of some parts in Chittagong city area facing problems due to NRWs and the amount found up to 40% (JICA, 2014). So, it is needed to compensated the value within the reasonable limit a convenient field survey to supplement the existing dataset.

In some countries it has been recognized for many years that for an effective leakage management strategy effective management of pressures are the essential foundation whereas assessing real losses from water balance is the most basic and widely used method (Lambert and Hirner, 2000). The integral component of water supply, water demand management and loss determination is reliable metering and for water balance calculation metering of source for abstraction, input volumes and inflows to sectorised distribution systems, imported and exported water, and treatment works production is essential. Generally, customer meters generate economic revenue based on metered consumption, but the accuracy of these meters is also a key issue in water balance calculations. An efficient organisation need to recognise and deal with potential problems such as improper meter type or meter sizing, incorrect meter installation, meter encrustation, deterioration with age, flow rates less than the meter can reliably register, insufficient maintenance/replacement, frequency of calibration, inability to obtain readings, and influence of meter reading cycles (Lambert and Hirner, 2000). Though it is not possible in metering accurately for such activities as fire fighting, flushing etc., it should be estimated accurately each component of water use to determine realistic quantities for the water balance. (Lambert, 2002).

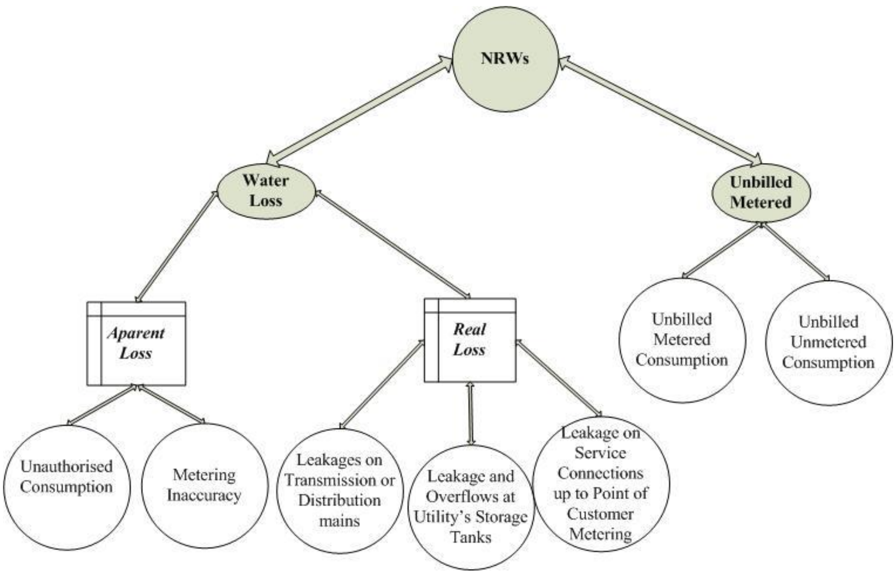


Fig 6: NRWs Component

MATERIALS AND METHODS:

Study Area

The study area, Khulshi is situated at 22˚ 21' 41" N to 22˚ 21' 59" N Latitude and 91˚ 48' 33' E to 91˚ 48' 52" E Longitude, consisting about 0.23 square kilometer area (Fig 2). In the existing WDS, there are 16 valves, 13 T joints, 5 end caps among total 2700 m distribution mains pipeline and the service connection length about 6700 m. The pipe materials are usually Poly Vinyl Chloride (PVC), Asbestos Cement (AC) and Galvanised Iron (GI) (Table 1). In an existing project by Chittagong Water Supply and Sewerage Authority (CWASA) NRWs data were collected for Khulshi area for a specific time duration (i.e. 2009 to 2013). In this reported ongoing research work thus a continuous secondary data is available for the model calibration.

Table 1: Physical properties of the WDS in Khulshi area (PANI, 2011)

Distribution main	Service Connection network	Total Length(m)	Comment
100mm AC	73% GI, 5% PVC and 22% (GI and + PVC)	1571	No leakage
100mm PVC	93% GI, 3% PVC and 4% (GI and PVC)	6457.1	1 leakage found
150 mm AC	GI and PVC	273	No leakage
200mm PVC	93% GI and 7% PVC	1009.6	No leakage
300mm AC	GI	132.5	1 leakage found

PVC = Poly Vinyl Chloride; AC = Asbestos Cement; and GI = Galvanised Iron

International Water Association (IWA) (Lambert, 2002) recommended for water balance calculation, (i) the essential first steps in the management of water losses, (ii) then the assessment and management of unbilled authorised consumption i.e., NRWs, and (iii) the assessment and management of components of apparent losses. In present study works metering inaccuracy is considered as a part of apparent loss as well as water loss.

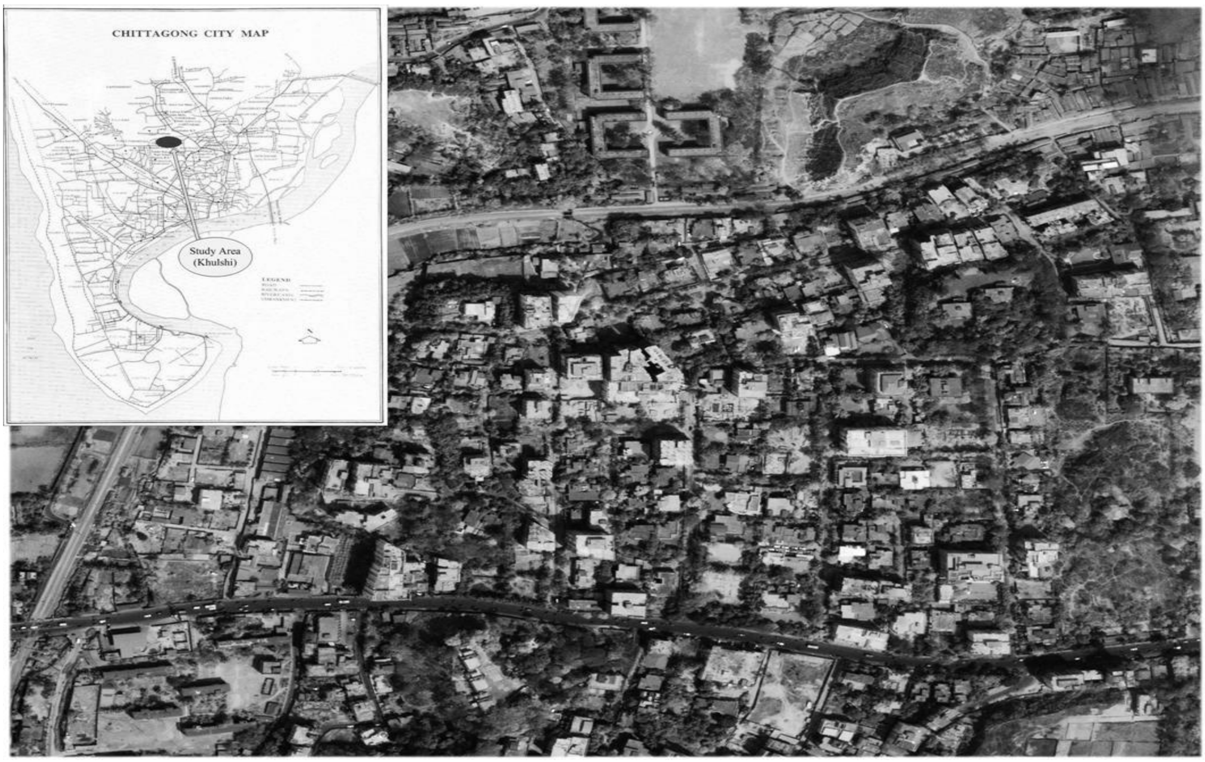
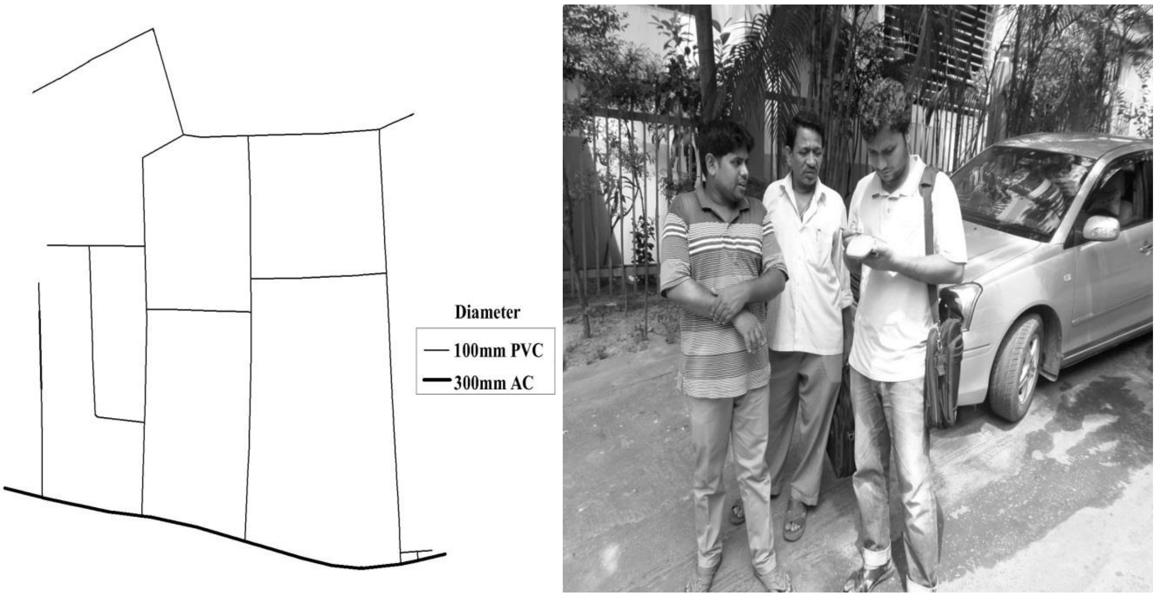


Fig 2: Study area



(a) Existing pipe network in the study area (b) Pipe layout checking using GPS

Fig 3: (a and b): Distribution main pipe network for Khulshi area by GPS

Stepwise works have been carried out as selection of a suitable study area was based on the available NRWs statistics followed by an intensive literature reviews. Then secondary data collection was carried out consisted of existing pipeline layout as well as details history on the studied WDS from the CWASA authority. A review has done on household metering along with pipe layout map and to view the present situation of this distribution system. Finally, Weekly field survey was focused on meter recording in cubic meter and physical properties of exposed pipe and valve for analyzing the consume volume with supply and thus calculating the NRWs. GPS information and meter reading were collected for all of the relevant records keeping (Fig 3b and Fig 4). During field survey, each household were marked with a serial number according to access road number. The difference between corresponding meter reading shows the consumption of water for the week. Then the supply value and the consumed value were calculated as a volume as well as in percentage with respect to supply.

Fig 4: Numbering and meter record keeping

FINDINGS:

The layout of distribution mains and maximum service connection pipe are in underground and these are expected to have an intensive check during dry season by an ongoing project. However, a part of this work showed quit reasonable information and the consumed details with acquired flow data during July to August 2014 (Table 2).

Table 2: Consumed details in Khulshi area WDS during July –August 2014

Description	**Field survey number**			
	1st	**2nd**	**3rd**	**4th**
Total consume in duration day (m3)	6958.4	4188.6	2132.3	2275.0
Total consume in per day(m3)	271.5	283.8	310.2	325.0
Total meter (no.)	168	168	168	168
Active metering (no.)	119	115	121	120
Flow per meter per day (m3)	2.3	2.5	2.6	2.7
Maximum consume per day (m3)	11.8	28.7	14.3	14.9
Comment: Few meters were inaccessible due to cover by soil or in under water and some were also missing those are reported to the concerned authority for immediate action.				

From the present analysis on these 4 cycle survey data of flow meter reading the average consumed value is found as 2.5 cubic meters per day (aprox). During this study around 17 meters were missing and among these 11 remain with same reading so as these might failed to record the flow passing through and the rest 6 showed negative consumed value. So far the field survey shows that the consumption in Khulsi area varies and the average consumption is about 53.14% of the supply, in contrast with this as time progresses water loss trends to decreasing trend (Fig 5). With respect to metering record more consumption minimize the water loss and so as NRWs and thus only during this period the amount reduces from 52% to 42% (Fig 5). So, there is an urgent need to ensure the metering accuracy for minimizing the NRWs.

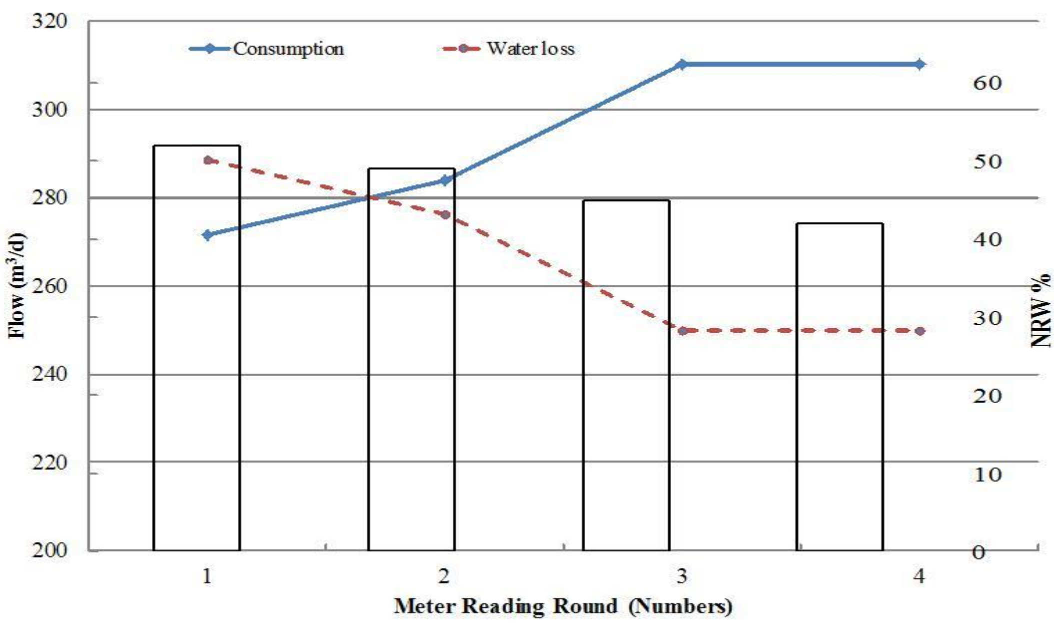


Fig 5: Relationship among NRW and the consumptions during July-August 2014

CONCLUSION:

Though Khulshi is considered a pilot area like Agrabad CDA, Halishahar B block, Chandgaon A and Chandgaon B block of Chittagong city CWASA also facing NRWs which subjected to unbilled metered, aparent losses and pipe leakages. Being a part of an ongoing research work, this paper aimed is to predict NRWs highlighted on metering inaccuracy as well as apparent loss and the calculated NRWs on an average was found as 47% during only July August 2014. Although the acquired NRWs exceed the acceptable limit, however an important relationship was observed among consumption, water loss and NRWs in a time space. As time progresses consumption increases and a proper metering can ensure the exact revenue and thus minimize the NRWs. These issues will be highlighted in the ongoing project work. This is expected that at the end of this research work it could be able to predict NRWs and analysis the relevent components with secondary data collected from CWASA.

ACKNOWLEDGMENTS

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A STREAM NETWORK APPROACH FOR HYDROLOGICAL STUDY OF A CATCHMENT

Ratul das*

Department of Civil Engineering, National Institute of Technology Agartala, Agartala, Tripura, INDIA,
<ratulnitagartala@gmail.com>
**Corresponding Author*

ABSTRACT

The primary objective of this study is to develop a computer aided stream-network model and to conduct a satisfactory flood hydrology study for the Kangsabati upstream catchment, located in the western part of West Bengal, India. Parametric study of this river-network model is performed based on the available meteorological data, observed floods and few severe rainfall events that have occurred over the years on Kangsabati catchment.Soil Conservation Service's (SCS) Curve Number (CN) method,in combination with Muskingum routing technique is applied for overland flow computations and flood routing. For processing the rainfall data and model simulation, Hydrologic Modeling System (HEC-HMS) simulation tool is used for two severe flood events to investigate the effect of watershed subdivision in terms of performance of the calibrated parameter. SCS loss method and Snyder's UH are used for rainfall-runoff simulation. The simulated results are found to be in good agreement with the observed values. Also, the present study confirms that the model input parameters such as sub-catchment area, average basin slope, channel slope, Curve Number (CN), and time of concentration used in this study can be effectively used for prediction of future hydrological events over the area. Moreover, the developed stream-network model can be applied to simulate the peak flood flow rate and flood volume from the upstream Kangsabati basin.

Keywords: SCS curve number, Muskingum routing, rainfall-runoff, stream-network.

INTRODUCTION

Partition of watershed is frequently used in many stream-network models to capture spatial variability of land cover and surface runoff. While using these models either at the time of designing or during actual operation, a certain level of accuracy and reliability is expected to describe the hydrologic response of the catchment. Also, delineation of sub-watersheds has the potential to affect model outputs. This study investigates the effect of watershed partitioning on model parameter values. Model parameters describe different hydrological processes and lead to a unique interpretation of the rainfall-runoff process. We analyse differences in such interpretations while simultaneously evaluating model performance with respect to peak flow magnitude and runoff volumes. Reviewing of related works indicate that in many cases the developed models offer satisfactory performance when data on the physical characteristics of the watershed are available (Line et al. 1997; Colby 2001; Miller et al. 2002). With the advancement of technology and research scope, Remote Sensing and Geographic Information System (GIS) made it easier to extract many land surface properties. Estimating direct runoff depths from storm rainfall by the United States Department of Agriculture (USDA) by curve number (CN) method (SCS 1972, 1985) probably the most widely used techniques. Many other researchers (Blanchard 1975; Jackson et al. 1977; Ragan and Jackson 1980; Bondelid et al. 1982) considered hydro-meteorological properties of the watershed and nature of land derived from satellite data and integrated them with GIS to estimate SCS CNs and runoff. Among many routing technique in river network modeling procedures the Muskingum method (Nash 1959; Overton 1966) probably the popular for flood routing. In Muskingum-methods of channel routing lesser number of data is required compared to other routing techniques like distributed kinematic wave flow routing. Gill (1978) and Luo, J. (1993) conducted a study to ascertain the impact of land use and management practices on rainfall-runoff relationship and used GIS techniques to route runoff through a watershed. Olivera and Maidment (1999) used a grid network for flood routing by employing the first-passage-time response function. In another study carried out by Swensson (2003) showed that Muskingum flow routing perform much better than distributed models such as Kinematic wave flow routing, when storage within the watershed is taken into consideration. Das (2004) developed an algorithm for parameter estimation that iteratively solves the governing equations to identify the Muskingum model parameters.

In hydrological analysis, infiltration losses and the hydraulics of the overland flow are of major concern. Overland flows and routing the flows through stream is a complex process. Hydrologic Engineering Centers Hydrologic Modeling System (HEC-HMS), developed by US Army Corps of Engineers is a widely used tool for simulation of overland flow process. In this paper we begin by introducing the study area, followed by a methodology and description of the HEC-HMS model and key parameters. Then, we discuss our stream network approach for watershed subdivision, the HEC-HMS model's calibration and validation. The subsequent sections explain the hydrologic simulations using two flood events observed in upper Kangsabati catchment, and the sensitivity of calibrated parameter values. We then discuss the interpretation of the simulated value with the observed value and end with a discussion of results and conclusions.

DESCRIPTION OF THE STUDY AREA

Kangsabati catchment is located in West Bengal, India, covering a geographical area of 3626 km^2 up to the dam sites. The basin is adjoined by Subarnarekha basin on the west and Damodor, Darakeshar, Sali, Rupnarayan basin on the north and the east, and Kalinghye basin on the south. Kangsabati is the major river which originates from Jabarband in the Hill of Chotanagpur range and traverses a length of 116.5 km up to the Kangsabati reservoir. River Kumari is the main tributary of the Kangsabati and joining the river on the right bank near Ambika Nagar in district Bankura, West Bengal. Bhairabbanki and Tarafeni are the other minor tributaries which meet the river on the right bank. On the left bank there is practically no tributary. The Kangsabati Dam is situated at Mukutmanipur on river Kangsabati (Longitude = 86^o 45' 30" N, Latitude = 22^o 7'30" E) as shown in Fig. 1. Average annual rainfall in the basin is recorded about 140 cm, the maximum being 182 cm observed in the year 1946, and the minimum was 96 cm occurred in 1947 and 83% of annual precipitation takes place during the monsoon months. During monsoon the average monthly rainfall in the months of June, July, August, September and October are 21 cm, 29.6 cm, 33 cm, 21.7 cm, and 10.6 cm respectively. From the available rainfall records it is observed that the rainfall exceeding 2.5 cm a day, generally occurs on 3 or 4 consecutive days and only in a few cases on 5 days whereas, in the lower catchment, total rainfall may be of the order of 18 cm to 20cm in a 3-day storms and 23 cm to 25 cm in a 4-day storm with maximum of 28 cm. The floods in the Kangsabati basin are flashy in nature and generally last for a short duration. During heavy storm, there may be two or more spells of rainfall in the same storm. On such occasion, flood may prolong and may have a high peak followed or preceded by lower peak or peaks. From the records it is observed that, the flood has duration of 3-days and discharge more than 566 m^3/s is considered as a flood in the study area. Also, frequency analysis shows that flood of 7590 m^3/s with volume of 7,70,653 km^3 has a returned period of 100 years, while flood of 6,788 m^3/s with a volume of 4,80,870 km^3 has a returned period of 50 years. The design flood for Kangsabati dam which is 10,620 m^3/s has a higher returned period and has not been exceeded even in 1978 during which peak discharge found to be 9,912 m^3/s and it is highest flood on record till date.

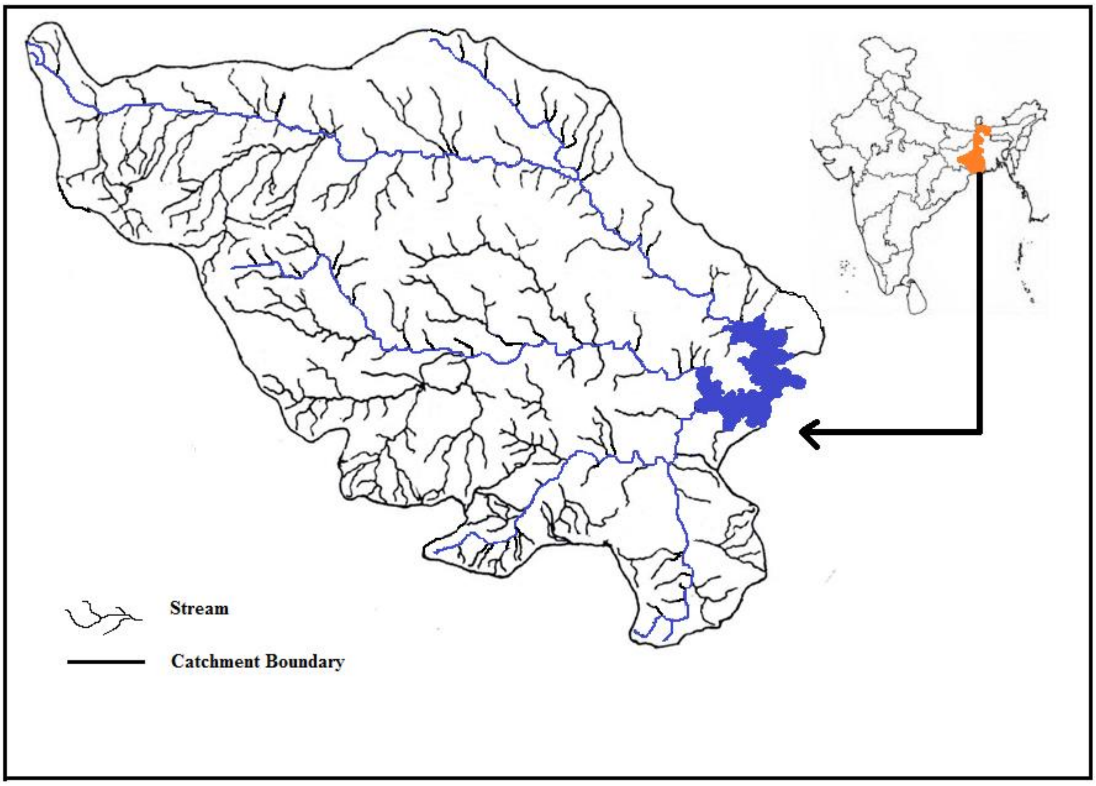


Fig.1. Kangsabati upper catchment

THEORETICAL FORMULATION

HEC-HMS is a physically based, semi-distributed hydrologic model developed by the US Army Corps of Engineers to simulate the hydrologic response of a watershed subject to a given hydrometeorological input. The HEC-HMS offers a variety of model options to simulate runoff production. These include SCS curve number, SCS unit hydrograph, and base flow separation methods which are necessary to calculate water losses, runoff transformation, and base flow rates. In the present study, the Muskingum and SCS loss method are used to calculate flood routing and water losses along the channel. The SCS loss model for basin loss is given by

$$P_e = \frac{(P-0.2S)^2}{(P+0.8S)} \tag{1}$$

$$S = \frac{25400}{CN} - 254 \tag{2}$$

$$I_a = 0.2S \tag{3}$$

where, P_e= runoff depth in mm; P = rainfall depth in mm; S = detention storage and; Ia = 0.2S = initial abstraction of rainfall by soil and vegetation. The CN value is a function of land use, soil type, and antecedent moisture. Using the tables published by the SCS, knowledge of the soil type and landuse, the single-valued CN can be determined. But for a river basin that consists of several soil types and land uses, a composite CN can be calculated by

$$\mathrm{CN}_{composit} = \frac{\sum A_i \mathrm{CN}_i}{\sum A_i}$$

(4)

where, $\mathrm{CN}_{composite}$ = the composite CN used for runoff volume computations; i = an index of sub divisions of uniform land use and soil type; CN_i = the CN for subdivision i; A_i = the drainage area of sub division i. In the present study, the methodologies adopted to convert the rainfall excess to the runoff hydrograph are (i) Snyder's Unit hydrograph and (ii) SCH Unit hydrograph methods whereas for routing the flood, Muskingum routing technique was adopted. The Muskingum routing technique is based on the continuity or storage equation in a river or channel and can be expressed by

$$\left(\frac{I_{t\text{-}1}+I_t}{2}\right)\text{-}\left(\frac{O_{t\text{-}1}+O_t}{2}\right)=\left(\frac{S_{t\text{-}1}+S_t}{\Delta t}\right) \tag{5}$$

where, I_t and I_{t-1} represent the inflow discharges, O_t and O_{t-1} the outflow discharges at section 1 and 2. S_{t-1} and S_t represent channel storages and ΔS is the increment or change in storage over time interval Δt. Muskingum model defines the storage as

$$S_1 = \mathrm{K}O_t + \mathrm{KX}\left(I_t - O_t\right) = \mathrm{K}\left[\mathrm{X}I_t + \left(1-\mathrm{X}\right)O_t\right] \tag{6}$$

where, K = travel time of the flood wave through routing reach; and X = dimensionless weight and ranges from 0 to 0.5. If storage in the channel is controlled by downstream conditions, such that storage and outflow are highly correlated, then X= 0.0. In that case, eq. (6) resolves to S = KO; If X = 0.5, equal weight is given to inflow and outflow, and the result is a uniformly progressive wave that does not attenuate as it moves through the reach. If eq. (5) is substituted into eq.(6) and the result is rearranged to isolate the unknown values at time t, the result is

$$O_t = \left(\frac{\Delta t - 2\mathrm{KX}}{2\mathrm{K}\left(1-\mathrm{X}\right)+\Delta t}\right)I_t + \left(\frac{\Delta t + 2\mathrm{KX}}{2\mathrm{K}\left(1-\mathrm{X}\right)+\Delta t}\right)I_{t-1} + \left(\frac{2\mathrm{K}\left(1-\mathrm{X}\right)-\Delta t}{2\mathrm{K}\left(1-\mathrm{X}\right)+\Delta t}\right)I_{t-1} \tag{7}$$

HEC-HMS solves eq. (7) recursively to compute ordinates of the outflow hydrograph, given the inflow hydrograph ordinates (for all t), an initial condition, and the parameters, K and X. Values of X vary from 0 to a maximum of 0.5 and the value of K is determined from eq. (7).

METHODOLOGY AND STREAM NETWORK BY HEC-HMS

Catchment boundary is traced from the topographical maps (Sheet No. 73 J & 73 I) collected from Survey of India (scale 1:50,000) followed by delineation of catchment boundary considering drainage density, basin slope, length of each tributary and the adjacent drainage basin. The delineated catchment is then compared with the river basin map obtained from department of Irrigation & Waterways, Govt. of West Bengal and adjusted accordingly. Total area of the upper catchment measured by a digital planimeter and found as 3626 km^2. As there are no storage structures or control structures in the upper catchment of Kangsabati reservoir, only two network components are considered: the sub-basin component and the channel component. Based on the topographical features of the basin, drainage density, land use pattern, soil type and rain gauge locations, the total upper catchment is divided into 25 (twenty five) sub-basins as shown in Fig. 2. Selection of the outlet points for each sub-basin is an important task and is done following the general stream network pattern traced from the toposheet. Once the outlet point is fixed, delineation of these sub-basin boundaries are performed following the procedure used for delineating the entire catchment, and with the help of other land marks like roads, canal layouts etc. After defining the sub-catchments, the streams and the junctions, the schematic network of the river basin is developed considering the hydraulics of flow for HEC-HMS model simulation. In this study the upper catchment of Kangsabati is divided into 25 sub-basin components; 14 channel or reach components (numbered as R-1, R-2, R-3, …, R-14), and 15 junction components. Fig. 2 shows the schematic of stream-network for the upstream Kangsabati catchment.

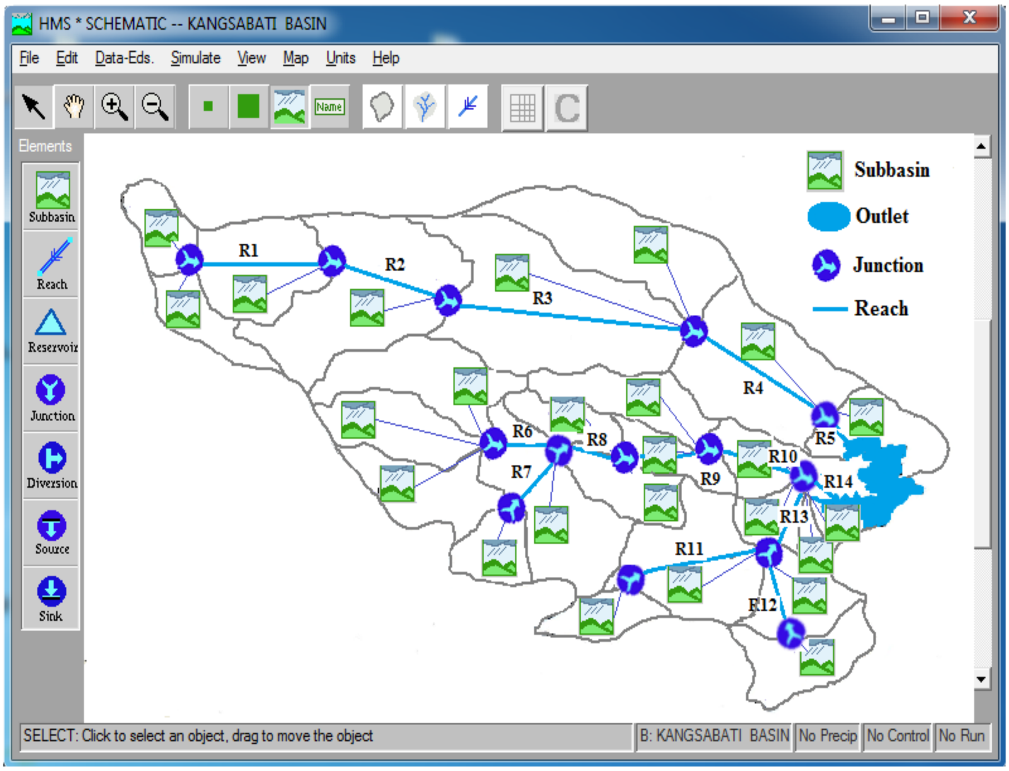


Fig.2. Stream Net-work of Kangsabati upper catchment

RESULTS AND ANALYSIS

Daily rainfall data for the upper catchment of the Kangsabati Reservoir are collected from the site office at Mukutmanipur of Irrigation and waterways department, Govt. of West Bengal, for all the four rain gauge station for the year 1997. Analysis of rainfall-runoff process developed in HEC-HMS is performed for a number of rainfall events. The analysis starts with the calibration of the model parameters as shown in Table 1. As this model is being developed for this river basin for the first time, it is essential to calibrate the model parameters with reference to observed outflow hydrograph in the study area. The next important step is to validate this model that is, with the developed model, another rainfall event is processed and resulting outflow hydrograph is compared with the observed outflow hydrograph for the same storm. Similar nature of two hydrographs in terms of peak flow, time to peak and volume of flow ascertain the satisfactory performance of the model. In the present study, validations were performed for two major flood events occurred on 23rd July and 6th August of 1997.

This section describes the processing of data and analysis of results with reference to the computations for the rainfall on 6th August 1997 and 23rd July 1997.

Event 1: From the records of reservoir operation data for 6th August 1997 at site, following observations were noted:
Average discharge through left bank feeder canal = 120.75 m^3/s.
Average Discharge through right bank main Canal = 14.53 m^3/s.
Peak outflow from the spillway = 990.60 m^3/s. (at 4.00 p.m.)
Total outflow = 1126 m^3/s.
During the peak outflow at 4.00 p.m., the reservoir outflow remained constant and started depleting after that. Hence, it may be assumed that, the corresponding peak flow at that time was 1126.0 m^3/s.
The stream-network model developed in HEC-HMS is used with the rainfall event on 6th August 1997. Initially, the model was run with the trial values of the model parameters. Both the Snyder's UH method and SCS lag methods were used separately. The trial values for these model parameters are selected from the available topographical and land use patterns. Outflow hydrograph at the catchment outlet as obtained from the first run is then compared for the flood peak and flood volume. The model parameters are then suitably modified for the observed discrepancies. The result of the final calibrated model is shown in Fig. 3. A comparison study is given in Table 2.

Event 2: From the records of reservoir operation data for 23rd July 1997 at site, following observations were noted:
Average discharge through left bank feeder canal = 79.84 m^3/s.
Average discharge through right bank main canal = 45.89 m^3/s.
Peak outflow from the spillway = 3383.03 m^3/s.
Total outflow = 3508.76 m^3/s.
The calibrated stream-network model is used with the rainfall event on 23rd July 1997. Here also, analysis is made using both SCS lag and Snyder's UH method. The result of the validation is shown in Figure 4. A comparison study is given in Table 3. The comparison study indicates a close similarity between the model results and corresponding observed values. Hence, the calibrated model is well accepted for future predictions.

Table 1. Calibration of parameters

Sub-basin	A (km^2)	L (m)	L_{ca} (m)	S (m/m)	t_c (hr.)	SCS Lag	t_{p1}	t_{p2}	t_r	C_p	CN	I (%)
S-1	67	12.80	5.70	0.03	1.95	1.17	1.36	1.38	0.25	0.31	45	20
S-2	78	15.33	9.63	0.01	3.00	1.80	1.68	2.09	0.31	0.37	45	30
S-3	217	16.22	8.62	0.01	5.78	3.47	1.65	2.77	0.30	0.37	45	30
S-4	300	24.20	6.34	0.01	11.85	7.11	1.70	3.51	0.31	0.38	45	30
S-5	442	28.89	17.23	0.01	11.15	6.69	2.42	4.98	0.44	0.52	40	35
S-6	187	20.15	12.17	0.02	6.29	3.77	1.95	3.29	0.36	0.36	40	20
S-7	227	20.02	11.02	0.02	9.59	5.75	1.89	3.90	0.34	0.38	40	35
S-8	154	15.71	8.99	0.01	6.17	3.70	1.66	2.90	0.30	0.38	40	20
S-9	137	17.11	11.28	0.01	7.96	4.78	1.82	3.59	0.33	0.33	42	30
S-10	70	13.31	7.10	0.01	3.48	2.09	1.47	2.00	0.27	0.27	42	30
S-11	168	16.35	11.21	0.01	7.54	4.52	1.79	3.49	0.33	0.32	42	25

S-12	60	14.32	10.52	0.01	6.47	3.88	1.69	3.16	0.31	0.31	42	25
S-13	130	18.50	10.01	0.02	8.69	5.21	1.80	3.58	0.33	0.32	40	20
S-14	90	10.90	7.86	0.02	3.61	2.16	1.42	2.12	0.26	0.27	40	20
S-15	130	19.77	3.04	0.01	12.30	7.38	1.28	2.70	0.23	0.24	42	20
S-16	75	27.24	13.69	0.01	10.71	6.43	2.22	4.47	0.40	0.39	40	20
S-17	164	7.34	4.31	0.01	2.04	1.22	1.06	1.27	0.19	0.21	40	30
S-18	178	12.94	7.10	0.02	5.76	3.45	1.46	2.57	0.26	0.32	40	30
S-19	45	5.20	3.29	0.01	2.62	1.57	0.88	1.30	0.16	0.30	40	25
S-20	118	8.24	4.44	0.01	3.42	2.05	1.10	1.66	0.20	0.21	42	25
S-21	185	11.53	7.09	0.01	5.04	3.02	1.41	2.40	0.26	0.26	42	25
S-22	145	20.40	8.24	0.01	6.01	3.60	1.74	2.77	0.32	0.22	42	30
S-23	158	16.09	7.35	0.01	5.67	3.40	1.57	2.58	0.29	0.25	40	20
S-24	55	8.87	5.20	0.01	4.64	2.78	1.18	2.05	0.22	0.21	40	20
S-25	48	10.64	6.34	0.01	4.57	2.74	1.33	2.19	0.24	0.24	40	20

Calibrated Data: *A* is the sub-basin area; *L* is the length of channel reach; L_{ca} is distance from the outlet to a point on the stream nearest to the centroid of basin; S is average sub-basin slope; t_c is time of concentration; t_p and C_p are regional constants; CN is Curve Number; and *I* is percentage of imperviousness.

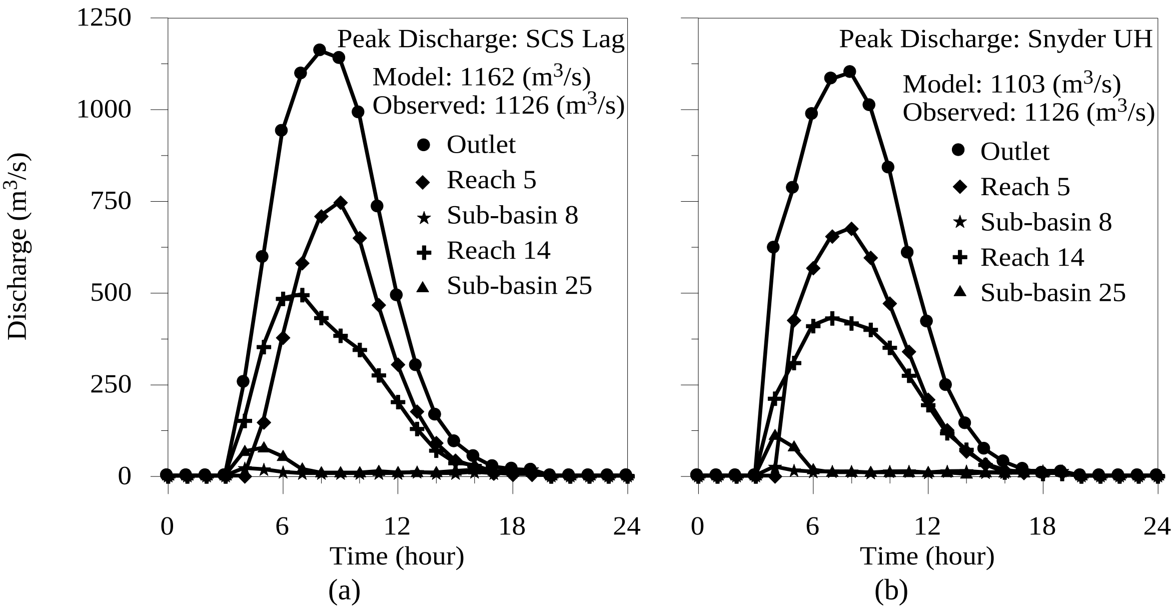
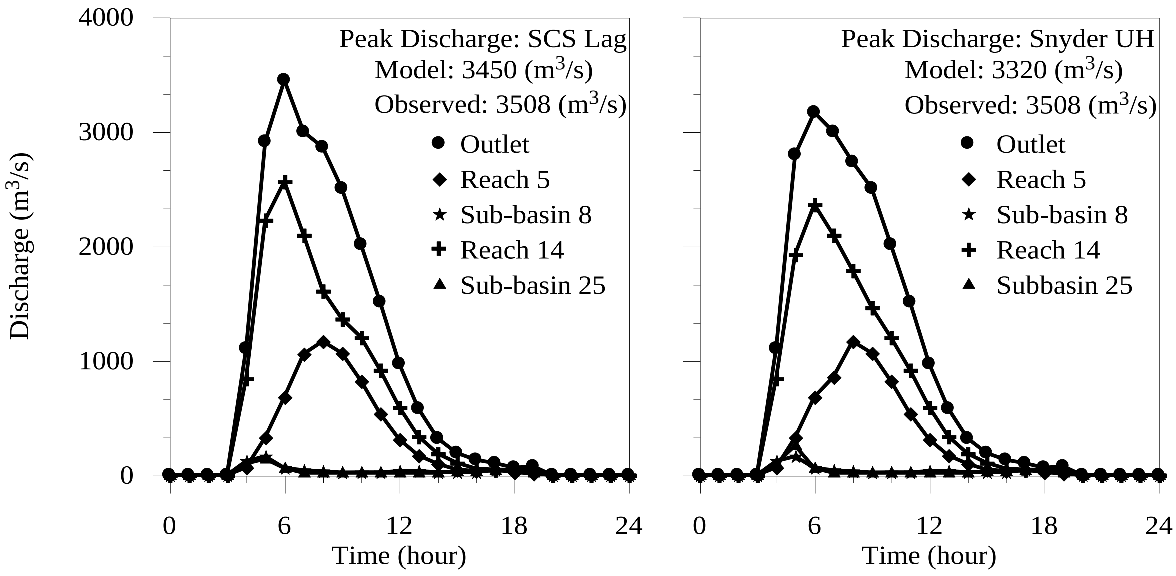


Fig.3. Hydrograph at the catchment outlet for storm of 6th August 1997: a) SCS UH method and b) Snyder UH method



(a) (b)

Fig.4. Hydrograph at the catchment outlet for storm of 23rd July, 1997: a) SCS UH method and b) Snyder UH method

Table 2. Comparison of simulated flood peak and flood volume with observed values

Event 1: 6th August, 1997

Method	Flood Peak (m^3/s)		Flood Volume (K.m^3)	
	Model	Observed	Model	Observed
SCS Lag	1161.2	1126.0	87904	87543
Snyders UH	1103.6	1126.0	87356	87543

Table 3. Comparison of simulated flood peak and flood volume with observed values

Event 2: 23rd July, 1997

Method	Flood Peak (m^3/s)		Flood Volume (K.m^3)	
	Model	Observed	Model	Observed
SCS Lag	3450.6	3508.76	226119	227812
Snyders UH	3320	3508.76	227213	227812

SENSITIVITY ANALYSIS

The Curve number (CN) is the main uncertain parameter that affects the result of simulation process. Selection of CN value depends on judgement about the watershed physical characteristics as well as its antecedent moisture conditions. A sensitivity analysis is performed to assess the effect of changing the SCS-CN values on the peak flow values. To support the results found from the simulation, both the calibration and validation model results were evaluated. The variation of runoff, peak discharge and outflow with CN value for both the rainfall event shows satisfactory results. Also the rainfall runoff correlations are 0.92 and 0.32 for the event 1 and event 2 respectively and found satisfactory as shown in Fig.6.

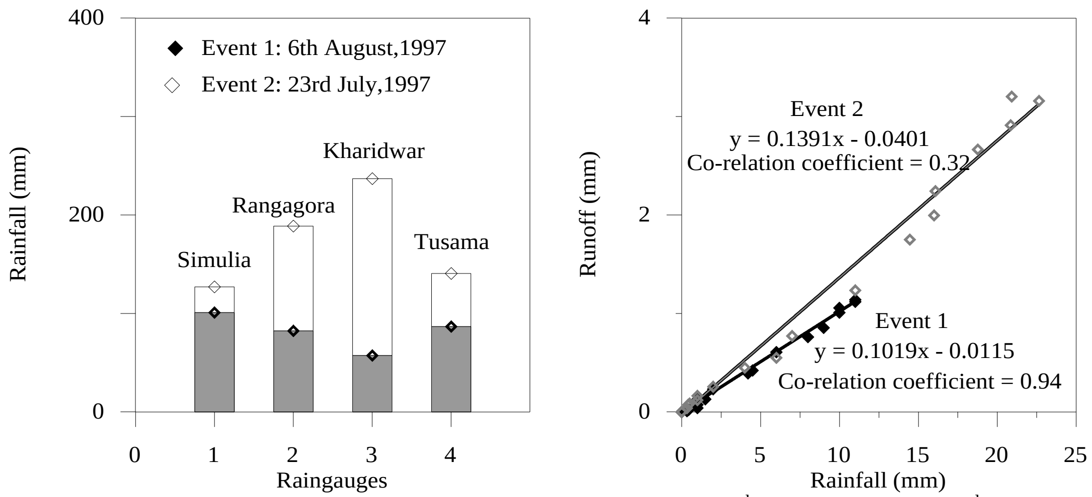


Fig. 6. Rainfall-runoff correlations for event1and event 2 (6th August, 1997, 23rd July, 1997)

CONCLUSIONS

Based on the above analysis and discussion, the following conclusions can be made.

The comparisons of peak flows and flood volume reveal that the developed stream-network model is quite capable to predict the outlet hydrographs of the Kangsabati catchment and the methodology applied here is acceptable for many hydrological studies. The present study suffered from the scarcity of field data. And more reliable observed data would have yielded better calibration, particularly for CN, K, and X parameters. Hence, for obtaining better results following suggestions are incorporated here.

(1) River stage-discharge curves at the junction points are very much required to develop any river network model. These values are significant to calibrate the Muskingum parameters.

(2) In stream-networks study, rainfall records in durations of 15 minutes at least are desirable and records of actual durations of the storms are very important. Hence, recording type rain gauges and corresponding recorded rainfall mass curve may provide precise duration of the storm event and information on variation of rainfall intensity.

(3) Landuse map and soil characteristics of the study area and adequate information of infiltration may help is assessing proper values for CN.

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EVALUATION OF CLIMATE CHANGE SCENARIOS OF UPPER MEGHNA RIVER BASIN USING HYDROLOGIC MODELING SYSTEM (HEC-HMS)

M. M. Ali[1*], A. Narzis [1] & S. Haque[1]

[1]*Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.*
**Corresponding Author, <amostafa@wre.buet.ac.bd>*

ABSTRACT

Hydrologic models have emerged as a basic tool for studying real processes in a watershed hydrologic system and systems responding to various climatic forcing. Bangladesh has been formed as the greatest deltaic plain at the confluence of the Ganges, Brahmaputra and Meghna and is highly vulnerable to climate changes according to IPCC report (IPCC 2007 and IPCC 2013) and many other studies. To understand the consequences of climate changes, hydrological study of these three river basins is required. In this study, as an initial project, a hydrological model of Upper Meghna river basin with drainage area of 70263 km^2 is developed using HEC-HMS. HEC-HMS is a semi-distributed hydrological model that can be used to simulate precipitation-runoff process for both event based and continuous precipitation. The model developed in the study can be used as a tool to understand the effects of human intervention and changed climatic condition in the basin area. Effects of climate changes are simulated by running the model using the future precipitation data obtained from Global Climate Models (CSIRO-30 and CCCMA-31). Results from those model data are used to predict flow hydrographs for the years of 2050 and 2080.

Keywords: Upper Meghna Basin, Climate Change, Hydrological Modeling, HEC-HMS, Global Climate Model.

INTRODUCTION

Changes in future climate will have an immense impact on agricultural production, food security, ecology, biodiversity, river flows, floods and droughts, water security, human and animal health and sea level rise of Bangladesh. Uncertainty remains in the prediction of future scenarios due to the diversified climate of GBM (Ganges-Brahmaputra-Meghna) basin. Climate change may alter the distribution and quality of GBM river basin water resources which may lead to severe natural hazards in Bangladesh. So it is necessary to evaluate the impacts of potential climate change scenarios which are often evaluated by combining atmospheric and hydrologic models.

Several studies have been carried out by the researcher using different hydrological models. It has been seen that hydrological models are better representative of river network and give more accurate results in the estimation of its parameters. GBHM is an example of fully distributed physical hydrologic model which is used to estimate the future rainfall and temperature values in Meghna river basin by Alam *et al.* (2011). Rahman (2012) also developed a hydrological model for GBM basin using SWAT model. HEC-HMS is a widely used semi distributed physical hydrological model that simulates precipitation-runoff processes for a watershed. Specific capabilities include the ability to specify losses and precipitation volumes for each sub-basin within a watershed, methods to transform precipitation excess to direct runoff, stream routing options, and parameter optimization techniques. Therefore, HEC-HMS is considered in this research to study the highest precipitation zone in GBM delta. Upper Meghna river basin is selected as the study area in this research. The prime concern of the study is to develop a semi-distributed hydrological model in HEC-HMS and use the model to predict future climate change scenarios for the years of 2050 and 2080.

STUDY AREA AND DATA COLLECTION

The Upper Meghna Basin occupies total area of 82,000 km2. 47000 km2 (57% of total area) and 35000 km2 (43% of total area) are contributed by India and Bangladesh respectively as shown in Fig. 1. About 0.4% of the area of Southeast Asia is covered by this study area.

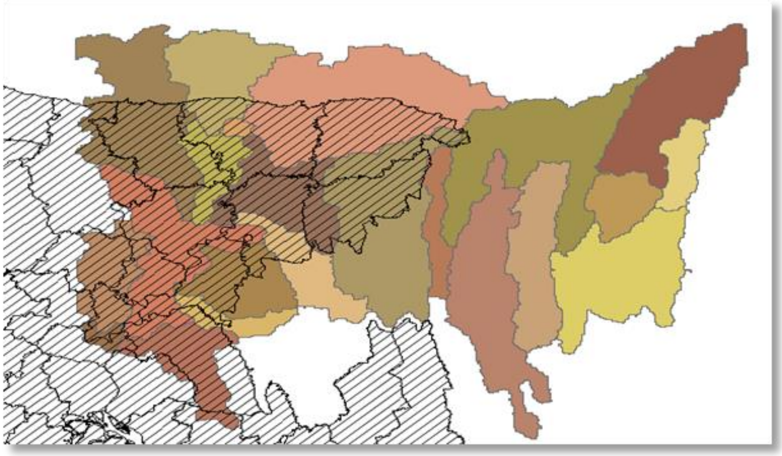

Fig. 1 Upper Meghna River Basin (Colored Area)

Digital elevation model (DEM) and stream network of the study area (resolution- 30s) are collected from the HydroSHEDS webpage of U.S. Geological Survey (source: http://hydrosheds.cr.usgs.gov/index.php). Land use and soil data are collected from GlobCover (source: http://due.esrin.esa.int/globcover/) and FAO (source: http://www.fao.org/climatechange/54273/en/) respectively to generate Curve number grid for each subbasin. The resolution of soil data and land use data are 1:5,000,000 and 1000 m respectively. The reference system is WGS1984.

Precipitation and discharge data of daily interval are collected for 1990, 2005 and 2006 time period. Rainfall data of eight rain gages station (Joydebpur, Lakhsam, Netrokona, Sylhet, Narayanganj, Sreemangal, Moulvibazar and Narshindi) located within Bangladesh and Discharge data at Bhairab Bazar are collected from BWDB. For Meghalaya, Silchar and Agartala station (within India), rainfall data are collected from NASA (Prediction of Worldwide Energy Resource).

Percentage change in Rainfall of 2050 and 2080 for two Global climate models, CSIRO-30 and CCCMA-31 are collected from Climate Wizard (source: http://www.climatewizard.org/) which is an online database source according to the latitude and longitude of the rainfall stations.

MODEL SETUP

Hec-GeoHMS 5.0 is used to delineate stream network and 22 sub-basins of Upper Meghna River basin through ArcGIS 9.3. HEC-HMS 5.0 is a hydrologic modeling software which includes many of the well-known and well applicable hydrologic methods to be used to simulate rainfall-runoff processes in river basins [USACE-HEC, 2006]. Only rainfall events are considered for performing precipitation runoff simulation in this study.

Basin Model

The whole basin was subdivided into twenty-two sub basins. Precipitation data was given at 11 gauge stations. Basin model (Fig. 2) with sub-basins and river network shape files are created through Archydro tools and Hec-GeoHMS in ArcGIS 9.3. Curve numbers values for each sub-basin are found to range from 71 to 100 which are generated by merging soil and land use data.

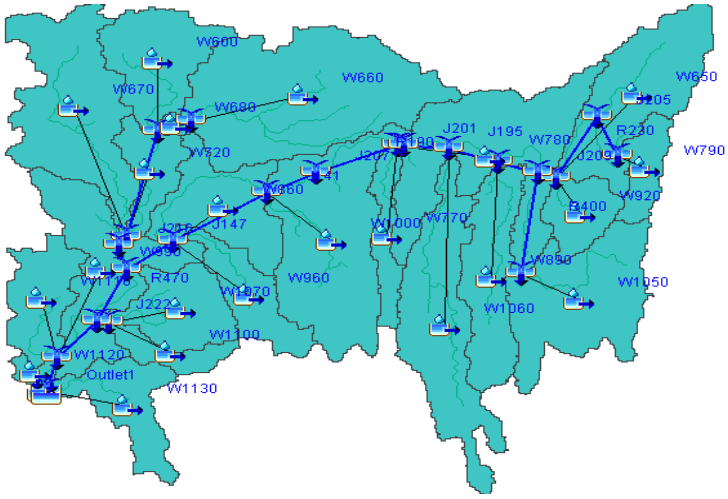

Fig. 2: Basin Model

Meteorological and Time Series Data

For each sub-basin, names of rainfall station, depth weights and time weights are added manually. Depth weights are calculated in GIS by the method of Thiessen polygon. Time series data of 11 precipitation gage stations are prepared as DSS files through HEC-DSSVue. Rainfall data of daily time interval with metric unit system is used as time-series data.

Model Parameters Calibration and Validation

The hydrological model is calibrated against the observed data at Bhairab Bazar station for the year of 2005 and validated for the year of 2006 at the same station. Though, hydrologic model is usually calibrated for longer periods, as an initial project and due to unavailability of the observed data of low flow months (December to April), only high flow months (May to November) are selected as a calibration period. The validation is also done for the same months of year 2006. Therefore, with this model, impacts of climate changes on low flows cannot be modeled or predicted. The final calibrated parameters are shown in Table 1 an final calibrated and validated hydrographs are shown in Fig. 3 and Fig.4. Different statistical parameters, e.g., NSE, RSR and PBIAS (according to Moriasi et al., 2007), for calibration and validation graph are also calculated and shown in Fig 3 and Fig.4. The statistical parameters are within the permissible limits as specified according to Moriasi et al., 2007.

Table 1: Final Calibrated Parameters

Elements	Parameters	Initial	Final
For all Subbasins	Lag time, tp	18.86	144
	Peaking coefficient, Cp	0.5	0.22
	Initial Discharge	10	100
	Recession constant	0.11	0.91
	Initial Abstraction, Ia	0	0.15488 to 0.15509
For all Reaches	Muskingum K	6	24
	Muskingum X	0.45	0.45

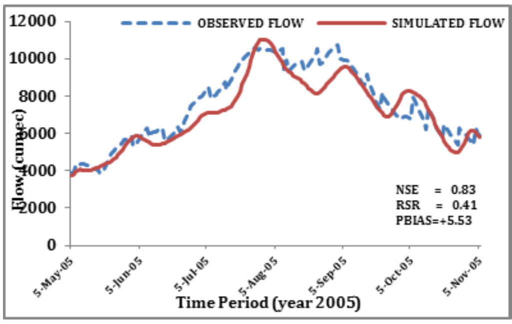

Fig.3: Calibration Graph

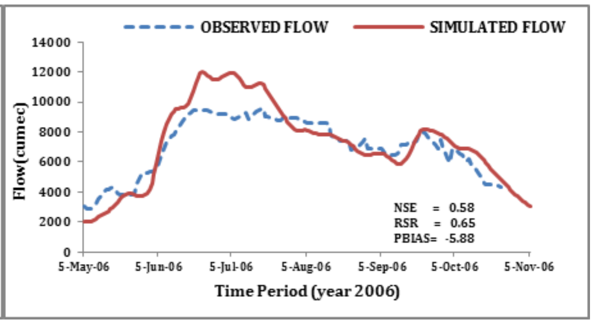

Fig.4: Validation Graph

RESULTS AND DISCUSSION

To study the effect of future climate change scenarios, the calibrated model is used to simulate flow hydrographs for the years of 2050 and 2080. Precipitations form GCM models (CSIRO-30 and CCCMA-31) are collected as a percent change in the future precipitation with respect to the baseline period of 1961-1990. A1B emission scenario is chosen in this study to predict future runoff. Part of Kishoreganj, Narshindi and Brahmanbaria comprises sub-basin W990 and sub-basin W1110 includes part of Narshindi and Narayanganj. Junction J144 is the calibration point which is located at the Bhairab Bazar.

Simulation Results

Table 2 is the representation of simulation results at Junction J144, contributing sub-basins and reach in inflow at J144 for 2050 and 2080. The following results are concluded-

- For the predicted precipitation as CSIRO-30 simulated HEC-HMS peak outflow at J144 will be decreased by 4.5% in 2050 and increased up to 17% in 2080. According to CCCMA-31, the peak flow will be increased up to 18.5% and 21% in 2050 and 2080 respectively. Percent changes are calculated with respect to the observed data (2005).

- Flow in sub-basin W990 decreases (up to 81.3%) almost in every month in the wet season. But the overall outflow at J144 increases (up to 58.83%) as an increase in flow in W1110 in the year of 2050 and 2080. A summary of results using both GCM models are shown in Table 2.

Table 2: Simulation results in HEC-HMS using both GCM model results

Year		**2050**		**2080**	
GCM Model		**CCCMA-31**	**CSIRO-30**	**CCCMA-31**	**CSIRO-30**
Peak Discharge/Peak Outflow (m^3/sec)	Junction J144	12724.5	10253.5	12990.75	12606.3
	Sub basin W990	571	533.5	460.0	454.2
	Sub basin W1110	273.4	289.8	315.9	288.0
Precipitation (mm)	Sub basin W990	1505.54	1554.94	1792.71	1734.53
	Sub basin W1110	1401.43	1414.83	1962.44	1706.47
Loss (mm)	Sub basin W990	47.85	48.09	47.89	48.05
	Sub basin W1110	40.35	40.68	40.36	40.55
	Junction J144	06 September	07 July	06 September	06 September
	Sub basin W990	02 September	05 July	02 September	02 September

Date of occurrence of peak	Sub basin W1110	04 July	04 July	04 July	05 July

Comparison of HEC-HMS Flow Hydrographs for Both GCM Model Results

The flow hydrographs (year-2050 and 2080) obtained from HEC-HMS model simulation at J144 for both GCM models are compared in Fig. 5. Results from both the GCM as percent change in precipitation are used to calculate future precipitation. In addition to graphical comparison, corresponding peak values and percent change in volume of each hydrograph are also represented. Percent change in total volume is calculated with respect to latest available observed data (2006). The results show that expected increase in total volume (wet period) using CCCMA-31 and CSIRO-30 precipitation will be in between 29%-54% and 50% -54% for the years of 2050 and 2080 respectively.

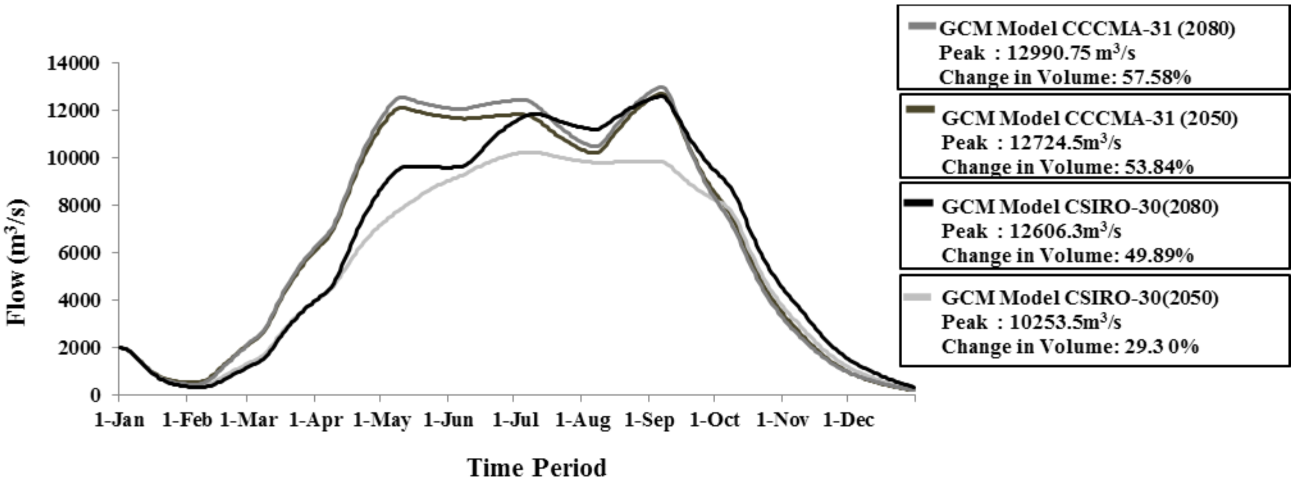

Fig.5: HEC-HMS flow hydrographs, peak flows and percent change in total volume

Changes in Subbasin Flow

Percent changes in subbasin flow are also analyzed for the fifteen subbasins that are within Bangladesh. Fig. 6 shows only the percent change in subbasin flow for the month of occurrence of peak (September) at J144 in 2050 and 2080. For example, according to both models, in September 2050, W990 will receive a decreasing flow up to 85%. However, for 2080, 25% decreased flow is

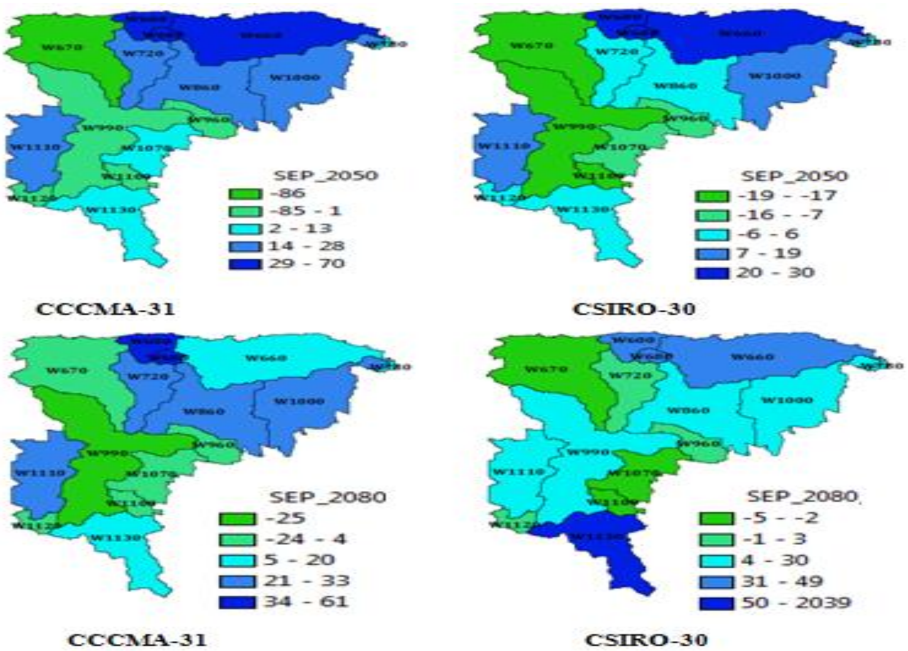

Fig. 6: Percentage change in sub-basin flow in September

predicted with CCCMA-31, whereas 30% increased flow is predicted with CSIRO-30 for this sub-basin. For the sub-basin W1110, model is predicted an increasing flow using both GCM model results. For year 2050 and 2080, flow will be increased up to 28% and 49% respectively. For results of other months are available in the M.Sc. thesis of the last two authors (Haque and Nazris, 2014).

CONCLUSION

Hydrological model of Upper Meghna river basin using HEC-HMS is successfully developed in this study. The delineated watershed of the basin using Arc GIS and HEC-GeoHMS has been an aid for the successful representation of basin hydrology. Model parameters are calibrated and the calibrated model is then used to evaluate climate change scenario in the year of 2050 and 2080. In this study, observed flow data at Bhairab Bazar was not available for dry period, so the model is calibrated mainly considering the flows for wet period. Peak outflow from the calibrated model is 11034.8 m^3/sec which has an estimated error of 3.03% in peak flow. Percent change in precipitation used relies on climate change scenario selected and GCM model performance. The results show that expected increase in total volume (wet period) with CCCMA-31 and CSIRO-30 precipitations will be in between 29%-54% and 50% -54% for the years of 2050 and 2080 respectively. Several studies can be made using this model in future. These includes, but not limited to, effects of landuse on flow, effects of any upstream development such as construction of dam, urbanization etc.

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DEVELOPMENT OF A DESIGN TOOL FOR SIZING STORAGE TANK OF RAINWATER HARVESTING SYSTEM IN COASTAL AREAS OF BANGLADESH UNDER FUTURE CLIMATIC SCENARIO

M.M. Islam[1*], S. Afrin[1], R. Jadid[1] & M.M. Rahman[1]

[1] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh*
**Corresponding Author, <maksimulislam075@gmail.com>*

ABSTRACT

As Bangladesh has tropical monsoon with large seasonal cycle in rainfall, Rainwater Harvesting (RWH) system and its utilization could be a good and environmentally sound solution for providing safe drinking water. The design of storage tank of RWH deals with a number of varying factors such as rainfall intensity, per capita water consumption, household size, material, pitch and area of roof, economic capability of the household, design period etc. As rainfall intensity is expected to be altered spatially and temporally in future under the effects of climate change, sizing of storage tank depends on climate change also. This study aims at developing a simulation model as design tool incorporating all the design variables and also climate change variability of our country i.e. to develop a demand responsive design tool. This simulation model is developed based on mass balance equation in which predicted future precipitation intensities are used. As the performance of rain water harvesting system is generally described in terms of reliability, in this study reliability of the system is also determined in terms of failure months and incorporated in the design tool.

Keywords: Rainwater harvesting, Climate change, Mass balance equation, Reliability, Storage Tank

INTRODUCTION

The impact of climate change on the availability of fresh water in Bangladesh is a major concern (Rajib et al., 2012). Safe drinking water is considered indispensable for advancement of any society. But increasing inadequacy of safe drinking water resources has made it necessary to think about the possible solution to cope with the crisis. Indiscriminate pollution of surface water sources (rivers), unhygienic activities polluting pond water in rural areas, high salinity in surface and groundwater in coastal belt, absence of good groundwater aquifers in hilly areas, difficulties in tube well construction in stony layers, arsenic contamination of groundwater, depletion of groundwater table due to overexploitation are the main constraints of developing a reliable water supply system in Bangladesh. As Bangladesh has tropical monsoon with large seasonal cycle in rainfall, Rainwater Harvesting (RWH) system and its utilization could be a good and environmentally sound solution for providing safe drinking water (Pandey and Andersion, 2003). In the coastal areas, scarcity of potable water is very acute (Kamruzzaman and Ahmed, 2006; Islam et al., 2011) as suitable aquifers at shallow depths are rarely available and surface water especially the river water is highly saline and turbid. Rainfall in the coastal areas is much higher and roof catchments are suitable (Karim, 2010); thus rainwater harvesting has a good potential to supply drinking and cooking water in the coastal areas of Bangladesh.

According to the third assessment report of Intergovernmental Panel on Climate Change (IPCC, 2001), developing countries are expected to suffer the most from the negative impacts of climate change. Changes in rainfall pattern are likely to lead to severe water shortages and flooding. The IPCC Special Report on the Regional Impacts of Climate Change (IPCC, 2007) indicates that there would be drastic changes in the rainfall patterns in the warmer climate and Bangladesh may experience 5-6% increase of rainfall by 2030, which may create frequent massive and prolonged floods. In contrary, the country is facing drought, which affects agriculture, food production, water resources and human health. Due to the variability of rainfall under a climate change scenario, the rainfall harvesting units designed according to the present rainfall records may face large uncertainties in providing adequate storage quantities. The study aimed at developing a systematic design tool for coastal areas considering precipitation variability due to climate change.

MATERIALS AND METHODS

Study Area

As potable water is scarce mainly in coastal areas of Bangladesh (Kamruzzaman and Ahmed, 2006; Islam et al. 2011), this study aims at developing the design tool for a coastal division named Khulna. Tube wells here get contaminated with high saline water. Pond water is available and comparatively less saline but turbid, colored and contaminated by pathogenic micro-organisms. Moreover, during cyclone or flood disaster, sea water enters into the ponds that are used for Pond Sand Filters (PSF) and damage the whole systems (Rahman et al., 2012).

System Simulation

Numerous methods are available for determining the size of the storage capacity required to satisfy a given demand. These methods vary in complexity and sophistication. They can be categorized as: graphical, mass curve, statistical, and simulation methods. Graphical and mass curve methods that can be used for rapid assessment are designated as preliminary design techniques. A statistical approach is sometimes adopted to determine the relationship of the capacity of a large reservoir with its inflow and potential releases (Tsai, 1996). Many researchers have used simulation (Liaw et al., 1997) to investigate the performance of rain water systems. In a simulation analysis, the changes in storage content of a finite capacity are determined using a mass balance equation. The procedure takes into account serial correlation and seasonality and applies any time interval. For most RRWHSs, the amount of rain water supplied depends on the quantity of rainfall, the area of the roof, and the calculated yield.

A simulation method has been developed in this study based on mass balance equation, which is used to determine the effective tank size and also to investigate the performance of rain water harvesting systems. The principles of mass balance equation can be illustrated mathematically as,

$$\mathbf{0 \le V_t = (V_{t-1} + Q_t - D_t) < V_s}$$

Where, V_t, is the volume of water in the tank at present, V_{t-1} is the volume of water in the tank remained from previous time step, Q_t is the rainwater captured at present, D_t is the total consumption per month and V_s is the volume of the tank.

The model was implemented in MATLAB which is a high level technical computing language and interactive environment for algorithm development, data visualization, data analysis, and implimenting numerical methods. For different water demand, roof area was varied and the volume of the storage tank and the corresponding values of reliability were recorded and used in plotting the Tank Sizing Curves (Fig. 1). The model development involved introduction of the input data and assumptions in the MATLAB environment and extraction of the data points to represent the model in the curve/graphical format. The model result is represented in curves so that they can be used by people who lack knowledge of hydrology and probability.

Sensitivity analysis

A sensitivity analysis was performed to assess the effect of major parameters on rain water harvesting system. The major parameters affecting storage tank and reliability are roof area, household size and water demand.

Reliability analysis

The reliability concept is important for the rainwater harvesting system as the performance of RWH system is generally described in terms of reliability (Karim et al., 2013; Liaw and Tsai, 2004). Reliability is defined as the probability that a given size of rainwater harvesting system will be sufficient to supply the necessary amount of water. Reliability (Re) can be illustrated mathematically,

Re = 1 - n/N

Where, n is the number of time units when demand exceeds storage; and N is the total number of time units in the rainfall sequence.

Simulation period starts in January 2014 with zero initial storage in the tank. It should be noted that the storage in the tank was not allowed to be negative and also overflow is allowed so volume of water in tank cannot be greater than volume of the tank. The number of months with zero volume (V_t = 0) of water in the tank was counted as failures.

Data

The model development required data as inputs or assumptions as the boundary conditions. These data are water demand, roof area, rainfall data, and runoff coefficient. The household water demands considered are 15, 30, 45 and 60 L/ capita/ day. There is no data available on the average roof size in Bangladesh. However a limited field survey was carried out by Ferdausi (1999) in rural areas and based on her findings Roof areas of 20, 30, 40, 50, 70, 90 and 100 m² are considered. Runoff coefficient of 80% is used due to assumptions that rainwater is lost due to first flush water and leakages in the systems. There is insignificant difference among the end results for variation of this parameter in the range of 0.75 – 0.85 (Yusuf, 1999).

The secondary data were future predicted monthly rainfall data. In order to estimate the storage volume requirement under the climate change regime, projections of future precipitation for the period of 2014 – 2050 from a climate model is used (Table -1). The future precipitation scenario was predicted by PRECIS, a regional climate model (RCM) system developed by the Hadley Centre of United Kingdom, for different areas of Bangladesh in a previous study (Rajib et al, 2011). PRECIS was basically adapted for generating projections of some specific climatic parameters for Bangladesh, side by side with GCM projections.

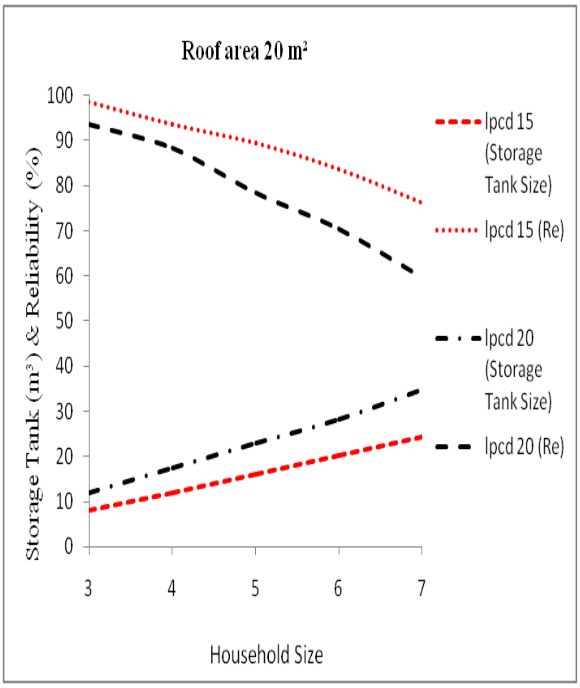
Table-1: Statistics of the predicted future monthly rainfall data of PRECIS model for Khulna for the period 2014 to 2050.

Statistics	Value	Unit
Number of values, n	444	-
Maximum values	3494.4	mm
Minimum values	0	mm
Mean, μ	273.6	mm
Standard Deviation, σ	441.8	mm
Coefficient of variation, CV = σ/ μ x 100%	161.5	%
Standard error of mean, SEM = σ/sqrt (n)	21	mm

RESULTS AND DISCUSSIONS

Sensitivity of major parameters

Fig. 1 shows the effect of household size on storage tank size and reliability for different roof area. Storage tank size shows an upward trend with increse in household size whereas corresponding reliability shows an decreasing trend. Storage tank size and corrsponding reliability exhibits opposite trend with increase of water demand as household size (Fig. 2). It is observed that increase in roof area causes smaller storage tank for attaining same reliability (Fig. 3).



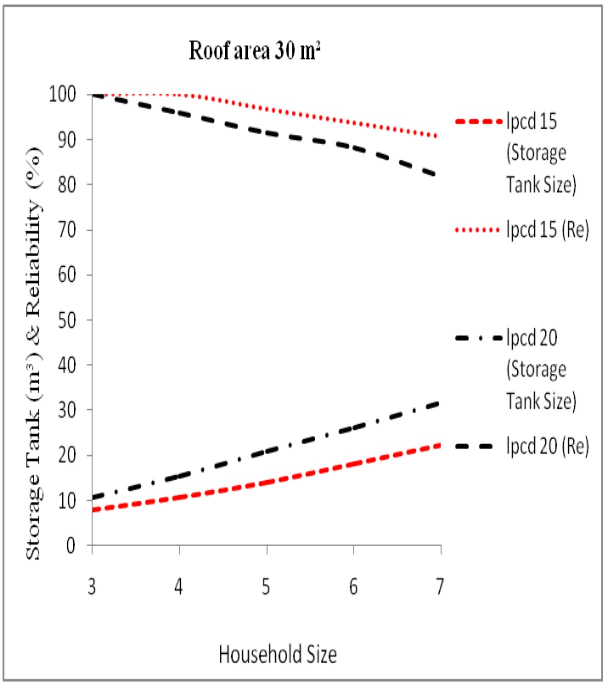


Fig. 1: Storage Tank Size, Reliability and Water Demand relationship for different roof area

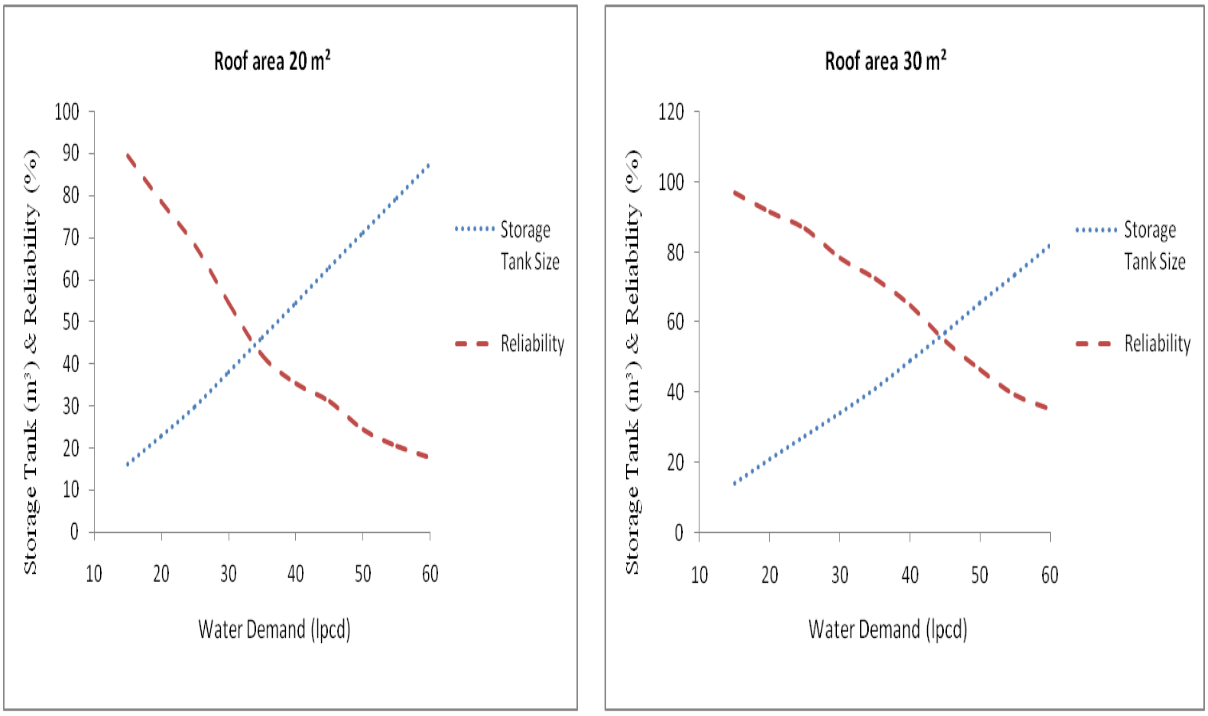


Fig. 2: Storage Tank Size, Reliability and Household size relationship for different roof area

Tank Sizing Curves

The tank sizing curves have been developed for demand of 15, 30, 45 & 60 lpcd considering household size of 5. The curves have nearly same pattern irrespective of particular demand and roof area (Fig. 3). Given data on household demand, storage tank and roof area, the reliability could be estimated from the curves. It is observed from the curves that, reliability increases with increase of storage tank size as expected except for higher household demand and lower roof area; in such situation reliability remain nearly constant with increase in storage tank. This is because lower roof catchment ($20m^2$) fails to catch water that satisfy such high demand (45, 60 lpcd).

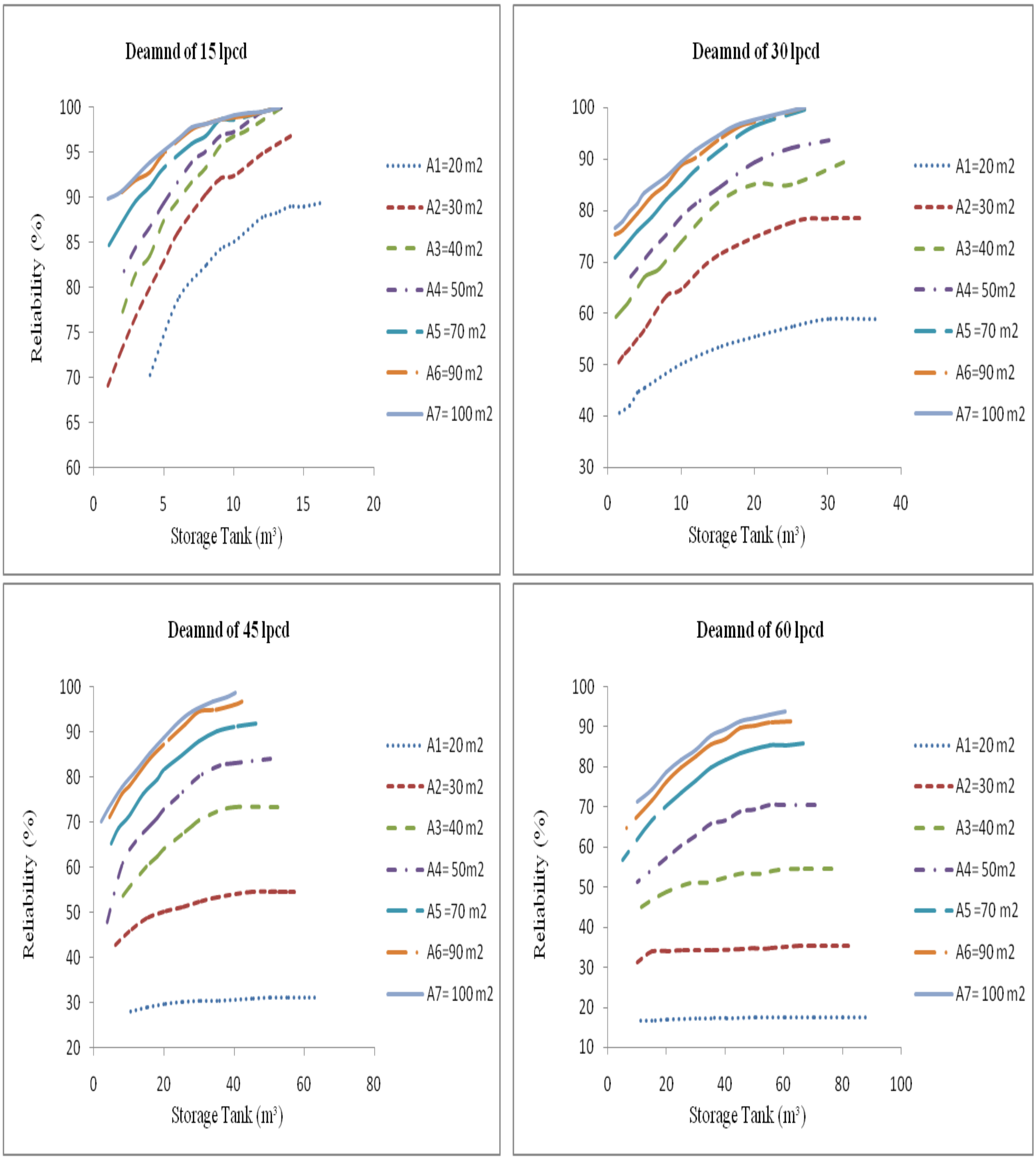


Fig. 3: Tank sizing curves for different water demand and roof area

CONCLUSION

In this study, considering future predicted monthly rainfall, a design tool for storage tank design is developed for a coastal division of Bangladesh using simulation model. The model result is represented in curves so that they can be used by people who lack knowledge of hydrology and probability. Reliability of rainwater harvesting system was also assessed using the model. Sensitivity of major parameters affecting storage tank and reliability was also analyzed. It is observed that increase in roof area causes smaller storage tank for attaining same reliability. Generally higher the storage tank, higher is the reliability, but reliability could not be increased by increasing the storage tank in case of smaller roofs and higher water demands. With the increase of household size and water demand, storage tank size increases while corresponding reliability decreases. Finally it can be said that, for a reliable and sustainable rainwater harvesting system, this type of analysis should be conducted for a particular geographical area of Bangladesh before undertaking a rainwater harvesting project for domestic water supply.

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COMMUNITY BASED URBAN DRAINAGE SYSTEM REDUCING FLOOD RISK VIA MODEL ANALYSIS

Rupayan Saha[1*], M.M. Haque [1] & Madhusudan Debnath[1]

[1]*Institute of Water Modeling, Dhaka, Bangladesh.*
**Corresponding Author, <rsa@iwmbd.org>*

ABSTRACT

A major problem in the informal settlement of urban area of Bangladesh, is flooding caused due to the low-lying topography and lack of storm water management systems. Conducting a case study and working with local residents, we analyzed the physical and social conditions underlying flooding problems and the current interventions used by residents. In addition, this paper demonstrates specific Sustainable Urban Drainage System methods that we have adapted to an informal community setting.The sustainable planning processes are conducted by eminent study on macro and regional level infrastructure, contour, land use and satellite image for a preliminary conceptual understanding of the Urban system with a review of extensive field reconnaissance and other available reports; identification of rivers/khals surrounding the study area and collection of suitable hydrological gauging with a view to considering and processing them for the understanding of hydrodynamic response; assessment of effective range of land levels; make an intensive field visit for identification of sustainable outfalls and drainage routes in authentication of the preceding planning processes and finally storm runoff assessment using empirical formula. A model has been developed using hydrological data (rainfall & evaporation) to generate catchment runoffs which are calibrated with flows for respective design year. Mitigation of flood risk and sustainable storm water management has been done by model result analysis.

The study area is located in south west region of Bangladesh at MathbariaPourashava. To minimize flood risk via sustainable drainage improvement, software like Arc-GIS for Map generation, Mike 11 and Mike VIEW for model development & result analysis respectively has been utilized by us.

Keywords: Urban Drainage, Flood Risk Model, Mike 11, Arc-GIS, Sustainable Drainage System.

INTRODUCTION

It has been identified that improvement of the drainage system is one of the highest priority needs of the Pourashava 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 suitable outfall. Moreover, those drains are insufficient to deal with the full drainage resulting from rainfall runoff. This Drainage Improvement Study for Mathbaria Pourashava has been carried out as part of the integrated development planning for Water supply, Drainage and Sanitation.

METHODOLOGY

The drainage system of Pourashava and its response to hydrology govern the planning for its storm drainage system. 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 rational method.

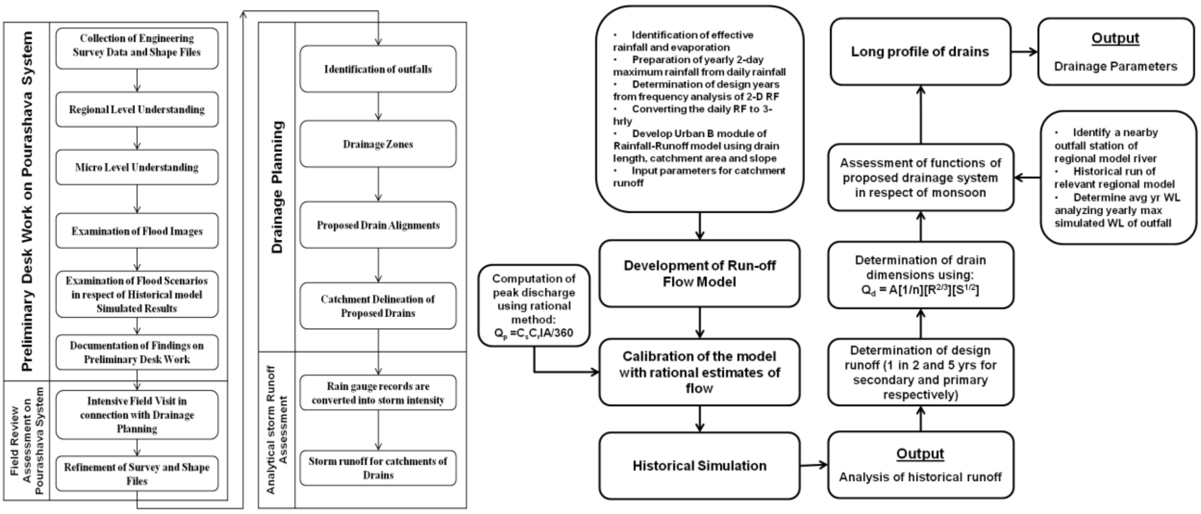


Figure 1: Flow Chart for Drainage Study (Analytical & Modelling)

The approach comprises development of Rainfall-runoff model for hydrological analysis and correlating the proposed drainage system with the existing river model to assess hydraulic performances. Modelling approach is used to generate catchment runoffs. The runoffs are calibrated against flows using rational formula known as modified rational formula. The study includes data collection from primary and secondary sources, analyzing and checking of data, development of hydrological model, reviewing and correlating Pourashava drainage system with existing regional models, identification of design year and simulation of the model for the design year, determination of design flows from model simulation, calculation of design parameters from design flows etc.

Topography

Mathbaria Pourashava is located at Mathbaria Upazila, Pirojpur District under Barisal Division. The land elevation of the Pourashava effectively ranges between 0.24 mPWD and 3.50 mPWD.

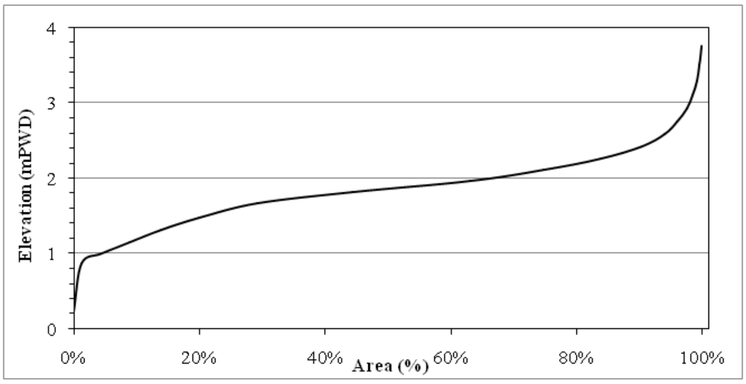


Figure 2: Area-Elevation curve of Mathbaria Pourashava

River and Khal System

Pourashava lies in the South West region of Bangladesh. The existing polders 39/1 & 39/2 protect the whole Pourashava area from flood tides. The Baleswar River flows on the west of polders. The Bishkhali River flows on the east of polders. Both the rivers flow down to the Bay of Bengal on the south direction.

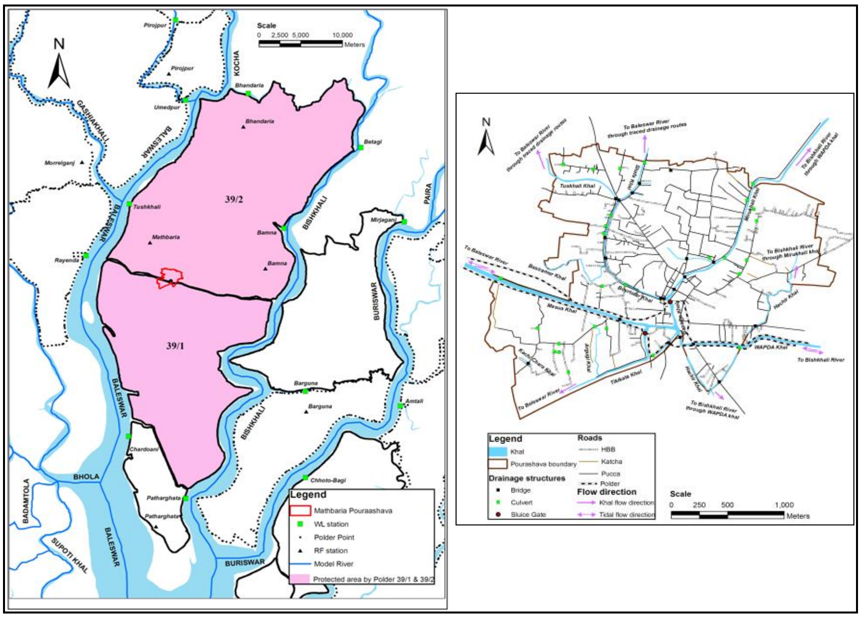


Figure 3: Regional river system of Mathbaria Pourashava

Masua/WAPDA khal is one of the major khals of Pourashava which flows east-west direction through Pourashava. This kahl is confined by the embankments of polders and is a thorough tidal khal which connects the Baleswar and Bishkhali Rivers. There are numbers of distributory khals of the Masua khal and they have regulating structures on the embankments.Tikikata khal flows on the south west part of Pourashava and finally routes to Baleswar River in west direction. Most of the khals of northern part of Pourashava flow towards north direction.Tuskhali & Shafa khals are two distributory branch of Boyrtolar khal. These khals flow north-west direction and eventually fall to Baleswar River through traced drainage routes.Mirukhali khal courses to north east direction in between pourashava boundary. After crossing the boundary it turns toward south-east way and drains to WAPDA khal in assistance with regulating structure. Hachir khal is the discrete branch of WAPDA khal. One part of Hachir khal collects local region water and flow in north direction to meet with Mirukhali khal. Other part runs towards south east direction and finally falls into Bishkhali River.

Rainfall

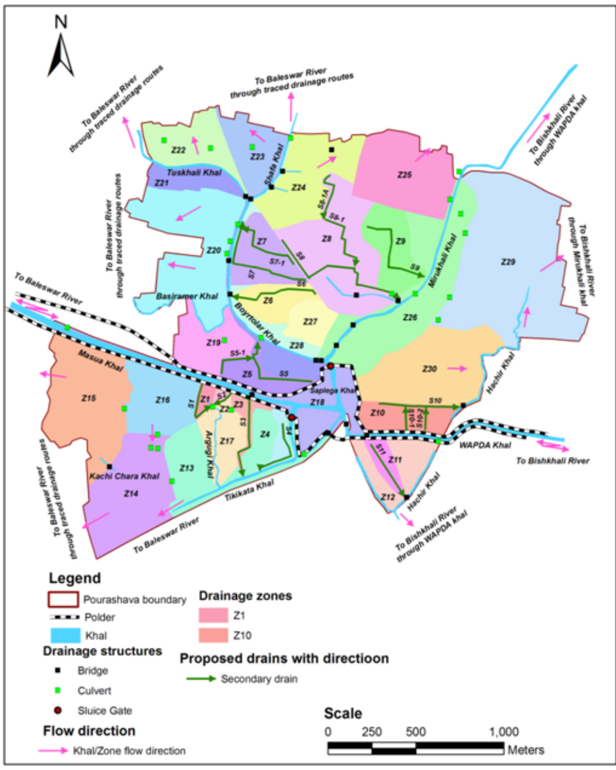
Mathbaria is a rainfall gauging station (R265) with reasonable length of records and is located nearest to the Mathbaria Pourashava. It is found that the average 1-day maximum rainfall event at Mathbaria is 1.11 times that of Dhaka and conversion factor for Mathbaria is estimated to 1.22. Consequently rainfall intensity at Mathbaria is assessed higher than Dhaka for same storm duration and frequency.

Flood

The nearest water level gauging station is at Tushkhali (107.1) on the Gorai-Madhumati River which is located 10 km off the north-west of Mathbaria Pourashava. The water level record at Tushkhali (107.1) is not representative for the Pourashava since the pourashava is located inside the polder system. The average year flood level for the Pourashava is estimated to 1.10 mPWD from field visit local people’s opinion and satellite based flood map analysis. The area of the Pourashava ranges from shallow to high lands.

Drainage Improvement Plan

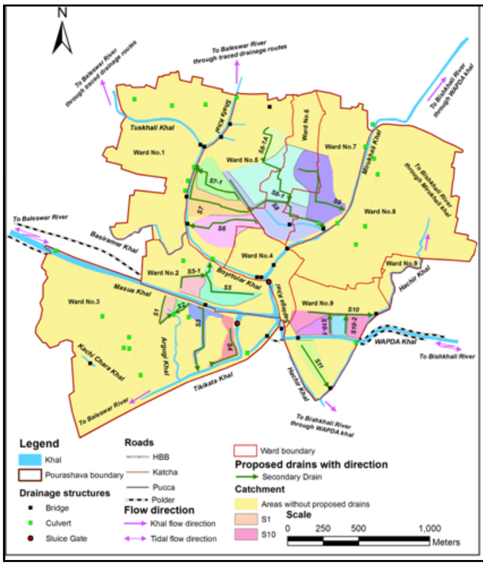
The whole Pourashava has been divided into 30 zones for drainage improvement plan. Zones 1 through 11 are planned with proposed storm drains as they are in the core area of Pourashava or will be characterized as core area in near future. Rest of the zones are not planned with proposed storm drains while they are planned with their outfalls for future drainage details.



Drainage Zone	Drainage Area (ha)	Discharge (m^3/sec)
Z1	2	0.18
Z2	1	0.09
Z3	3	0.24
Z4	3	0.26
Z5	7	0.55
Z6	7	0.51
Z7	11	0.78
Z8	29	2.03
Z9	11	0.82
Z10	9	0.69
Z11	3	0.23
Z12	10	0.81
Z13	20	1.46
Z14	24	1.84
Z15	27	2.06
Z16	14	1.10
Z17	9	0.78
Z18	21	1.48
Z19	11	0.98
Z20	30	2.24
Z21	14	0.92
Z22	12	0.99
Z23	11	0.95
Z24	19	1.46
Z25	21	1.64
Z26	33	2.42
Z27	6	0.54
Z28	6	0.54
Z29	54	3.75
Z30	26	1.97

Figure 4: Drainage Zones & its attributes of Pourashava

The delineated catchments and drainage routes are shown in **Figure 5**. Topographic data such as land and road crest level and drain cross sections, hydrometric data such as water level and discharge, meteorological data such as rainfall and evaporation were collected to develop Mathbaria Drainage Model.



Sl No	ID of Drain and Catchment	Length (m)	Area (ha)	Drain Type	Outfall
1	S1	313	2	Sec, Rec, RCC	Argugi khal
2	S2	130	1	Sec, Rec, RCC	Argugi khal
3	S3	529	3	Sec, Rec, RCC	Tikikata khal
4	S4	404	3	Sec, Rec, RCC	Tikikata khal
5	S5-1	248	2	Sec, Rec, RCC	S5
6	S5	535	5	Sec, Rec, RCC	Boyrtolar khal
7	S6	516	7	Sec, Rec, RCC	Boyrtolar khal
8	S7-1	429	5	Sec, Rec, RCC	S7
9	S7	678	6	Sec, Rec, RCC	Boyrtolar khal
10	S8-1A	324	3	Sec, Rec, RCC	S8-1
11	S8-1	414	14	Sec, Rec, RCC	S8
12	S8	918	12	Sec, Rec, RCC	Mirukhali khal
13	S9	571	11	Sec, Rec, RCC	Mirukhali khal
14	S10-1	135	2	Sec, Rec, RCC	S10
15	S10-2	129	2	Sec, Rec, RCC	S10
16	S10	460	5	Sec, Rec, RCC	Hachir khal
17	S11	355	3	Sec, Rec, RCC	Hachir khal

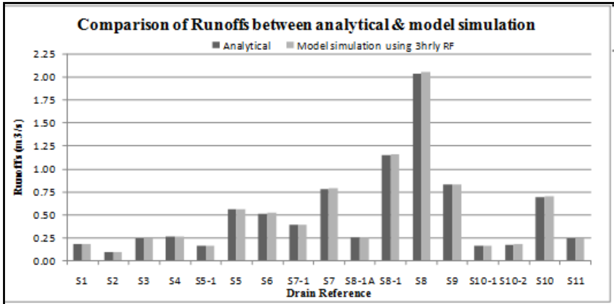
Figure 5: Drainage Routes and Catchments for Pourashava

Model Development

An independent drainage model for Mathbaria Pourashava has been developed in consideration of existing and proposed hydraulic and hydrologic features in the area. The model acts as Urban Drainage Model for Mathbaria Pourashava which comprises only rainfall-runoff model for hydrological analysis. Historical simulation of the model has been carried out. Runoffs generated from the model have been analyzed to determine design flow of drains.

MODEL CALIBRATION & SIMULATION

All catchments within the Mathbaria Pourashava area were modelled with the MIKE Urban model B concept. Hydrodynamic parameters were estimated for the drainage channels and topographic storages at each cross section were also included in the model. The models have been calibrated for the secondary and primary drains against flows estimated using rational method. The main parameter to carry out calibration procedure is Manning's Roughness, M (inverse of roughness coefficients n). The model simulations have generated runoff in the drains draining towards the outfall channels proposed for the Mathbaria Pourashava area. An analysis of 20 (1988-08) year runoffs from historical simulation of model is given in figure 6 where design runoffs for 2 and 5 year return period for secondary and primary drains respectively is shown also.



ID of Drain and Catchment	Average Flow (m^3/s)	Minimum Flow (m^3/s)	Maximum Flow (m^3/s)
S1	0.21	0.04	0.32
S2	0.10	0.01	0.16
S3	0.27	0.07	0.42
S4	0.29	0.08	0.44
S5-1	0.18	0.06	0.27
S5	0.63	0.21	0.92
S6	0.64	0.10	1.01
S7-1	0.47	0.10	0.73
S7	0.88	0.50	1.17
S8-1A	0.29	0.06	0.45
S8-1	1.31	0.50	1.95
S8	2.23	1.25	2.89
S9	0.98	0.20	1.55
S10-1	0.20	0.01	0.32
S10-2	0.20	0.01	0.32
S10	0.80	0.50	1.03
S11	0.27	0.02	0.45

FIGURE 6: COMPARISON OF RUNOFFS BETWEEN RATIONAL METHOD AND MODEL SIMULATIONS

RESULTS & DISCUSSIONS

For a given discharge the geometric section of a drain depends mainly on bed slope and the frictional resistance of the contact surface to flowing water. Manning's Equation is used for the calculation of flow velocity and determining drain section. These cross sections have been finalized after several trials in consideration outfall conditions. Geometric shapes of drains have been determined and given in **Table 1**. A sample longitudinal profile of drain S1 is shown in **Figure 7**.

Table 1: Geometric section &Drainage parameters

Drain ID	Chainage (m)	Design Flows Q_d (m3/s)	Bottom Width (m)	Actual drain depth* (m)	Design Capacity, Q_c (m^3/s)	Ground Level		Drain Bottom level		Remarks
						U/S	D/S	U/S	D/S	
S1	0-313	0.18	0.45	0.60	0.19	3.06	2.00	2.46	1.40	RCC Drain
S2	0-130	0.09	0.35	0.50	0.10	3.80	2.00	3.3	1.50	RCC Drain

S3	0-529	0.24	0.65	0.65	0.25	3.70	2.20	3.05	1.55	RCC Drain
S4	0-404	0.26	0.65	0.70	0.28	2.90	2.10	2.2	1.40	RCC Drain
S5-1	0-248	0.16	0.55	0.60	0.17	2.47	2.04	1.87	1.44	RCC Drain
S5	0-535	0.56	0.75	1.00	0.58	2.30	2.20	1.74	1.20	RCC Drain
S6	0-516	0.52	0.75	0.80	0.54	2.92	2.02	2.12	1.22	RCC Drain
S7-1	0-429	0.39	0.70	0.85	0.42	2.75	2.24	1.9	1.39	RCC Drain
S7	0-678	0.79	1.00	0.85	0.83	2.80	1.95	2.07	1.10	Low Cost Drain
S8-1A	0-324	0.25	0.65	0.70	0.28	2.14	1.92	1.55	1.22	RCC Drain
S8-1	0-414	1.16	1.20	1.05	1.17	1.92	1.80	1.13	0.75	Low Cost Drain
S8	0-918	2.05	1.40	1.40	2.11	2.72	2.54	1.91	1.14	RCC Drain
S9	0-571	0.83	1.05	1.00	0.87	2.30	2.09	1.57	1.09	Low Cost Drain
S10-1	0-135	0.16	0.50	0.65	0.17	3.18	2.53	2.53	1.88	RCC Drain
S10-2	0-129	0.17	0.55	0.65	0.20	3.10	2.44	2.45	1.79	RCC Drain
S10	0-460	0.70	0.85	1.00	0.70	2.75	2.39	1.85	1.39	RCC Drain
S11	0-355	0.24	0.60	0.70	0.25	2.83	2.42	2.13	1.72	RCC Drain

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 1.1 mPWD

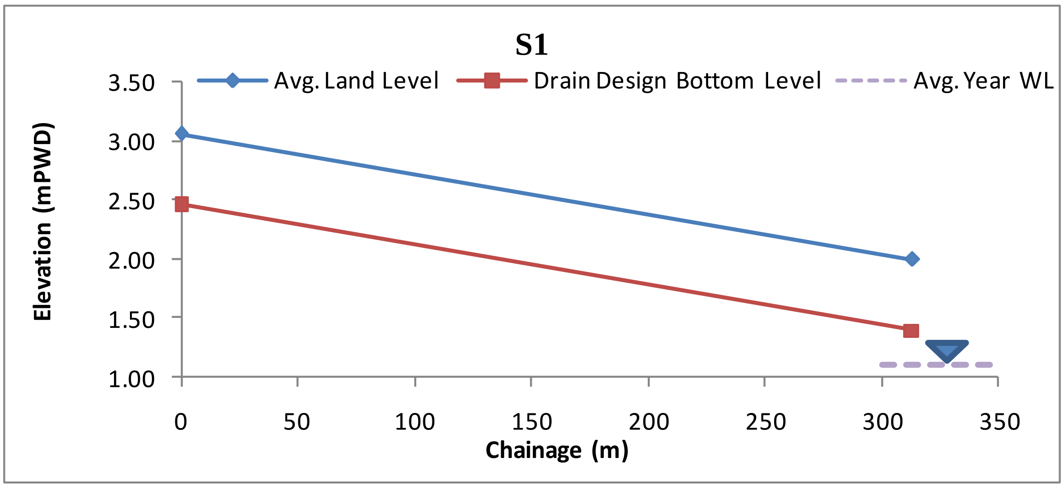


Figure 7: Longitudinal profile of S1 drain

A flood map is also developed for pourashava in respect of which functions of the proposed drainage system can be assessed is shown in **Figure 8**. 93% land of Mathbaria Pourashava is above the average year flood level. As the drains are above average year flood level, the drains will function properly.

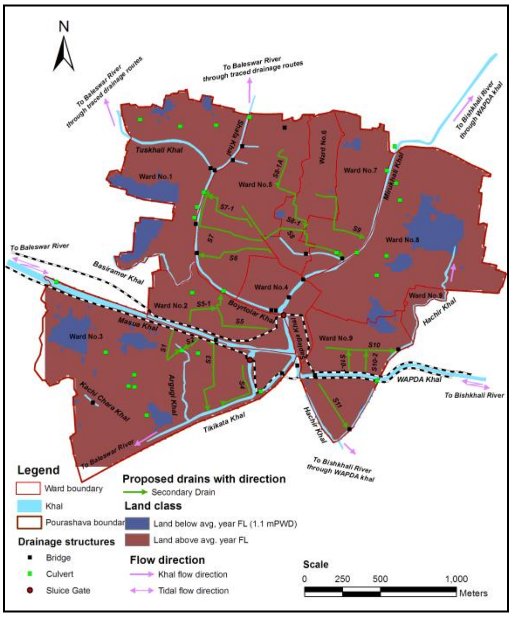


Figure 8: Lands above and below average flood level

CONCLUSIONS & RECOMMENDATIONS

Systems analytical study followed by the assessment of its performance in aid of model application is found to be a suitable and convenient method adopted for drainage study of the Pourashava. S1, S2, S3, S4 , S5, S6, S7, S8, S9 S10 & S11 drains have priority needs while S5-1, S7-1, S8-1A, S8-1, S10-1 & S10-2 drains are proposed in view of near future needs for the Pourashava. Detailed engineering survey is a pre-requisite for planning of tertiary drains, and is recommended for planning and design at the time of detailed engineering. About 32 nos. of cross drainage works will be required for the priority drainage systems and the rest will be required for the future drainage systems. It is recommended that such land is raised to the similar level of high land (preferably not less than 2.5 mPWD) of the Pourashava. Part of Zone 14, 15, 20, 22, 23, 25 & 29 of the Pourashava will drain overland across the Pourashava boundary to the finally route to Baleswar & Bishkhali River. The Pourashava authority will have institutional linkages with all relevant line agencies for the continuation of drainage provision of these Zones in view of long term consideration. The proposed drainage system is found adequate for storm drainage of the Pourashava, and is recommended for detailed study and necessary modification in consideration of social, environmental, technical, economical and institutional constraints.

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STUDY ON SEVERITY OF GROUND WATER QUALITY AS WELL AS ALTERNATIVE SOURCES OF WATER AT SOME SELECTED AREAS OF COMILLA DISTRICT

M. N. A. Siddique[1], B. M. G. Kibria[2] & M. R. Amin[2*]

[1] *BETS Consulting Services Limited, Gulshan-1, Dhaka-1212, Bangladesh.*
[2] *Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
**Corresponding Author, <ruhulbuet@gmail.com >*

ABSTRACT

There is an urgent need for Bangladesh to identify the arsenic (As) contaminated tube-wells (TWs) in order to assess the health risks and initiate appropriate mitigation measures. This involves testing water in millions of TWs, raising community awareness about the health problems related to chronic as exposure from drinking water and providing alternative safe water option for the exposed population of the country. The use of spatial maps in a participatory context emerged as an important tool for an effective and rational distribution of alternative safe water options for the exposed population of the country. Field test kit offers the only practical tool available to screen all the TW water considering the time frame and financial resources of the country. Lack of financial resources and identification of an appropriate distribution tools are some of the major obstacles to provide sustainable solution to the exposed population of the country. Spatial mapping exercise along with community participation can help in maximizing the safe water coverage of different alternative safe water options by reducing financial involvement. Combining peoples' voice with that of spatial information gave better results and the method has already been proved to be useful in targeting non-served areas. Participatory Rural Appraisal (PRA) methods along with geographical information system (GIS) used in the study to obtain relevant information. Participants from different focus groups were asked to determine their 'own priorities' for spatial planning of alternative arsenic-safe water options. This study discusses community perspectives on demand-based safe water options and reveals the suitability of using participatory geographic information system (PGIS) technique to target non-served areas for rational distribution of safe water options.

Keywords: Arsenic, Iron, Phosphate, Tube-Well, GIS

INTRODUCTION

Access to safe drinking water is essential to health, a basic human right and a component of effective policy for health protection. Chronic arsenic (As) poisoning results from drinking water with high levels of As over a long period of time and the consequences are dependent on the susceptibility, the dose and the time course of exposure. Today, tens of millions of people mainly in developing countries are affected by levels of As in drinking water exceeding the World Health Organization (WHO) drinking water guideline value of 10 μg/L and the global impact now makes it a top priority water quality issue. Proper identification of the As contaminated tube-wells (TWs) is thus needed to assess the health risks and initiate appropriate mitigation measures. This involves testing water in millions of TWs and raising community awareness about the health problems related to chronic As exposure from drinking water, and provision of alternative safe water option for the exposed population of the country. Further, it is also important to optimize the various approaches of As mitigation and the need to integrate a process of community participation together with the spatial information for developing an effective and rational distribution of alternative safe water options to be accepted by community.

Our study area is highly Arsenic contaminated according to NAMIC report. In the BAMWSP (Bangladesh Arsenic Mitigation Water Supply Project) study, it was observed that more than 90% Tube-Wells are contaminated by Arsenic in Chandina Upazila, Comilla. On the other hand, in this study area good numbers of surface water pockets are unearthed during satellite image study and field observation. During our limited field study it was also revealed that most of the TWs are arsenic contaminated.

The overall objectives of the study are to:

■ Assess the extent of As contamination in the study area by testing TW water using digital Arsenator and field test kit.

■ Identify the best suitable alternative safe water option for the As exposed population in the study area.

■ Scientific investigation of a people's driven initiative to mitigate the As problem in a sustainable way.

■ Use of spatial maps in a participatory context for an effective and rational distribution of As mitigation options of the project area.

METHODOLOGY

Identification of the Tube-wells (TWs) positions in the project area is the first and foremost important work and to do so, a good precision GPS is required. A well defined checklist is produced to collect the physical and chemical characteristics of the TW. Digital Arsenator (Wagtech) is used to identify the arsenic content of the tube-wells. Arsenator is calibrated daily by Blank & 50 μgm Standard and monthly by five point calibration. Iron and Phosphate are tested by Hach Kit. After getting the physical characteristics of the well data, the tests are to be done on spot. It is noteworthy that purge the stagnant water from the well prior to collecting water samples to ensure that the samples are representative of the fresh groundwater. The volume of water that must be purged from a well is dependent on the depth of the well. The volume (V) of water (in litres) that must be purged from a well is defined as follows:

$$V = \pi \times r^2 \times L \times 1000 \times 3$$

Where, $\pi = 3.14159$, r = radius of the tube-well pipe (measured in meters i.e. 1 inch = 2.54×10^{-2} m), L = length of the tube-well pipe measured in meters i.e. 1 m = 3.28 feet), 3 is the safety factor that accounts for the static water in the well. Finally, by using ArcMap10 the collected data are interpreted in a way that the well position, characteristics (physical and chemical) etc. are to be well defined. In line with the well characterization identification of the suitable alternative potable water options i.e. Surface Water pockets (Ponds) and its water content over the year is to be collected.

Analysis through GIS

Geographic information system (GIS) technology can be used for scientific investigations, resource management, and development planning. For example, a GIS might allow emergency planners to easily calculate emergency response times in the event of a natural disaster, or a GIS might be used to find wetlands that need protection from pollution. The power of a GIS comes from the ability to relate different information in a spatial context and to reach a conclusion about this relationship. Most of the information we have about our world contains a location reference, placing that information at some point on the globe.

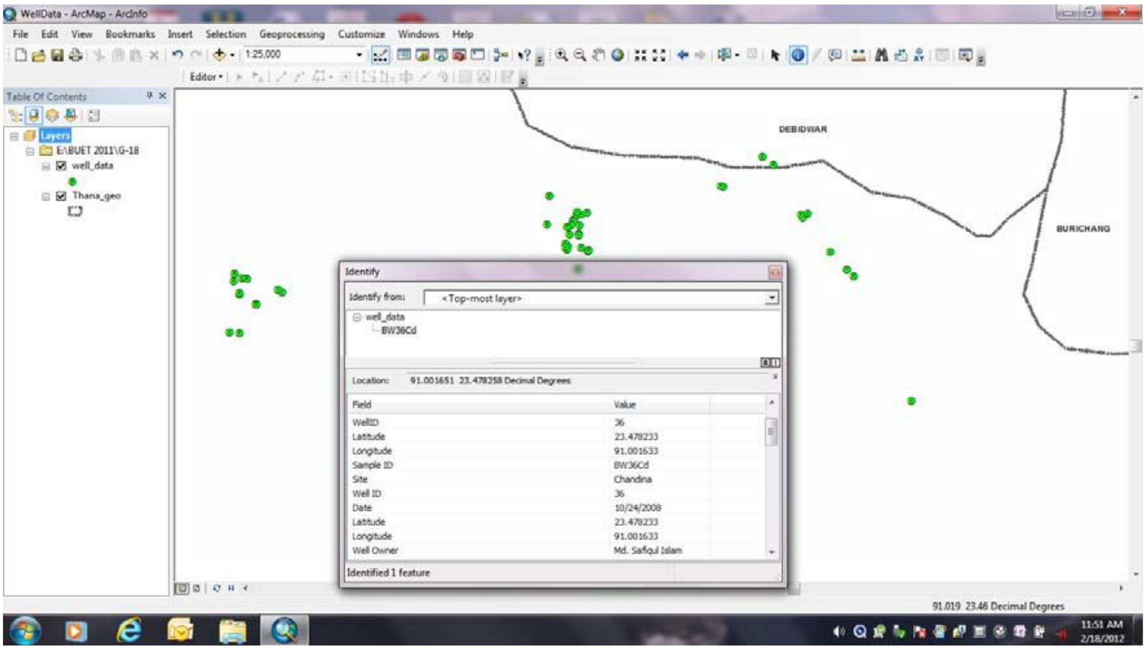

Fig. 1: GIS application example tool

Data capture putting the information into the system involves identifying the objects on the map, their absolute location on the earth's surface, and their spatial relationships. Software tools that automatically extract features from satellite images or aerial photographs are gradually replacing what has traditionally been a time consuming capture process. Objects are identified in a series of attribute tables the "information" part of a GIS. Spatial relationships, such as whether features intersect or whether they are adjacent, are the key to all GIS-based analysis. Figure 1 given above shows the GIS application example tools.

Data Collection

Basically, the data are collected from the field in a program of Bangladesh Environmental Technology Verification-Support to Arsenic Mitigation (BETV-SAM). It was the Second Verification Field Testing Program of OCETA (Ontario Centre for Environmental Technology Advancement), certified by BCSIR and directed by DPHE in the year 2008-09. More on, secondary data from intensive internet browsing and from NAMIC, BAMWSP, DPHE etc. are collected according to the requirements of this study.

Table 1: Study area

<table>
<tr><th>Serial No.</th><th>Village</th><th>Union</th><th>Thana</th><th>District</th></tr>
<tr><td>1</td><td>Dumuria</td><td>Kiron Khal</td><td rowspan="6">Chandina</td><td rowspan="6">Comilla</td></tr>
<tr><td>2</td><td>Biswas</td><td rowspan="3">Chandina Pouroshova</td></tr>
<tr><td>3</td><td>Rarir Char</td></tr>
<tr><td>4</td><td>Hanong</td></tr>
<tr><td>5</td><td>Chandiara</td><td rowspan="2">Borkuit</td></tr>
<tr><td>6</td><td>Borkut</td></tr>
</table>

Table 2: Statistical data of the study area

Union/ Village	Tube Well Survey				Household Survey				Patients		
	Total TW	No. of Safe TW	No. Contaminated TW	% of Contamination	No. of Household	Population (Male)	Population (Female)	Total	Male	Female	Total
12/209	17585	1696	15889	90.36	62610	211269	180260	391529	63	75	138

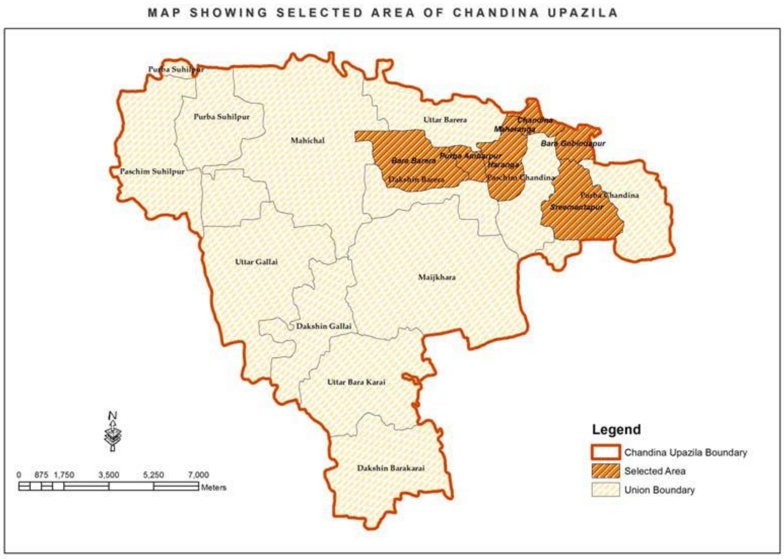

Fig. 2: Study area of Chandina upazilla, Comilla district, Bangladesh

Data Analysis

Based on the checklist, the following data are collected from different tube-wells. From more than fifty tube-wells data are collected and analyzed in the GIS software considering the As, Fe and PO_4 intensity of six villages in the Chandina Upazilla. The data are listed below in the tables considering different fields.

Table 3: Location details of the tube-wells

Sample ID	Site	Well ID	Latitude	Longitude	Well Owner	Father's name	Specific location	Village	Union	Thana (Upazila)
BW1Cd	Chandina	1	23.473	90.980	Koruna Mohon Das	Late Norendo Chandra Das	Dumuria Primary School	Dumuria	Keron Khal	Chandina
BW2Cd	Chandina	2	23.474	90.980	Nitai Kanti Das	Late Norendro Chandro Das	Dumuria Primary School	Dumuria	Keron Khal	Chandina
BW3Cd	Chandina	3	23.474	90.980	Sento Sarkar	Late Bosonto Sarkar	Dumuria primary School	Dumuria	Keron Khal	Chandina

Sample ID	Site	Well ID	Latitude	Longitude	Well Owner	Father's name	Specific location	Village	Union	Thana (Upazila)
BW4Cd	Chandina	4	23.473	90.979	Sukumar Chandra Sarkar	Late Biddyut Chandra Sardar	Dumuria Primary School	Dumuria	Keron Khal	Chandina
BW5Cd	Chandina	5	23.974	90.980	Dulal Chandra Sarkar	Late Sachindra Chandra Sarkar	Dumuria	Dumuria	Keron Khal	Chandina

Table 4: Sample data tube-wells physical characteristics

Well ID	Well Type	Well Depth (Feet)	Well Depth (Meter)	Static Water Table (Feet)	Platform	Availability of Electricity	Space for Technology Installation	Drainage	Access
1	Shallow	115	35.06097	20	No	No	Yes	Bad	Good
2	Shallow	115	35.06097	20	Yes	No	Yes	Good	Good
3	Shallow	100	30.4878	20	Yes	No	Yes	Good	Good
4	Shallow	85	25.91463	20	Yes	No	No	Good	Good
5	Shallow	90	27.43902	20	Yes	No	Yes	Good	Good
6	Shallow	105	32.0122	18	Yes	No	Yes	Bad	Good
7	Shallow	100	30.4878	20	Yes	No	Yes	Good	Good
8	Shallow	110	33.53659	20	No	No	Yes	Bad	Bad
9	Shallow	100	30.4878	18	Yes	No	Yes	Good	Good
10	Shallow	80	24.39024	18	Yes	No	Yes	Good	Good
11	Shallow	100	30.4878	20	No	No	Yes	Bad	Good
12	Shallow	115	35.06097	20	No	No	Yes	Bad	Good
13	Shallow	110	33.53659	18	No	No	Yes	Bad	Good
14	Shallow	110	33.53659	20	Yes	Yes	Yes	Good	Good
15	Shallow	110	33.53659	20	Yes	Yes	Yes	Good	Good
16	Shallow	110	33.53659	25	Yes	Yes	Yes	Good	Good
17	Shallow	120	36.58537	25	Yes	Yes	Yes	Good	Good
18	Shallow	115	35.06097	15	Yes	No	Yes	Good	Good

Table 5: Chemical characteristics of tube-wells

Well ID	As Sample from Calibration Curve Equation (µg/L)	Fe Sample (mg/L)	PO_4 Sample (mg/L)
1	310	2.4	7
2	231	2.6	6
3	310	2.6	6
4	318	2.4	5
5	222	3.2	6
6	392	2.8	6
7	301	2.6	7
8	336	3	8
9	336	4	8
10	205	5.6	7
11	222	11	6
12	248	8.4	8
13	397	3	7
14	226	1.9	7
15	234	3.4	5
16	252	2.6	7

It is clear from the Table 5 data above that all the tube-wells (TWs) As and Fe content are above the DOE standards (As = 0.05 mg/l, Fe = 0.3-1.0 mg/l). Whereas about 48% of the TWs PO_4 content exceeds the DOE standard (PO_4 = 6.0 mg/l) and it found as high as 15 mg/l. Figures below shows the As, Fe and PO_4 content of the wells in the study area.

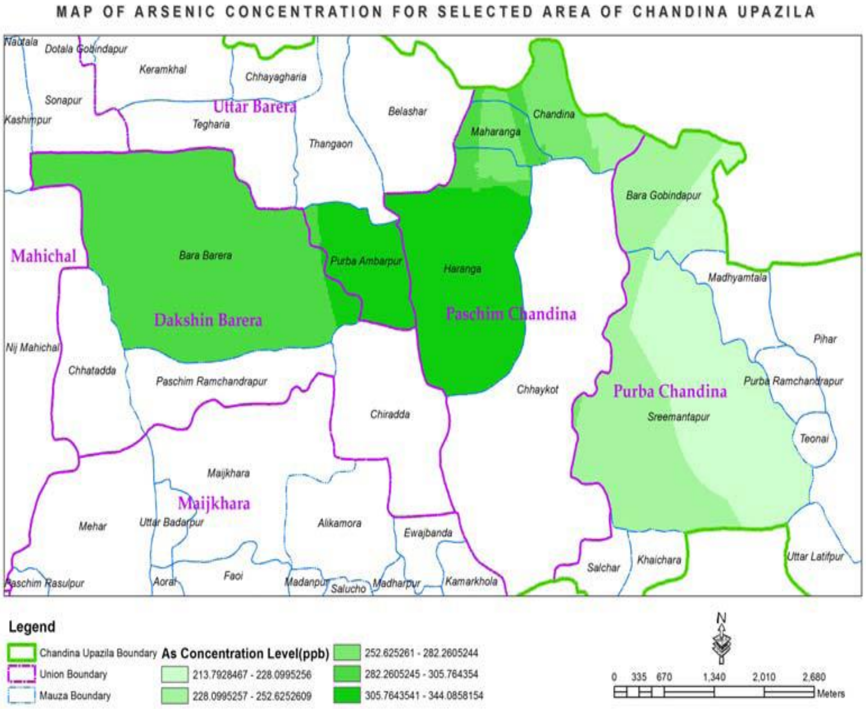

Fig. 3: Map of Arsenic (As) concentration in the study area

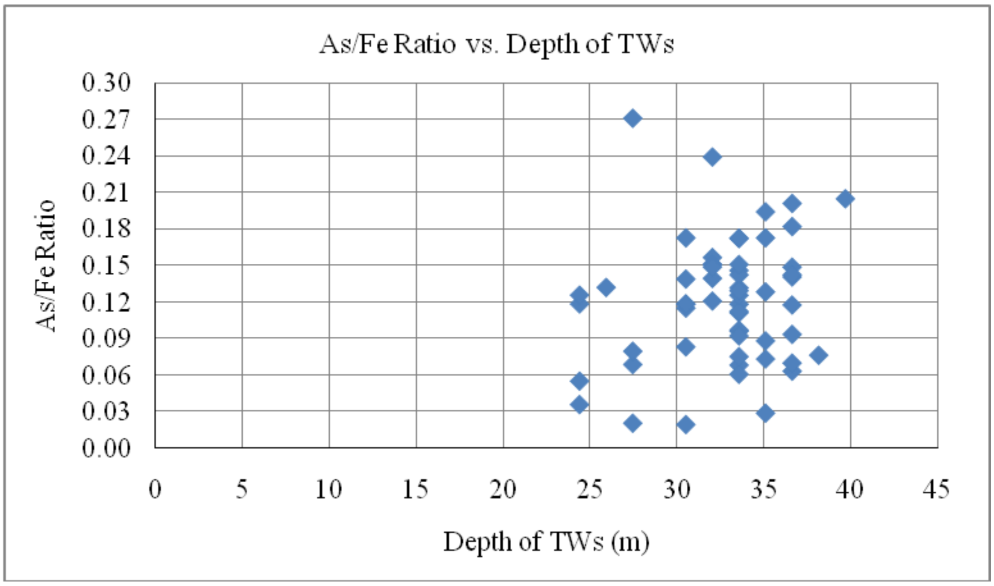

Fig. 4: As/Fe Ratio vs. Depth of Tube-wells

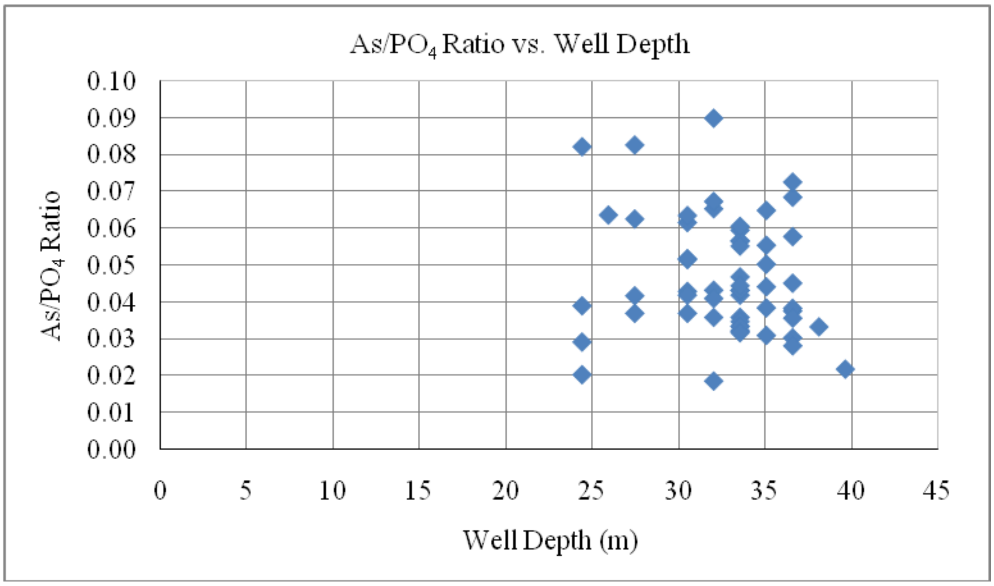

Fig. 5: As/PO_4 Ratio vs. Depth of Tube-wells

RESULTS AND DISCUSSIONS

Arsenic Removal Technology in the Study Area

As/PO_4: In our study area, Arsenic Removal Technology will not be effective as the PO_4 is very high. We know, PO_4 has strong affinity for solid surface and presence of high concentration of PO_4 can reduce removal efficiency of Arsenic in adsorption-based treatment systems. It is evident that PO_4 has the higher affinity for metal oxides and its presence reduces the Arsenic removal significantly. The theoretical affinities at neutral pH for anions sorption on metal oxides are as follows:

$$PO_4 > SeO_3 > AsO_4 > AsO_3 > SiO_4 > SO_4$$

As/Fe: On the other hand, effect of Iron may significantly affect the performance of an adsorption media. The reason is the possible fouling of the porous adsorption media by precipitated iron particles on the surface. It will clog the filter medium thereby reducing the filtration rate. Accumulation of Iron particles on the adsorption surface will decrease the readily available sorption sites for arsenic. Presence of Iron in water leads to scaling and membrane fouling. The membrane once fouled by impurities cannot be backwashed. Pretreatment of ground water is must before membrane filtration in our country.

Alternate Option (Surface Water)

There are enough water pockets in the study area. More than twenty water pockets are observed in and around the village areas. These water pockets are locally used mostly for fish culture, cattle bathing, irrigation of the paddy lands etc. Location of the water pockets (ponds) are shown in the figure 6 below. These ponds would be the best alternative sources of the potable water with proper pre-treatment by slow sand filtration.

Table 6: Location and characteristics of ponds in the study area

Pond ID	Longitude	Latitude	Area (m^2)	Depth in Dry Season (m)	Amount of Water in Dry Season (m^3)	Depth in Wet Monsoon (m)	Amount of Water in Wet Season (m^3)	Fish Cultivation	Water Quality	Possibility of Surface Water Treatment
1	23.48922	91.00417	1775	3	5325	2	3550	yes	Moderate	yes
2	23.48903	91.00316	770	2.5	1925	1.75	1347.5	no	Bad	yes
3	23.48936	91.00481	2132	2.5	5330	2	4264	yes	Moderate	yes
4	23.48921	91.00726	5292	3	15876	2	10584	yes	Good	yes
5	23.48978	91.00862	4290	3	12870	2	8580	yes	Good	yes
6	23.48792	91.00935	7980	3	23940	2	15960	yes	Good	yes
7	23.48579	91.01013	14065	3.5	49227.5	2.25	31646.25	yes	Good	yes
8	23.48592	91.00784	4590	3	13770	2.25	10327.5	yes	Moderate	yes
9	23.48567	91.01514	4050	3	12150	2	8100	yes	Moderate	yes
10	23.48666	91.00017	1500	3	4500	2	3000	yes	Moderate	yes
11	23.48632	91.00497	1350	2.5	3375	1.75	2362.5	no	Moderate	yes
12	23.48065	91.00354	2790	2.5	6975	1.74	4854.6	yes	Moderate	yes
13	23.48023	91.00003	2016	2.75	5544	2	4032	yes	Moderate	yes
14	23.47835	91.00205	1540	2.75	4235	2	3080	yes	Moderate	yes
15	23.47737	91.00233	1482	2.75	4075.5	2	2964	yes	Moderate	yes
16	23.47674	91.00423	1628	2.25	3663	1.25	2035	yes	Bad	yes
17	23.47556	91.01029	1540	2.25	3465	1.5	2310	yes	Moderate	yes
18	23.47459	91.01098	1680	2.25	3780	1.5	2520	no	Bad	yes
19	23.47604	91.00777	2080	2.25	4680	1.25	2600	yes	Moderate	yes
20	23.48462	90.99161	3584	2	7168	1.5	5376	yes	Moderate	yes
21	23.48692	90.98978	2088	1.75	3654	1.25	2610	no	Moderate	yes
22	23.48204	90.99177	2880	2	5760	1.5	4320	no	Moderate	yes

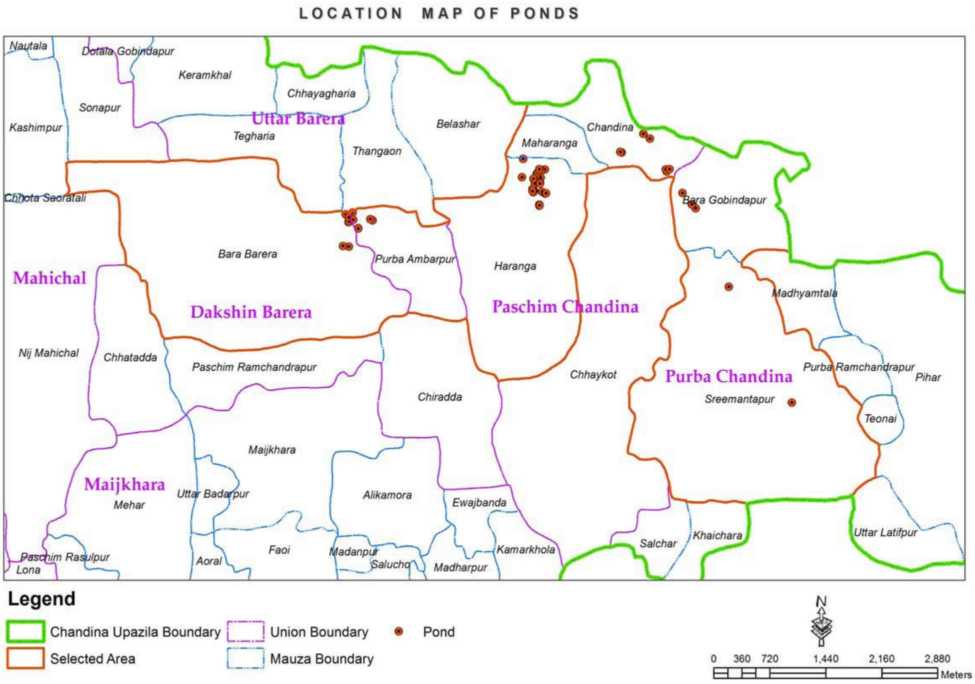

Fig. 6: Location map of ponds in the study area

CONCLUSION

In this study, the major findings are as follows:

■ Study areas ground water is highly As contaminated (Minimum 200 μgm/l and Maximum 431 μgm/l).

■ Our study echo with the BAMWSP result.

■ As/Fe and As/PO_4 ratio is comparatively very high i.e. on an average 0.12 and 0.047 respectively conceding the tube-well water depth around 32m (105 ft).

■ Iron content is comparatively high enough i.e. minimum 1mg/l and Maximum 11mg/l.

■ PO_4 content is as high as 15 mg/l and as minimum as 3mg/l.

■ Enough surface water sources are available in the study area.

Arsenic situation still remains as one of the major development threat to Bangladesh in the absence of proper implementation of the As mitigation guidelines to ensure safe drinking water for the exposed population. Testing of TW water is considered to be the first step to mitigate the As problem. In the absence of any clear evidence that As concentration changes over time, it is also necessary to monitor the TWs on a regular basis to ensure safe supply of drinking water. By virtue of GIS application, we can quicken our decision and share the information for future planning. Supply of safe drinking water remains the most crucial issue in the whole As mitigation programme in Bangladesh. Provision of safe drinking water reduced the risk of health hazards and disease caused by drinking As contaminated water. Surface water would be the best alternative safe water options in the study area. In the present study, we could not find the chances to observe the sustainability of alternative options considering the technical viability, financial viability, and community acceptability.

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BACTERIOLOGICAL CONTAMINATION OF DRINKING WATER IN DIFFERENT SEASONS IN DHAKA CITY

M. G. Kibria [1], M. S. Alam [1], M. N. Amin [2] & M. R. Amin [1*]

[1]*Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.*
[2]*Department of Sociology, Govt. M. M. College, Jessore, Bangladesh.*
**Corresponding Author, <ruhulbuet@gmail.com>*

ABSTRACT

This study was devoted to acquire a better understanding of the quality of water supplied by DWASA in Dhaka city. The quality was enumerated on the basis of bacteriological contamination. Water samples are collected from forty-two different locations in two different seasons of Dhaka city. The water samples were mainly tap water. The samples were collected in two different seasons thus providing an understanding of water quality in different environmental conditions. It was found that the microbial quality of drinking water supplied in Dhaka city varied in different seasons. The microbial quality of drinking water was found relatively better in sunny season than dry season. It was also found that older areas were subjected bacteriological contamination more than the newly developed areas. Micro-organism concentration was found less in residential areas than commercial and industrial areas. Data and results from this study have showed that as a whole the drinking water quality of Dhaka city is not satisfactory. The potential contamination of drinking water may lead to dreadful situation. It is essential to take necessary steps to eliminate the forthcoming threat to our existence.

Keywords: DWASA, Water Quality, Bacteria.

INTRODUCTION

Bangladesh, the 1,44,000 square kilometre delta in the Bay of Bengal, is inhabited by 170 million people who are mostly illiterate and are among the lowest per capita income-generating group of the world. Rivers, ponds, ditches etc. have been the sources of drinking water for ages. In comparison to the problem of microbial and heavy metal pollution of water is increasing with the increase in population, urbanization and indiscrimination discharge of wastewater. Water is used not only for drinking purpose but also other various purposes. The risk of using this water is greatly dependent on the concentration of micro-organisms and the extent of heavy metals especially arsenic, lead and chromium. This study is primarily concerned with contamination of drinking water by bacteria. In Bangladesh, supplied drinking water is commonly associated with contamination caused by bacteria. The quality of drinking water is greatly dependent on the extent of these micro-organisms. So, there is an urgent need to assess the quality of drinking water supplied by DWASA.
The specific objectives of the proposed study are as follows:

- To study the quality of drinking water supplied by DWASA regarding bacteriological contamination during different seasons.
- To assess the quality of drinking water supplied by DWASA.
- To have a better understanding of the activities and performance of DWASA.

METHODOLOGY

Sample water for the determination of faecal and total coliform was collected form different regions of Dhaka. Standard bags supplied by the laboratory were used for this purpose. The specialities of these bags are that they are bacteria free and each of them has a volume of nearly 100 ml. Water was collected and afterwards the opening tightly fastened to prevent the entrance of air. The bags carrying the water sample were kept in an ice bag to ensure a temperature appropriate for the existence of micro-organisms in the water. The sampling was done about two to two and half hours prior to the test to ensure the existence of micro-organisms. There are two methods for detection and enumeration of indicator organism - (i) multiple tube method & (ii) membrane filter method. Since it inception, membrane filter method has gained worldwide acceptance because of its high degree of reproducibility, its ability to testing relatively larger volume of sample and for the savings in time to gain definite result, the membrane filter method was selected for faecal coliform analysis in this study.

DRINKING WATER SUPPLY IN DHAKA CITY

In Dhaka, ground water is the main source of water supply. DWASA (Dhaka Water Supply and Sewerage Authority) is entrusted with the task of water supply in Dhaka. Groundwater is extracted through a network of about 395 deep tube-wells and distributed to the city dwellers through a distribution system. Any contamination of groundwater would endanger the entire water supply system in Dhaka. For a long time people have been expressing concerns about the possibility of drinking water contamination. This section will provide a brief description of water supply in Dhaka city by DWASA.

The DWASA service area has been divided into 7 zones of which 6 are in Dhaka and 1 in Narayanganj. According to DWASA sources, the groundwater and the surface water extracted by the WASA are monitored regularly through its own quality control and research division. The quality of groundwater supplied is, in most cases, within the acceptable limits set by the WHO guidelines. There

Table 1: Water production by DWASA

Source	Production capacity (mega litre/day)	Actual production		Source-wise % of production
		Mega litre/day	% of capacity	
Ground water	1239.20	1186.61	95.76%	84.89%
Surface water	310.10	211.21	68.11%	15.11%
Total	1549.30	1397.82	90.22%	100%

is a provision for chlorination to prevent contamination in the supply line. Like groundwater, the surface water is also chlorinated before it is delivered into the supply line. Color, pH, turbidity, odour, temperature, dissolved oxygen, alkalinity, hardness, chloride, residual chlorine, calcium, total coliform and faecal coliform are usually tested at the organization's quality control and research laboratory. In addition, groundwater samples from the deep tube-wells are also tested for arsenic after every six months. The DWASA quality control and research division also collect water samples from the supply lines and test the parameter listed above. Necessary mitigation is adopted if there is any change in the quality of water.

Table 2: Zone-wise performance of deep tube-wells

Zone	Total number of deep tube-wells	number of deep tube-wells in operation	number of deep tube-wells not in operation	Capacity (million litre/day)	Average daily water production		Average hours operated per deep tube-well	
					Total (million litre/day)	Average / deep tube-well (million litre/day)	Monthly	Daily
1	71	69	2	225.70	213.48	3.09	644.04	21.47
2	37	37	0	137.80	136.87	3.70	650.00	21.67

3	61	61	0	191.81	191.81	3.14	659.00	21.97
4	74	74	0	237.21	237.21	3.21	615.36	20.51
5	67	66	1	231.74	231.74	3.51	379.34	22.64
6	77	77	0	188.94	164.06	2.13	625.20	20.84
7	8	8	0	26.00	11.44	1.43	325.50	10.85
Total	395	392	3	1239.20	1186.61	3.03	4198.44	139.95

Many countries in world have developed drinking water criteria and standards. Bangladesh developed the first water quality standards in 1976 based on WHO 1971 International Drinking Water Standards. The revision of Bangladesh Standards for drinking water was felt desirable after publication of the WHO drinking water quality guidelines. The Bangladesh Standard Specification for drinking water (BDS 1240: 1989) was prepared and published by the Bangladesh Standard and Testing Institution (BSTI) for the control of drinking water. The Ministry of Environment and Forest, Government of Bangladesh adopted comprehensive water quality standards for drinking water by Gazette notification in 1997 as Environmental Conservation Rules under the Environmental Conservation Act, 1995. The Bangladesh Drinking Water Standards, 1997 along with WHO guideline values, 1993 are presented in Table 3.

Table 3: Drinking Water Standards

Water quality parameter	Unit	Bangladesh standard, 1997	WHO guideline values, 1993
Coliform (faecal)	No./100 ml	0	0
Coliform (total)	No./100 ml	0	0

RESULTS AND DISCUSSION

In this study, water samples were taken from 42 points of different locations of Dhaka city. The study was carried out in two different seasons -

- From March to May (Summer season)
- From September to November (Winter season)

The seasonal variation of climate and environment has much influence on the quality of water. Though there are different factors that determine the quality of water, effort was made to determine the bacteriological contamination of drinking water supplied by DWASA in Dhaka city. The results found in this regard are very much importance to have understanding of the present condition of drinking water in an urban area like Dhaka city.

Bacteriological Contamination

As stated earlier, Membrane Filter method was adopted for the determination of bacteriological contamination. The results were based on the presence of the amount total coliform and faecal coliform in water. The results are as follows –

March to May: As Bangladesh is a tropical country; the climate is quite hot, sunny and humid in between March and May. The consumption of drinking water is relatively high in this period. The effect of temperature is also an important factor in determining the bacteriological quality of drinking water. Bacteria survive longer in low temperature. But as the temperature increases, the rate of bio-chemical reaction that occurs within the cell also increases. Increased cell activities place added demand on nutrient reserves, which may not be renewed in dilute natural systems, leading to an increase in death rates. As temperature is relatively high in this season, the presence of faecal and total coliform is small because of increased death rate. The results found regarding the amount of total and faecal coliform present in drinking water supplied by DWASA in this season are given below:

Table 4: Amount of total coliform (TC) and faecal coliform (FC) in drinking water in different regions of Dhaka city (March to May)

Zone	Location	Sampling point	TC no./100 ml.	FC no./100 ml.
I	Gandaria	Bangladesh Bank Colony	TNTC	300
	Narinda	2, Narinda road	-	-
	Lakhsmibazar	6, Govinda Datta lane	TNTC	200
	Bangshal	71, Nawabpur road	10	-
	Islambag	32/1, Islambag	-	-
	Jatrabari	Johra market	50	-
	Dhania	568, Dhania	TNTC	-
II	Rayerbazar	10, Rayerbazar	TNTC	32
	Hazaribagh	25, Hazaribagh	2	-
	Nawabganj	10, Nawabganj	TNTC	400
	Azimpur	35/2, Duri Anguli Lane	30	-
III	Dhanmondi	Road # 19, House # 166-167	TNTC	300
	Lalmatia	2/8, Block # F	-	-
	Zigatala	67/2, Zigatala	6	-
	Mohammadpur	35/2, Tajmahal road	TNTC	12
IV	Mirpur-12	Block # A, Plot # 6, House # 2	TNTC	TNTC
	Pallabi	Block # A, Plot # 9, House # 6	TNTC	32
	Mirpur-6	Block # C, Road # 8, House # 8	180	130
	Mirpur-1	132, Darus Salam road	TNTC	26
	Paikpara	BADC Quarter	-	-
	Kallyanpur	62, Kallyanpur	60	-
	Shyamoli	Shyamoli apartment	150	-
	Uttara-8	Postal colony	TNTC	300
	Uttara-1	Road # 9, House # 2	200	8
	Ashkona	Customs colony	250	30
	Khilkhet	Khilkhet bazar	300	80
	Moghbazar	682, Moghbazar	35	-

Zone	Location	Sampling point	TC no./100 ml.	FC no./100 ml.
V	Badda	59/Ja, Badda	90	8
	Rampura	6/7, East Rampura	100	-
	Khilgaon	320/A, Tilpapara	5	-
	Shahjahanpur	223, North Shahjahanpur	50	20
	Malibagh	315/1/E, Shantibagh	20	-
	Siddeshwari	25, Siddeshwari	5	2
	Segunbagicha	Segunbagicha Complex	-	-
	Gulshan-2	Road # 6, House # 4	-	-
	Banani	Road # 2, House # 2	50	-
	Cantonment	56, Mominul road	200	40
	Tejgaon	68/1, Tejkunipara	TNTC	90
VI	Gopibagh	17, Gopibagh	-	-
	Sayedabad	68/2, Sayedabad	150	16
	Goran	368, South Goran	200	70
	Bashabo	49, North Bashabo	40	-

[*TNTC = too numerate to count]

If the results are analysed, it will be seen that Zone-IV is subjected to be affected by coliforms relatively more than the other zones. Water sample of some of the areas were found not to have any coliform. But it does not imply that the water of that particular area is free from coliforms. It may be due to the effect of chlorination, a disinfection process, just prior to the collection of sample.

The results found regarding presence of coliforms imply that the bacteriological quality of water supplied by DWASA needs proper treatment to make the water better. As indicated earlier, Bangladesh standard for microbial quality of drinking water is 0 per 100 ml. and 0 per 100 ml. for total and faecal coliform respectively. So, it was seen that the water samples collected from different areas are very much affected by coliforms. In some areas, e.g, Gandaria, Lakhsmibazar, Nawabganj, Mirpur-12, Uttara-8, the situation is alarming.

September to November: The period of time from September to November can be termed as the winter season. In this time, the weather is quite cool. Mainly this part of year ranges from late autumn to the early winter period. The average temperature is moderate to low. This season, which is also called dry season, is favourable for the increasing growth of pathogens. So, the population of coliforms remain almost unchanged. The degradation of quality of drinking water is extreme in this season. The results found regarding the amount of total and faecal coliform present in drinking water supplied by DWASA in this season are given below:

Table 5: Amount of total coliform (TC) and faecal coliform (FC) in drinking water in different regions of Dhaka city (September to November)

Zone	Location	Sampling point	TC no./100 ml.	FC no./100 ml.
I	Gandaria	Bangladesh Bank Colony	TNTC	200
	Narinda	2, Narinda road	50	20
	Lakhsmibazar	6, Govinda Datta lane	TNTC	250

Zone	Location	Sampling point	TC no./100 ml.	FC no./100 ml.
	Bangshal	71, Nawabpur road	25	5
I	Islambag	32/1, Islambag	10	-
	Jatrabari	Johra market	60	10
	Dhania	568, Dhania	TNTC	50
II	Rayerbazar	10, Rayerbazar	TNTC	40
	Hazaribagh	25, Hazaribagh	50	10
	Nawabganj	10, Nawabganj	TNTC	300
	Azimpur	35/2, Duri Anguli Lane	40	5
III	Dhanmondi	Road # 19, House # 166-167	TNTC	250
	Lalmatia	2/8, Block # F	20	-
	Zigatala	67/2, Zigatala	40	20
	Mohammadpur	35/2, Tajmahal road	TNTC	30
IV	Mirpur-12	Block # A, Plot # 6, House # 2	TNTC	TNTC
	Pallabi	Block # A, Plot # 9, House # 6	TNTC	200
	Mirpur-6	Block # C, Road # 8, House # 8	TNTC	30
	Mirpur-1	132, Darus Salam road	200	120
	Paikpara	BADC Quarter	400	20
	Kallyanpur	62, Kallyanpur	80	10
	Shyamoli	Shyamoli apartment	140	-
	Uttara-8	Postal colony	TNTC	200
	Uttara-1	Road # 9, House # 2	240	10
	Ashkona	Customs colony	200	32
	Khilkhet	Khilkhet bazar	300	100
V	Moghbazar	682, Moghbazar	30	2
	Badda	59/Ja, Badda	80	10
	Rampura	6/7, East Rampura	120	10
	Khilgaon	320/A, Tilpapara	20	2
	Shahjahanpur	223, North Shahjahanpur	40	10
	Malibagh	315/1/E, Shantibagh	24	-
	Siddeshwari	25, Siddeshwari	16	2
	Segunbagicha	Segunbagicha Complex	10	-
V	Gulshan-2	Road # 6, House # 4	100	2
	Banani	Road # 2, House # 2	40	4

Zone	Location	Sampling point	TC no./100 ml.	FC no./100 ml.
	Cantonment	56, Mominul road	180	30
	Tejgaon	68/1, Tejkunipara	TNTC	100
VI	Gopibagh	17, Gopibagh	20	-
	Sayedabad	68/2, Sayedabad	170	10
	Goran	368, South Goran	180	80
	Bashabo	49, North Bashabo	50	-

[*TNTC = too numerate to count]

If the data of this dry period is compared with those of sunny season, it will be found that the amount of coliforms present in drinking water has been increased in most of the areas. For example, in the area of Zigatala, the amount of total and faecal coliform was 6 and nil per 100 ml. of water respectively during the period of March to May while the amount of those was 40 and 20 per 100 ml. of water respectively during the period of September to November. So, the bacteriological contamination of drinking water in the dry season should be given prime importance to supply pure and acceptable water in the Dhaka city.

CONCLUSION

The study was mainly based on an effort to identify potential contamination (bacteriological) of drinking water supplied by DWASA in Dhaka city. The results found in this study reflect the present scenario of drinking water on Dhaka. By analyzing the results obtained, the following conclusion may tentatively be drawn:

■ In some of the areas, faecal coliforms were found nil. But it does not imply that drinking water in those areas is free from bacteriological contamination. It may be caused due to chlorination prior to collection of sample water.

■ Bacteriological contamination of drinking water may be caused from the contamination at the source or from the underground reservoir or overhead tanks.

■ Bacteriological contamination of drinking water was found less in residential areas e.g. Lalmatia, Segunbagicha, Siddeshwari, Zigatala, etc. than commercial and industrial areas e.g. Mirpur-12, Nawabganj. Lakhsmibazar, etc.

■ Bacteriological contamination of drinking water was found less in newer areas e.g. Uttara, Gulshan etc. than older areas e.g. Rayerbazar, Nawabganj. Lakhsmibazar, etc.

■ Drinking water is affected by pathogens more in the winter season than the summer season. As lower temperature is favorable for growth of population of coliforms, winter season is susceptible for extreme potential contamination of drinking water.

■ Along with total and faecal coliforms, other bacteria were also found while performing the test. But these bacteria were not taken into account as their population was not significantly large and their presence in water is less indicative of faecal contamination.

■ Drinking water supplied by DWASA needs proper and adequate treatment to make it free from pathogens. The present treatment procedures followed by DWASA are not being able to resolve the problem.

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A STUDY ON RIVER BANK EROSION AND CHANNEL MIGRATION PATTERN OF JAMUNA RIVER USING GIS AND REMOTE SENSING TECHNOLOGY

I. Khan[1], M. Ahammad[1*], S. Sarker[1] & M.S.S. Towhid[1]

[1]*Institute of Water Modelling, Dhaka, 1206, Bangladesh.*
**Corresponding Author, <muneer.buet@gmail.com>*

ABSTRACT

Jamuna River is characterized by its extremely dynamic and unstable alluvial channels. This 240km long braided river is located in lowest reach of the Brahmaputra River in Bangladesh. The Brahmaputra River catchment supplies enormous quantities of sediment from the actively uplifting mountains in the Himalayas, its erosive foothills and the great alluvial deposits stored in the Assam Valley. Large discharge and heavy sediment load during floods causes the Jamuna River to be extremely unstable, because of which it consistently migrates laterally. This tendency of lateral channel migration, results in erosion of the river banks which causes severe problem to the people living in the floodplain of the river Jamuna. This particular study was carried out using the satellite imagery of the last decade as well as some old historical images to show the pattern and extent of channel migration and bank erosion of the Jamuna River. A comprehensive analysis was carried out in this study using the state of the art GIS technology to identify the vulnerable reaches of the river in respect of bank erosion.

Keywords: Morphological change, Channel migration, GIS, Jamuna River, Remote Sensing

INTRODUCTION

The gradient, water volume, water velocity and nature of river are responsible parameters for changing the rivers shape and size. The geomorphologic formation and the physical dynamics of the Brahmaputra basin are subject to runoff from the highest and most tectonically active mountain range in the world, the Himalayas. The combination of large and variable discharges of water and sediment is responsible for the Jamuna’s braided pattern which is characterized by unstable banklines and rapid rates of lateral movement. River bank erosion is treated as one of the foremost natural disasters responsible for poverty in Bangladesh due to the enormous destruction of resources and displacement of large numbers of the population. The hazard also has an impact on unemployment levels in rural Bangladesh. Hence, study on bank erosion and identification of vulnerable reaches, specially for the case of Jamuna, has great importance as it provides the opportunity to adopt possible strategies that may assist in mitigating the human suffering and adverse socio-economic impacts of these recurring natural events.

There is growing trend in research community to apply the various high tech techniques for human being's problems identification and to seek its solutions. GIS and RS are important tools which can aid in identifying river changes and bank erosions situations. This computer oriented techniques provides a platform to analyze spatially varied data and helps to find influential results. This study is aimed at quantifying the actual rates of bank erosion along the river Jamuna based on time series analysis of satellite images. This work also focuses on pattern of the change of channel alignment and identification of the reaches which are under potential risk of bank erosion as well.

STUDY AREA

The Jamuna with 240km long reach, from Noonkhawa to Aricha, is the lowest reach of the Brahmaputra River in Bangladesh. The Study area is located between the longitude 89°30′ E and 89°50′ E and latitude 23°50′ N and 25°15′ N. (Figure 1).

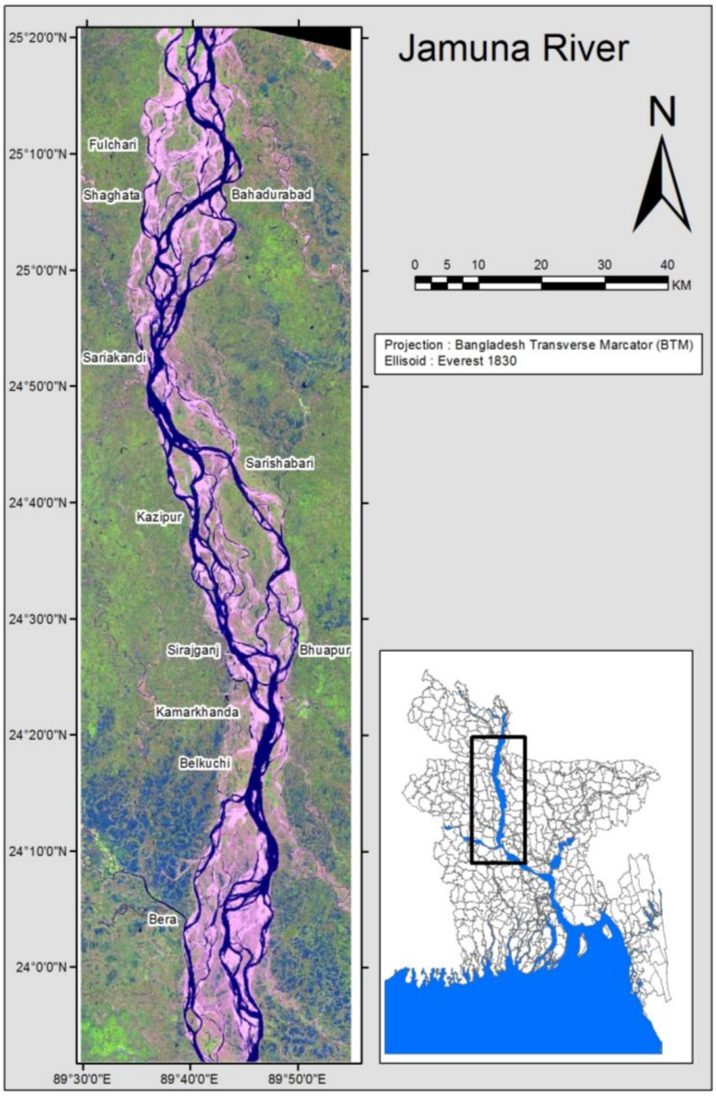


Figure 1: Location Map of the Jamuna River in Bangladesh

DATA AND METHODOLOGY

Identification of the channel migration pattern of rivers from satellite images of different years using GIS and Remote Sensing technology is found very much useful for studying the fluvial geomorphology of a river. For the purpose of this study the dry season satellite images from 1973 to 2014 have been collected. The details of these images are shown in Table 1. Bank lines of these years were digitized from the geo-referenced satellite imageries using the ARC GIS and erosion-accretion pattern due to the lateral movement of the active river channel have been estimated using the GIS software. Along with these, all the major channels of the Jamuna River were also digitized to show the channel migration pattern of this highly morphologically active river.

Table 1: List of Images used in the study and its properties

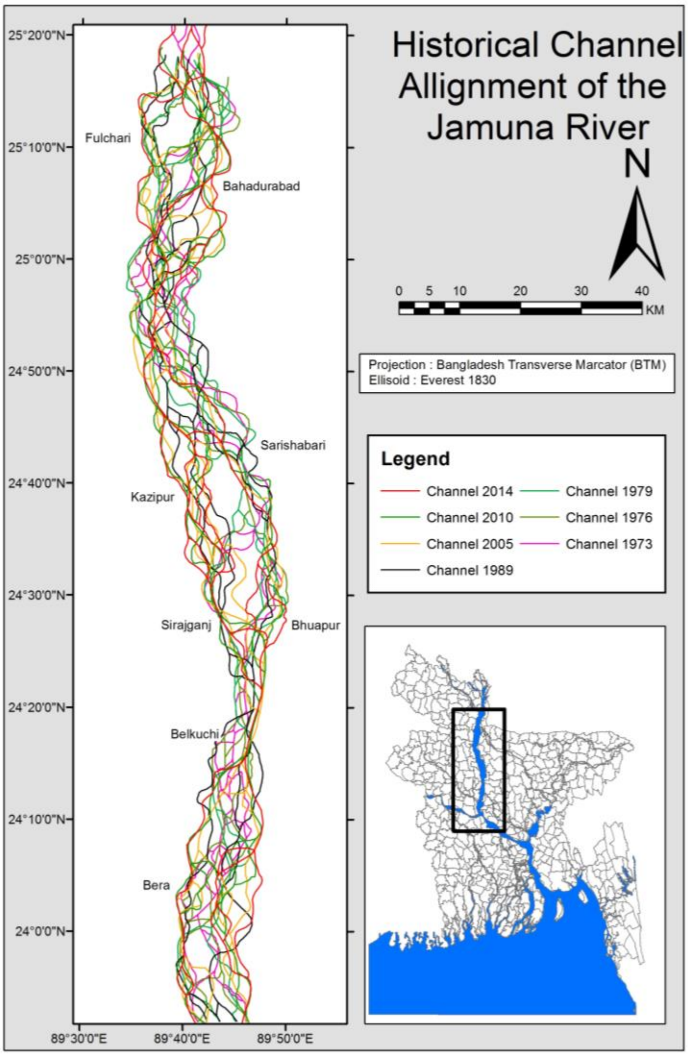
Image Year	Satellite	Ground resolution	Source
1973	LANDSAT (LM) 1	60m x 60m	USGS
1976	LANDSAT (LM) 2	60m x 60m	
1979	LANDSAT (LM) 3	60m x 60m	
1989	LANDSAT (LT) 4	30m x 30m	
2004	LANDSAT (LT) 5	30m x 30m	
2005	LANDSAT (LT) 5	30m x 30m	
2006	LANDSAT (LT) 5	30m x 30m	
2007	LANDSAT (LT) 5	30m x 30m	
2008	LANDSAT (LT) 5	30m x 30m	
2009	LANDSAT (LT) 5	30m x 30m	
2010	LANDSAT (LT) 5	30m x 30m	
2011	LANDSAT (LE) 7	30m x 30m	
2012	LANDSAT (LE) 7	30m x 30m	
2013	LANDSAT (LE) 7	30m x 30m	
2014	LANDSAT (LC) 8	30m x 30m	

CHANNEL ALIGNMENT

The historical channel alignment is shown in Figure 2. The figure clearly shows the dynamic braided nature of the Jamuna River. Due to the high sediment of load of the river, the channel pattern becomes highly irregular and changes abruptly. On the other hand a generalized output of the river bankline for the year 1973 and 2014 is shown in Figure 3. From this figure we can see near Sariakandi there is tendency of the channel to move towards the right bank. However near Bhuapur the channel is moving towards the left bank side and then takes more of a straight shape after Sirajganj.

BANKLINE MOVEMENT

The historical bankline movement of the Jamuna River is shown in Figure 5. An interesting thing visible from this Figure as well as from Figure 3 is that the average width of the Jamuna River is showing an increasing trend. To verify this observation the average width of the river for all the available satellite images were determined using the digitized banklines. A plot was prepared showing the changing width of the Jamuna river which is shown in Figure 4. The Figure clearly shows the increasing average width of the river.



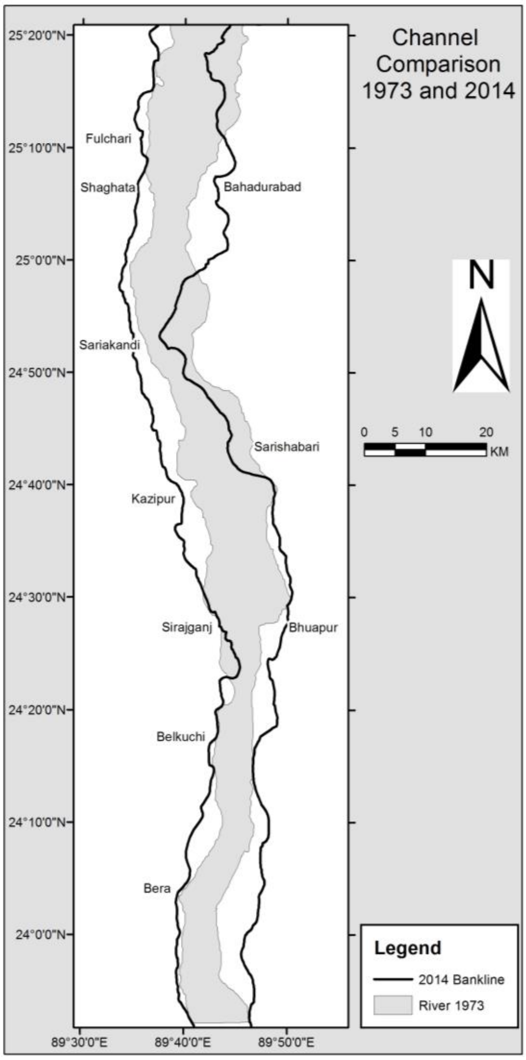


Figure 2: Historical Channel alignment of the Jamuna River

Figure 3: Channel Comparison 1973 and 2014

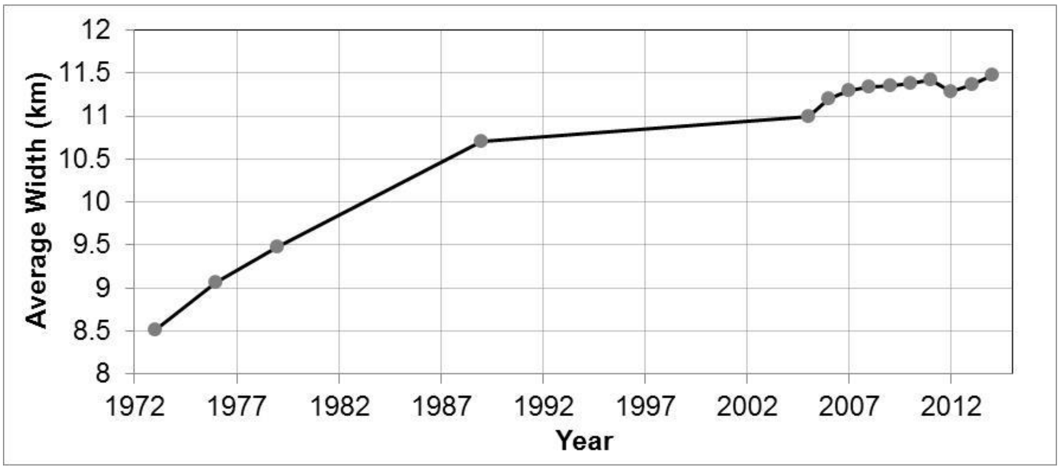


Figure 4: Average width of the Jamuna River

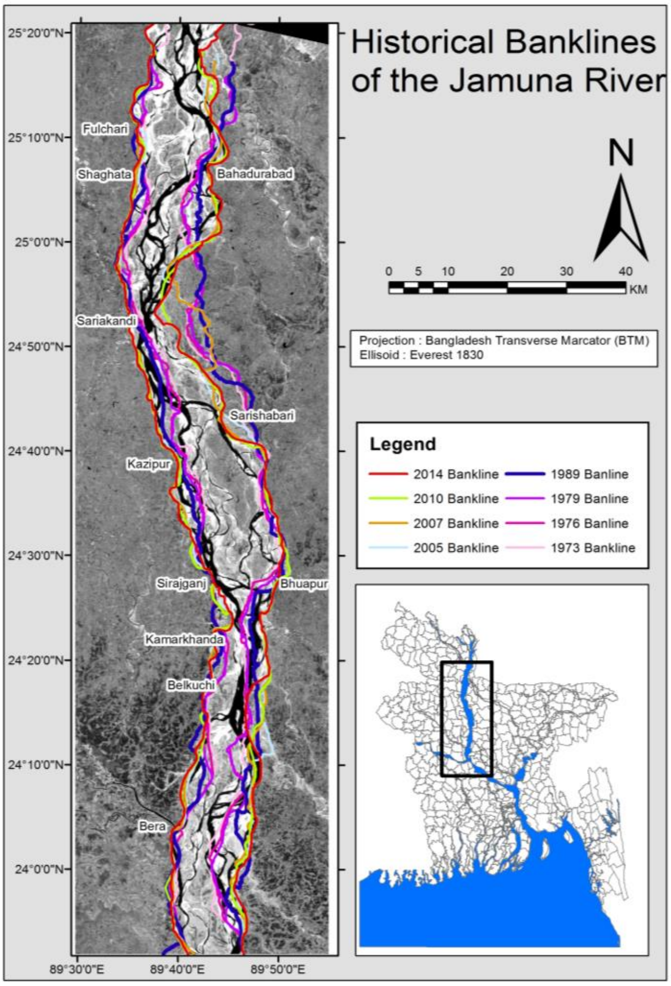


Figure 5: Historical Bankline of the Jamuna River

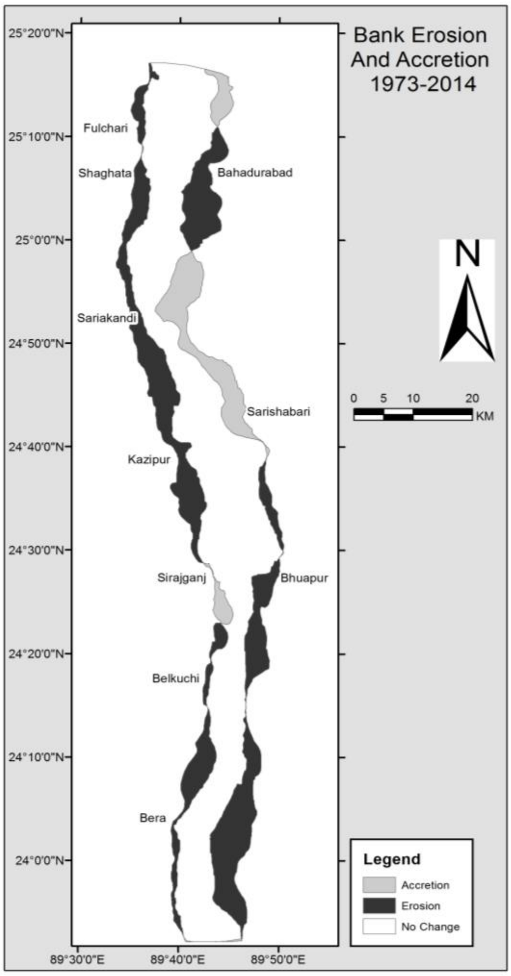


Figure 6: Bank Erosion and Accretion extent between 1973 and 2014

EROSION AND ACCRETION

The area of erosion and accretion due to the river bank movement has been calculated with the aid of GIS for the purpose of this study, the details of which is shown in Table 2. On the other hand the details of the areas where erosion and where accretion has taken place over the last forty years are shown in Figure 6. It is visible from the Figure that in most places the unstable river bank has caused river bank erosion. However around about 15km downstream of Bahadurabad, up to Sarishabari has shown accretion tendency. Over here around about 125km^2 of land has accreted over the last forty years. Interestingly although the summation of the year wise erosion shown nearly 1000km^2 of erosion, the net erosion is actually around 607km^2. This is because some of the area eroded in one flood may get aggraded in the next flood. The similar thing is also seen in case of accretion where the net accretion is found to be 166km^2.

Table 2: The Details of Erosion/Accretion of the Jamuna River over the last 40 years

Year	Erosion (m^2)	Erosion (km^2)	Erosion Per Year (km^2)	Accretion (m^2)	Accretion (km^2)	Accretion Per Year (km^2)
1973-76	107138927	107.14	35.71	50248130	50.25	16.75
1976-79	88715740	88.72	29.57	14011496	14.01	4.67
1979-89	296898479	296.90	29.69	23532147	23.53	2.35

Year	Erosion (m^2)	Erosion (km^2)	Erosion Per Year (km^2)	Accretion (m^2)	Accretion (km^2)	Accretion Per Year (km^2)
1989-2005	235668510	235.67	14.73	179607166	179.61	11.23
2005-06	23550654	23.55	23.55	43754557	43.75	43.75
2006-07	39858172	39.86	39.86	14559736	14.56	14.56
2007-08	35575796	35.58	35.58	89977507	89.98	89.98
2008-09	33761060	33.76	33.76	14145263	14.15	14.15
2009-10	19606770	19.61	19.61	25937682	25.94	25.94
2010-11	42639525	42.64	42.64	39380559	39.38	39.38
2011-12	5941332	5.94	5.94	37135304	37.14	37.14
2012-13	25346899	25.35	25.35	9645637	9.65	9.65
2013-14	43952023	43.95	43.95	15585053	15.59	15.59

CONCLUSION

Based on this study following conclusions can be drawn,

- The overall channel pattern of the Jamuna River is shown a rightward shifting tendency near Sariakandi and left ward moving tendency near Bhuapur.
- The average width of the Jamuna River is increasing and over the last forty years the average width has increase around about 3 km.
- The net erosion was calculated as 607km^2 while the net accretion was found to be 166km^2.
- Most of the accretion has taken place from 15km downstream of Bahadurabad up to Sarishabari.

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APPLICATION OF 1D MODEL TOWARDS ESTABLISHING FLOW PATTERNS FOR THE SOUTHWEST COASTAL REGIONS OF BANGLADESH

M. M. Rahman[1*], M. Billah[1], A.K.M. S. Islam[1], S.Paul[1], M. A. Hasan[1], G.M. T. Islam[1], S. K. Bala[1], & M. A. A. Pavel[1]

[1]*Institute of water and flood management, BUET, Dhaka.*
**Corresponding Author, <mostafizsust@gmail.com>*

ABSTRACT

Around one-fourth of the population of Bangladesh live in coastal regions where water plays an important role in socio-economic and livelihood developments. The southwest region is known as the zone of vulnerabilities because the combination of natural and man-made hazards, such as, storm surge, cyclones, floods, water logging, water and soil salinity, risks from climate change, etc., have adversely affected lives and livelihoods of the region. It is necessary to conduct biophysical modelling to understand the underlying processes, which lead towards assessment of coastal vulnerabilities. Hydrodynamic modelling is used to assist quantifying the flows, sediment and salt loads carried by the coastal river systems. To simulate numerical hydrodynamic models, discharge data at the lateral boundaries of the model domain is required. Nevertheless, discharge measurement station in the coastal region is not available. The only available hydrologic data are water level stations data measured by Bangladesh Water Development Board. Therefore, HEC-RAS is very important for generating discharge data of the coastal rivers. HEC-RAS is designed to perform 1D hydraulic calculation. With a geo-referenced modelling of river channel networks, it can be used for assessing the river flow. In this regards, this study has made an attempt to establish a real river network of the southwest region of Bangladesh using 1D model HEC-RAS for flow analysis.Coefficient of determination (R2) between observed and modelled water level has been found as 0.98, 0.97,0.99,0.85,0.84,0.61,0.92 and 0.88 at Kamarkhali, Goalunda Transit, Gorai Railway Bridge, Mawa, Bagerhat, Kabirajpur, Gournadi and Khulna respectively. However, except three locations, NSE for all locations have been found greater than 0.5 that indicate simulated water level are close to observed mean. Another statistical parameter (RMSE) has been analyzed to find the error between simulated and observed water level. Most of the stations have shown good result according to this statistical parameter.

Keywords: Hydrodynamic Model, Flood Simulation, HEC-RAS, Nash Sutcliffe Efficiency, Coastal Region.

INTRODUCTION

Natural and man-made hazards, such as, storm surge, cyclones, floods, erosion, high arsenic content in groundwater, water logging, earthquake, water and soil salinity, various forms of pollution, risks from climate change, etc., have adversely affected lives and livelihoods of the south west region of Bangladesh, where one-fourth of the population of this country reside. Water, in this region, plays a significant role in both socio-economic and livelihood developments. The discharge measurement data is unavailable for the stations in the coastal region. Although the water level data is available on those locations which have been measured by BWDB (Bangladesh Water Development Board). The objective is to calibrate the channel roughness coefficient (Manning's 'n' value) for the rivers in this region using 1D hydrodynamic modelling, HEC-RAS. HEC-RAS is an open source software which is user friendly, designed to perform one dimensional hydraulic calculation (HEC-RAS, 2010). HEC-

RAS can be used for assessing the river flow of the coastal zone with a geo-referenced modelling of river channel networks.

Depending upon the variation in channel characteristic along the flow, the channel roughness shows variations along the river. HEC-RAS has been extensively used all over the world to develop hydraulic model by calibrating the channel roughness for different rivers (Patro et al., 2009; Usul and Burak, 2006; Vijay et al., 2007 and Lal,1995). Single channel roughness value, using optimization method, has been estimated for open channel flow by taking the boundary condition as constraints (Ramesh et al., 1997). Channel roughness has been calibrated for Mahanadi River, India (Parhi et al., 2012) and for Lower Tapi River, India (Timbadiya et al., 2011) using HEC-RAS model. Once calibrated, the model can be utilized for flood inundation mapping and flood forecasting mapping of the southwest region which will enable the concerning authorities to take necessary precautions to save lives and properties therein. It can also be used for coupling with other models which may enable us to understand the other river dynamics involved.

METHODOLOGY AND DATA

The real river networks of the southwest region of Bangladesh have been selected as a study area of this study as shown in Fig. 1. The major rivers of this region are the Ganges, the Padma and the Lower Meghna and others river are the Arialkhan, the Gorai, the Kaliganga, the Chandana, the Kumar, the Sitalakhya, the Madhumati, the Bhairab, the Pussur, the Bishkhali, the Tentulia, the Baleswar and the Burishwar etc. Firstly, the whole river networks of southwest region have been digitized in the Google Earth software. The river networks that are obtained as KML files from the Google Earth, have been converted into shape files using the ArcGIS software. The river name, reach and junctions have been assigned along the south-west river network using HEC-GeoRAS extension. The corrected networks have been imported in HEC-RAS software. In HEC-RAS, data of all the cross sections which have been collected from BWDB (Bangladesh Water Development Board) have been set up for the whole river network of the southwest region.

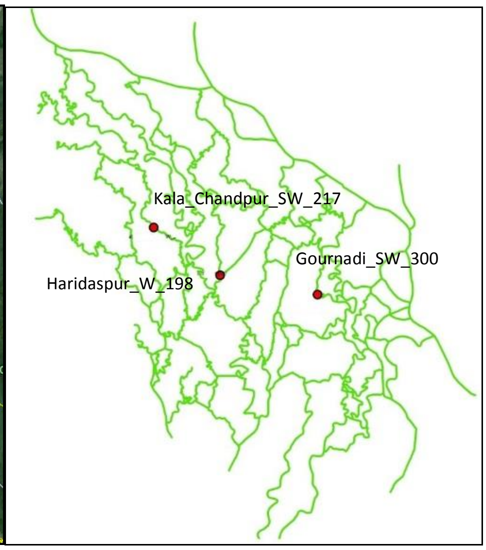

Figure 1: The southwest river network of Bangladesh (left side) and Three gauge station used for calibration (right side)

Boundary conditions are applied to define the inflows and outflows at the model boundary. Boundary fluxes are expressed in terms of mass and momentum exchanges. For hydrodynamic modeling, the boundary conditions are commonly specified at inflow and outflow elements of the model domain. Numerically, three types of boundary conditions are identified: Dirichlet condition (specified head boundary), Neumann condition (specified flow boundary), and Cauchy condition (head-dependent flow boundary). The Cauchy boundary condition that is also called mixed boundary condition relates heads to flows at the outflow elements. The flow is computed based on the difference between specified heads outside the model domain as supplied by the user, and the computed heads at the boundary elements (Alemseged and Rientjes, 2007).The boundary condition of the HEC-RAS model has been established from the observed upstream discharge data obtained from Hardinge Bridge station at the Ganges, the Bahadurabad station at the Jamuna, and Bhairab Bazar station at the Upper Meghna have been selected as flow hydrograph data and water level data obtained from Shahabaz, Tentulia, Buriswar, Bishkhali, Baleswar and Mongla stations have been selected as stage hydrography. The data concerning the floods for years 1998 have been used for calibration of Manning's roughness coefficient, "n".Three gauging stations, Kala Chandpur, Haridaspur and Gournadi have been chosen to perform the calibration of roughness coefficient (Manning's "n") of the corresponding channels. The stations have been pointed out in figure 1. Finally, the model has been simulated for six-month hydrograph and as an unsteady flow and four months has been used as warm period which is necessary to stabilize model.

Model Description

In the present study, unsteady, gradually varied flow simulation model i.e. HEC-RAS, which is dependent on finite difference solutions of the Saint-Venant equations (Equations (1)-(2)), has been used to simulate the flood in the South West river network of Bangladesh.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA\frac{\partial H}{\partial x} + gA(S_0 - S_f) = 0 \qquad (2)$$

Here A = cross-sectional area normal to the flow; Q =discharge; g = acceleration due to gravity; H = elevation of the water surface above a specified datum, also called stage; S_0 = bed slope; Sf = energy slope; t = temporal coordinate and x = longitudinal coordinate. Equations (1) and (2) are solved using the well known four-point implicit box finite difference scheme (HEC-RAS, 2010). This numerical scheme has been shown to be completely non dissipative but marginally stable when run in a semi-implicit form, which corresponds to weighting factor (θ) of 0.6 for the unsteady flow simulation. In HEC-RAS, a default θ is 1, however, it allows the users to specify any value between 0.6 to 1. The box finite difference scheme is limited to its ability to handle transitions between subcritical and supercritical flow, since a different solution algorithm is required for different flow conditions. The said limitation is overcome in HEC-RAS by employing a mixed-flow routine to patch solution in sub reaches (HEC-RAS, 2010).

RESULT AND DISCUSSION

In the present study, an effort has been made to calibrate the HEC-RAS model through setting Manning's roughness coefficient ('n')_ for as a single value for each channel in the network using aforesaid data. Subsequently, different 'n' values have been chosen for each network to justify their adequacy for simulation of flood in the river reaches. Model has been calibrated for floods of year 1998. Nash and Sutcliffe Efficiency (NSE) test has been used for comparison of simulated flow hydrograph with the observed flow hydrograph for various Manning's "n" as used by Moriasi et al., 2007. Cofficient of determination(R^2) describes the proportion of the variance in measured data explained by the model. R^2 ranges from 0 to 1, with higher values indicating less error variance, and typically values greater than 0.5 are considered acceptable (Santhi et al.,2001). R^2 have been yielded ranging from 0.6 to 0.9 which are acceptable. Table 1 presents a summary of the statistical parameters used in this study.

Table 1: Statistical parameter of the model.

Statistical parameter	Coefficient of Determination (R^2)
Locations	Model evulation(1998)
Gouranadi Station	0.79
Haridaspur Station	0.89
Kala Chandpur Station	0.75

The comparison between observed and simulated stage hydrograph at Haridaspur,Gournadiand Kala Chandpur gauging stations for the flood year 1998 is shown in Fig.3. Visual interpretation from all hydrograph shows that peak flow of simulated discharge are overestimated the observed discharge. But the pattern of simulated hydrograph at all locations are similar to the observed pattern. Both Haridaspur and Kala Chandpur stations have shown higher stage than observed during pre-monsoon although Gournadi station represents quite close to the observed stage. From June to mid September, the difference of simulated stage and observed stage is maintained by one meter at Haridaspur station, 0.02m at Gournadi station and 0.3m at Kala Chandpur station. After moonson period, the stage difference between simulated and observed at Haridaspur are higher than at Gournadi and Kala Chandpur stations. After setting model and modifing the Mannings "n" value, the model still have shown bias because the datum of cross-sections are not correct in many locations. Moreover, many cross section of the southwest are not updated according to the present bathymetry of the river.

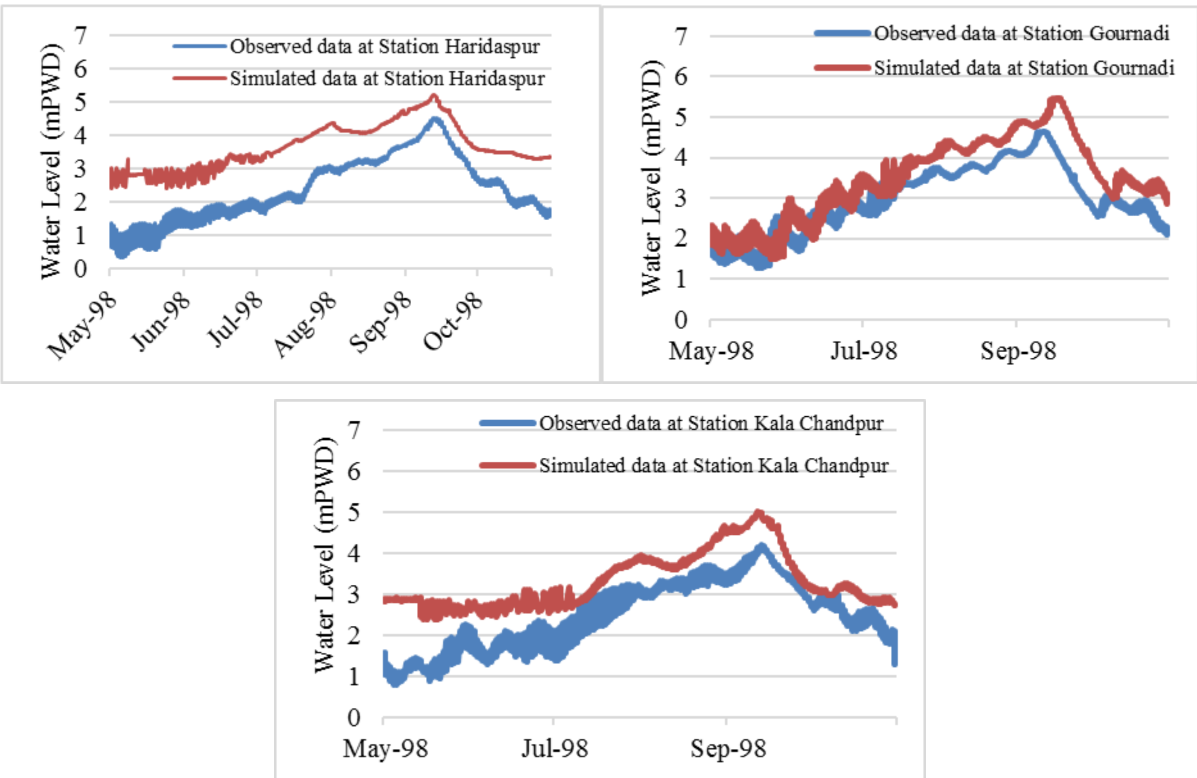

Figure 3: Observed and simulated stage hydrograph at Haridaspur,Gournadi and Kala Chandpur stations.

CONCLUSION

1D model for South-west region of Bangladesh is very important for flood mapping and coupling with 2D costal model. So, to set up and calibrate model is first step for futre impacr study. On the basis of simulation carried out for the river network on the southwest region of Bangladesh following findings can be summarized:

1. Manning's roughness co-efficient of 0.02 can provide better calibration for the rivers..

2. Performance of calibrated and validated model has been assessed through determining the coefficients of determination (R^2) between simulated and observed water level. A close agreement is seen between the simulated and observed water level at station Haridaspur, Gournadi and Kala Chandpur.

3. There needs further modification of the datum of the cross-sections due to subsidence occured in the southwest coastal belt.

ACKNOWLEDGMENTS

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SENSITIVITY ANALYSIS OF DIFFERENT LOCAL SCOUR PARAMETERS AT BRIDGE SITE

A. Reza[1*] & M. Ahammad[1]

[1]*Institute of Water Modelling, Dhaka-1206, Bangladesh.*
**Corresponding Author, <aar@iwmbd.org >*

ABSTRACT

Assessment of possible scour at bridge site is important for design and monitoring of bridge since most of the bridge failure occurred due to scouring action. This study mainly focuses on river analysis and scour due to various changing hydraulic conditions in the vicinity of Bridge on Ghior Khal. To obtain the hydraulic variables (water depth, velocity, etc.) for local scour computation near the bridge site on Ghior Khal, a two-dimensional hydrodynamic model of IWM has been used which was based on pre-monsoon 2005 bathymetry covering 14 km reach of Ghior Khal and its surrounding floodplains. Model set up in MIKE21C includes the preparation of the computational grid, superimposing the bathymetry into this grid and preparation of the boundary data. Using hydraulic variables from this model result, the maximum local scour depth was computed at bridge location of Ghior Khal in three methods (FHWA, Laursen’s and Breuser’s) for piers and in two methods (Simon, Sentruk’s and Liu et al) for abutments. To investigate the behavior of local scour against different parameters a sensitivity analysis had been made in the present study for circular pier. It is found that for pier scour, the Laursen's Method gives the highest scour depth, FHWA's Method gives the lowest scour depth and the Breusers Method gives the value in between. It is also observed that velocity is of greater sensitivity than pier width followed by flow depth and the effect of angle of attack of flow is insignificant.

Keywords: Bridge Scour, Local Scour Formula, Ghior Khal, Sensitivity of Scour Parameters.

INTRODUCTION

Scour is the result of the erosive action of flowing water excavating and carrying away material from the bed and bank of streams. Assessment of possible scour at bridge site is important for design and monitoring of bridge since most of the bridge failure occurred due to scouring action. In most severe case, bridge failure may occur which would bring catastrophic effects. So, in-depth research on scour analysis for different bridges is very important. Many investigators had studied the various aspects of scour like temporal and equilibrium scour (Melville and Chiew 1999, Kothiary et al 1992, Johnson and Bilal 1992, Laursen 1963), clear water and live bed scour (Vittal et al 1994, Jain 1981, Laursen 1962), scour in uniform and non-uniform bed materials (Melville and Chiew 1999, Molinas 1998, Wilson 1998, Raudkivi and Ettema 1977), scale effects in pier scour (Kabir et al 2000, Ettema et al 1998a, Ettema et al 1998b, Kabir 1984, Jain and Fischer 1980, Shen et al 1969, Laursen 1963, Laursen 1962) and so on. Cardoso and Bettess (1999) confirmed the validity of Melville's suggestion that scour at abutments on floodplains can be approximated by scour in rectangular channels if an imaginary boundary is assumed, separating the flow in the main channel from that on the floodplain. Khatun (2001) developed an algorithm to predict local scour. Barbhuiya (2004) concluded that investigations on scour at abutments in nonuniform sediments and with armor-layered beds are important. Experimental study of Roy and Matin (2010) showed that scour around circular structure is highest than scour around any other shapes of structure both at floodplain and main channel.

Local scour at piers has been studied extensively in the laboratory. As a result of the many laboratory studies, there are numerous pier scour equations. Scour depth depends on various parameters (such as depth of flow, shape of pier, angle of attack of flow, sediment size etc). Different engineers have evaluated their equations to predict scour depth using different parameters and field of application. Thus to investigate the behavior of local scour against different parameters a sensitivity analysis is being made in the present study for circular pier.

MODEL DESCRIPTION

Ghior Khal has a very dynamic flow condition during monsoon season (July-October). In this respect, a two-dimensional hydrodynamic model had been developed in MIKE21C based on pre-monsoon 2005 bathymetry covering 14 km reach of Ghior Khal and its surrounding floodplains and the model was calibrated for the year 2002. The model computations were made for the flood events representing 1 in 50 year return period. The computational grid generated from the bank lines of the observed bathymetry and satellite imagery has a resolution of 850 cells in the flow direction and 25 cells in the transverse direction. The grid is sufficiently orthogonal and fine enough to resolve detail of the engineering works in both the horizontal and transverse directions.

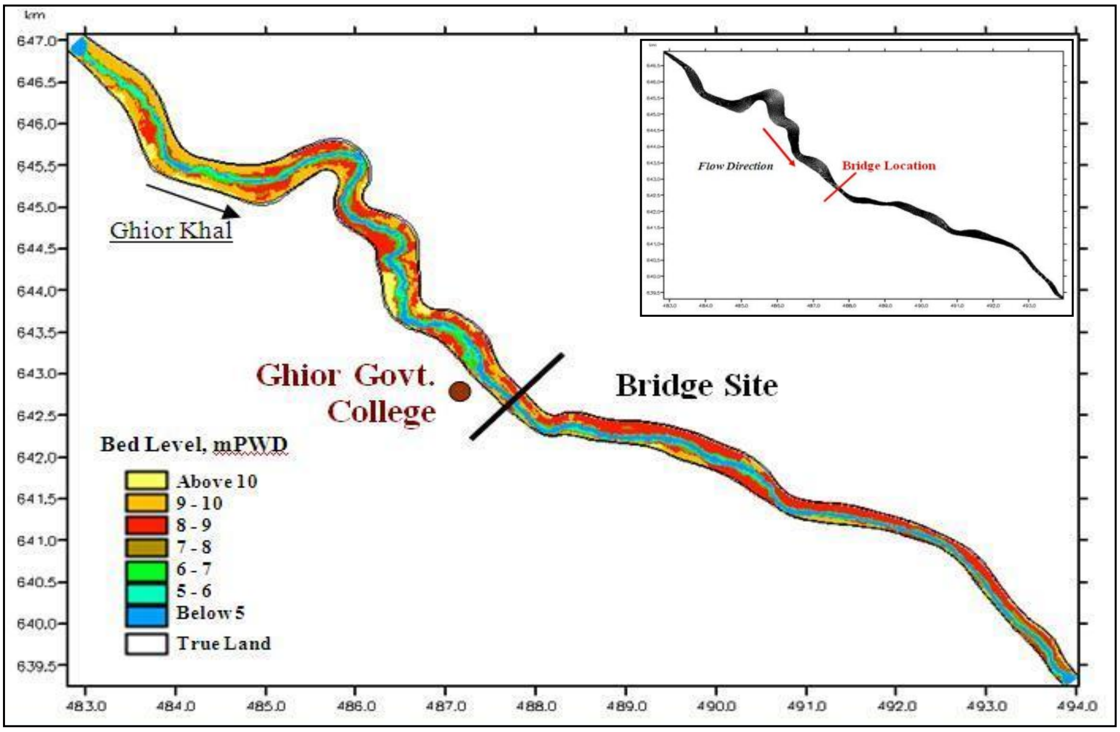

Fig 1: Extent of 14 km long two-dimensional model area with computational grid

From the calibrated model, the following hydraulic variables (in Table 1) were used to estimate local scours.

Table 1: Hydraulic Variables used for local scour computation

Flood event (in return period)	Pier (from right to left bank)	Flow depth directly upstream of the pier, (m)	Mean velocity of flow directly upstream of the pier, (m/s)	Angle of Attack of the Flow, (degree)	Abutment	Average upstream flow depth, (m)	Mean flow velocity (m/s)
	No.	$y_1=Y_1$	$V_1=U$	θ		$h_o=Y_o$	u_o
1 IN 50 YEAR	1	5.90	0.46	0	Right	5.00	0.48
	2	11.10	0.48	0	Left	6.80	0.45
	3	5.80	0.48	0			

SCOUR COMPUTATION AND SENSITIVITY ANALYSIS

Pier and abutment scours are calculated here using different empirical formulas as stated earlier. The formulas are given here in following table.

Table 2: Scour formulas used for computation

Scour Type		Formula	Remarks
Abutment scour	Simon and Sentruk's Formula	$D_s = 4Y_0{*}Fr^{0.33}$	D_s is scour depth (m), Y_0 is average upstream flow depth in the channel (m), Fr is Froude number and u_0 is mean flow velocity (m/s)
	Liu et al Method	$Y_{m,e} = K_L\, h_0\, (b/h_0)\, 0.4Fr^{1/3}$	$Y_{m,e}$ is equilibrium scour depth below mean bed level, Fr is Froude number, h_0 is average upstream flow depth, U_0 is mean flow velocity,b is protrusion of the structure into the channel and K_L is factor for abutment shape
Pier scour	FHWA Method (CSU eq.)	$y_s = 2.0\, K_1K_2\, K_3\, K_4(a_{pier}/y_1)^{0.65}Fr^{0.43}y_1$	y_s is scour depth, a_{pier} is pier width, y_1 is upstream flow depth and K_1,K_2,K_3,K_4 are factors for pier shape, angle of attack, bed condition and armoring of bed material size
	Laursen	$Ds = C.s.b.K_s$	Ds is scour depth, C is correction factor for angle of attack, s is multiplying factor

Pier scour			from Laursen's curve, b is pier width and K_s is shape coefficient
	Breusers	$D_{gb}/b = f(u/u_c)\ 2.0 \tanh(y_1/b)\ K_\theta K_s$	D_{gb} is scour depth, b is pier width, y_1 is upstream flow depth and K_θ, K_s are factors for angle of attack and pier shape

The calculation of abutment scour is shown in Table 3. The maximum abutment scour depths are 8.26 m and 10.45 m respectively for right and left abutment.

Table 3: Local Scour Computation around Bridge Abutments

Data				Calculation		Methods		Maximum
						Liu et al	Simon & Sentruk	
Abutment	Average upstream flow depth, (m)	Mean flow velocity (m/s)	Protrusion of the structure into the channel	Froude Number = $u_o/(gh_o)^{1/2}$	Shape Factor	Scour depth, m	Scour depth, m	Scour depth, m
	$h_o=Y_o$	u_o	b	Fr	K_L	$Y_{m,e}$	D_{s1}	y_{am}
Right	5.00	0.48	7.32	0.07	1.1	1.33	8.26	8.26
Left	6.80	0.45	7.32	0.06	1.1	1.24	10.45	10.45

With the hydraulic variables from model result, pier scour is calculated here in Table 4. Results show that Laursen's method gives most conservative value and CSU equation gives lowest scour depth among the three methods used.

In this study, sensitivity analysis is being carried out in the same table (Table 4) for the different hydraulic variables to check their influence on local scour computation for bridge pier. The analysis is being made by varying (increasing and decreasing) each parameter individually to 10 percent of their original value, but letting other parameters fixed at their respective values. These variable parameters are Grain size (D_{50}), Pier width (b), Flow depth (y), Mean velocity (V_1) and Angle of Attack of the Flow (θ). The different combinations of sets of inputs and the resultant scour depths are given in Table 4. For each case, sensitivity of each parameter on scour depth is calculated by Eq. (1).

$$\text{Sensitivity} = \mathbf{I}\ 100^*\{(\text{new depth} - \text{original depth})/\text{original depth}\}\ \mathbf{I} \qquad (1)$$

From the results obtained from Table 4, sensitivity curves are produced (Fig. 2) to observe and analyse the effect of different hydraulic variables. It summarizes that, here Laursen’s method gives the highest scour depth while FHWA method gives minimum and velocity is of greater sensitivity than pier width followed by flow depth.

Table 4: Sensitivity of Different Local Scour Parameters of Bridge Piers

Data					Methods			Maximum	Sensitivity (%)			
					FHWA	Breuser	Laursen					
Pier width, m	Flow depth directly upstream of the pier, m	Mean velocity of flow directly upstream of the pier, m/s	Angle of Attack of the Flow, deg	Grain size, m	Scour depth, m	Scour depth, m	Scour depth, m	Scour depth, m	FHWA	Breuser	Laursen	Remarks on input
$a_{pier}=b$	$y_1=Y_1$	$V_1=U$	θ	D_{50}	y_s	Dgb	Ds	y_{pm}				
0.80	11.10	0.48	0	0.00014	1.22	1.60	2.64	2.64	0	0	0	Actual Value
0.72	11.10	0.48	0	0.00014	1.14	1.44	2.46	2.46	7	10	7	a_{pier} 10%decreased
0.88	11.10	0.48	0	0.00014	1.30	1.76	2.82	2.82	6	10	7	a_{pier} 10% increased
0.80	11.10	0.48	0	0.00014	1.22	1.60	2.64	2.64	0	0	0	Actual Value
0.80	9.99	0.48	0	0.00014	1.26	1.60	2.56	2.56	3	0	3	y_1 10% decreased
0.80	12.21	0.48	0	0.00014	1.17	1.53	2.72	2.72	4	4	3	y_1 10% increased
0.80	11.10	0.48	0	0.00014	1.22	1.60	2.64	2.64	0	0	0	Actual Value
0.80	11.10	0.43	0	0.00014	1.11	1.28	2.64	2.64	9	20	0	V_1 10% decreased
0.80	11.10	0.53	0	0.00014	1.33	1.60	2.64	2.64	9	0	0	V_1 10% increased
0.80	11.10	0.48	0	0.00014	1.22	1.60	2.64	2.64	0	0	0	Actual Value
0.80	11.10	0.48	0	0.00013	1.22	1.60	2.64	2.64	0	0	0	D_{50} 10% decreased
0.80	11.10	0.48	0	0.00015	1.22	1.60	2.64	2.64	0	0	0	D_{50} 10% increased

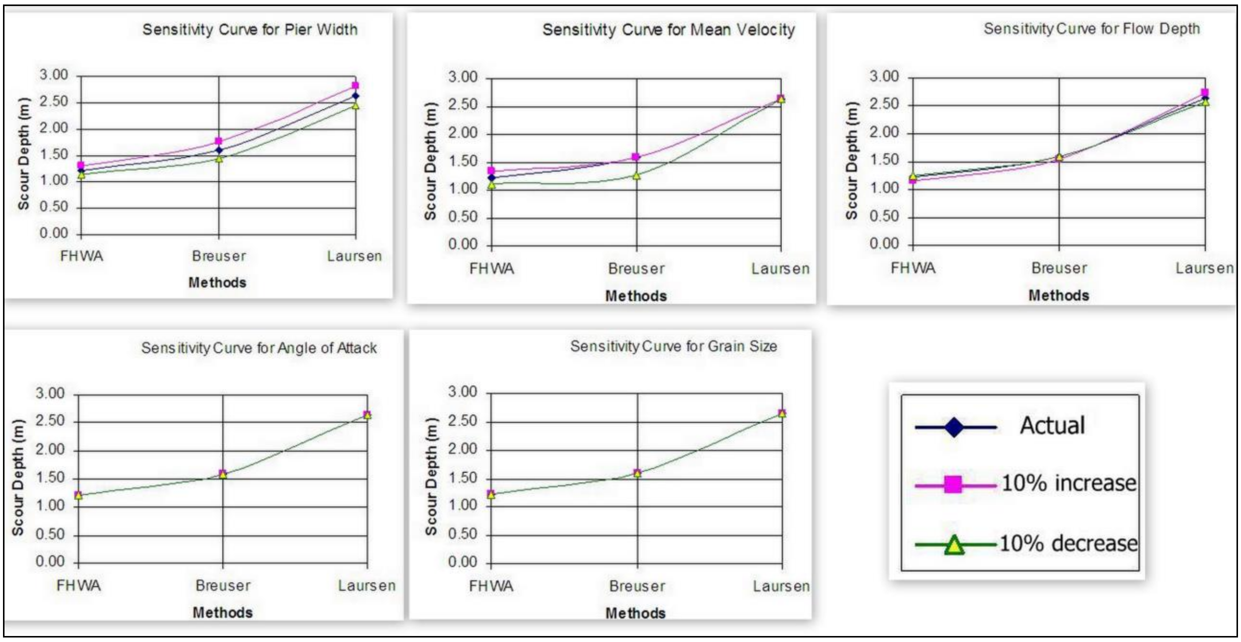

Fig 2: Sensitivity Curves of Different Local Scour Parameters of Bridge Piers

CONCLUSION

Scour formulas should be selected in proper way so that it does not achieve too conservative value of local scour depth, includes variable parameters as many as possible and at the same time, be on the safe side to design structures economically. Three methods (FHWA, Laursen’s and Breusers) to calculate the maximum local scour depth of piers and two methods (Liu et al and Simon & Sentruk's) to calculate the maximum local scour depth of abutments are selected in the present study; among which the Laursen’s Method gives the highest scour depth, FHWA Method gives the lowest scour depth and the Breusers Method gives the value in between. From the comparative analysis, maximum scour depth is selected for design. The sensitivity analysis of pier scour depth is very important to select proper scour formula considering local conditions. However, it can be concluded as follows:

- Flow velocity is greater sensitive than pier width followed by flow depth. In Laursen's method, there is no change if mean velocity is varied.
- Breuser's method is independent of pier width. And other two Methods are of equally sensitivity to the pier width but less than the sensitivity of mean velocity.
- The methods are equally sensitive to the flow depth. And flow depth is less sensitive than pier width, which is again less than the sensitivity of mean velocity.

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ANALYSIS OF RIVER BANK EROSION AND DEPOSITION OF KARNAFULI RIVER IN CHITTAGONG, BANGLADESH USING REMOTE SENSING AND GIS APPROACH

Kutub Uddin Chisty[1*], Md. Ashraful Islam[1] & Md. Shahjalal Misuk[1]

[1] Department of Urban and Regional Planning, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.
**Corresponding Author, <chisty_cuet10@yahoo.com>*

ABSTRACT

An attempt has been made to analyse the bank erosion of Karnafuli River due to regular shifting of river channels by remote sensing approach. Twelve years interval data of Landsat TM & ETM+ Satellite image of 1989, 2001 & 2013, respectively are used to delineate the historical changes of the both bank alignment of the river course. Analysis also showed that last 2001 to 2013 years erosion rate was higher than 1989 to 2001 years and about 1191 feet was eroded. It is also most interpreted that erosion is higher than deposition and more dramatic changes may result from the formation of zig-zag courses. GIS and remote sensing are used to correlate temporal and spatial information so that a comparative analysis has been analyzed among shorelines of different years and tentative changing morphology of the flashy Karnafuli River.

Keywords: River shoreline, Satellite images and GIS

INTRODUCTION

Bank erosion and channel shifting of the untrained alluvial rivers of Bangladesh are major problems to the socio-economic and environmental sectors of the country (Ahmed, 1989). Generally, river bank erosion or deposition is mechanism of sediment transportation or carrying away the bank materials of the river by itself and it affects the river channel courses (Biswajit, et al., 2013). Moreover, it stated that fluvial geomorphic process is very much active most of rivers in Bangladesh. As a result, river erosion is also common in the study area, Karnafuli River. This paper focused on erosion and deposition considering the shifting river shore lines from satellite images of Karnafuli river area. Moreover, Shoreline evolution is the change in shore position during a time scale. In fact, it is the material resistance of the coastal geologic underpinnings against the impinging hydrodynamic forces (Alam, 1991). Most of the change from 1989 year to 2013 year in shore positions will be quantified in this paper only. Thus the erosion and deposition has been clarified by the shifting pattern of shorelines through the satellite images interpolation.

MATERIALS AND METHODOLOGY

To analysis erosion and deposition scenarios of the study area from 1989-2013 by Landsat TM and ETM+ Satellite images and finally GIS approach was adopted, the data of river left bank line is extracted from the same time satellite imagery at different years from US Geological Survey. Fixed in eye altitude of 150 km selecting the study area, 30×30 meter resolution of landslide satellite imagery was acquired from Google Earth-6.2v (table 1). Geographic Information systems (GIS) and Remote Sensing (RS) were used in data encoding and analysing purposes. ArcGIS 9.3 and ERDAS Imagine 10.1 version software and human interpretation have been used in extracting the data from 1989, 2001 and 2013 remote sensing images. Different shape file layers (i.e. District, Upazila, Union Boundary, River, Char land and River Shoreline) for vector data were created and assigned projection in each shape file in the ArcGIS (Arc Catalog) environment. Digitization was performed to collect shoreline or bank line layers from individual year basis. Edge detection techniques give a clear idea about demarcation of land and water boundary.

Satellite Data

River bank lines were derived, using certain precise zooming of pixel where bank lines were demarcated and digitized accurately in between bank line and adjacent water features pixel. Geo referenced Landsat TM and Landsat ETM+ was considered at the preliminary stage to delineate the river banks lines. All the satellite data (fig 1) were acquired on same dates from US Geological survey, 2014.

Table 1: Landsat satellite images at a glance

Respective Year	Data set	Date Acquired (Day/Month/Year)	Center Time (Hour: Minute: Second)	Sensor
1989	LT4136045 1989021XX X03	21-01-1989	03:51:24	Landsat _ 4 Thematic Mapper (TM)
2001	LE7136045 2001038SG S01	20-01-2001	04:09:25	Landsat _ 7 Enhanced Thematic Mapper Plus (ETM+)
2013	LC8136045 2013335LG N00	27-01-2013	04:20:39	OLI_TIRS

Fig 1: Composite band combination (RGB= band 432) of (a) 1989, (b) 2001 and (c) 2013

STUDY AREA

Karnafuli River the largest and most important river in Chittagong and the Chittagong hill tracts, originating in the Lushai hills in Mizoram state of India. It travels through 180 km of mountainous wilderness making a narrow loop at Rangamati and then follows a zigzag course before it forms two other prominent loops, the Dhuliachhari and the Kaptai. After coming out from the kaptai loop the river follows another stretch of tortuous course through the Sitapahar hill range and flows across the plain of Chittagong after emerging from the hills near Chandraghona. therefore, the river drains into the Bay of Bangal cutting across several hill ranges, viz the Barkal, Gobamura, Chilardak, Sitapahar and Patiya of the Chittagong hill tracts and Chittagong (fig 2).

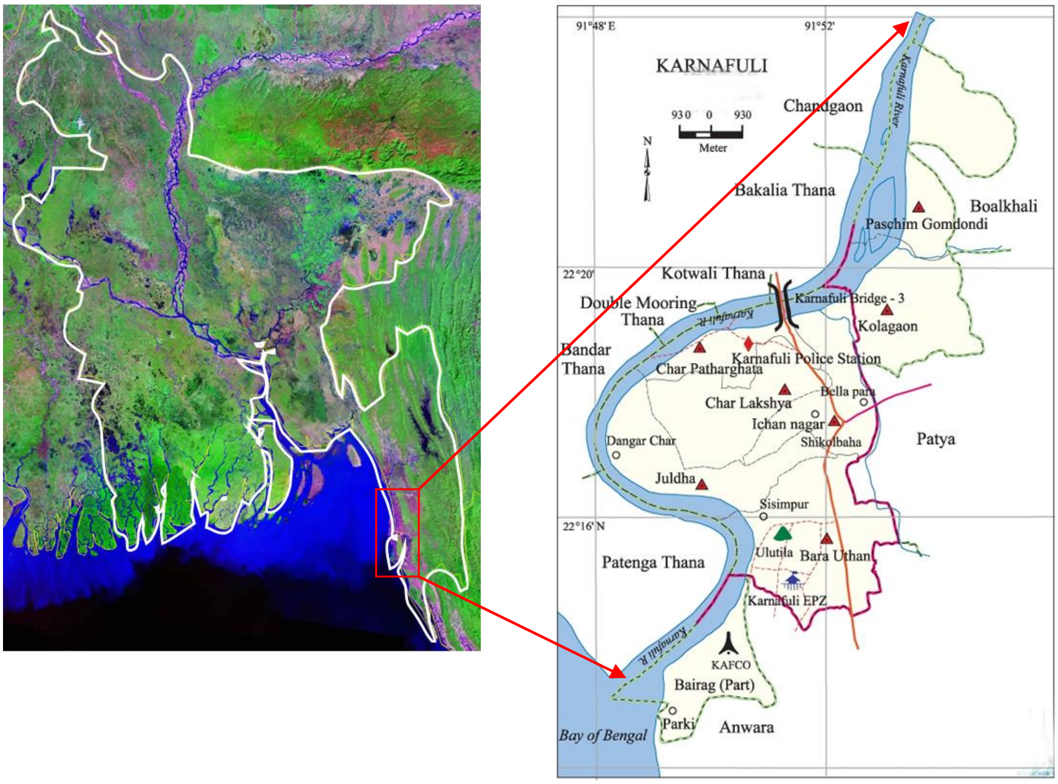

Fig 2 : Flase colour combination map of Bangladesh (left) and Map of Karnafuli river (right)
(Source: US Geological Survey, 2014 [left] and Banglapedia [right])

It has possibly maintained its older course keeping pace with the uplift of the hill ranges and can be classified as an antecedent river. The Karnafuli is narrow and straight from prankiang to Waggachhari along Kaptai-Chandraghona road. The straightness of the river is probably due to a fault, which controlled the channel from prankiang to Wagga. The main tributaries of the Karnafuli are the Kasalong, Chengi, Halda and Dhurung on the right and the Subalong, Kaptai, Rinkeong and Thega on the left. Flowing to the west through Rangunia upazila and then keeping Raozan upazila on the north and Boalkhali upazila on the south, it receives the waters of the Halda river at kalurghat just above the railway bridge (fig 2). It then turns south, receives the waters of the Boalkhali and other khals and turns west circling round the eastern and southern sides of Chittagong town. from the extreme corner of the Chittagong port to the west, it moves southwest to fall into the bay of bengal 16.89 km below (Ahmed, 1989) . It is navigable throughout the year by sea-going vessels up to chittagong port and by large boats, shallow draughts and all sorts of freighters and launches up to Kaptai river in the hill tracts.

THEORETICAL FRAME WORK

If the river is collecting sediment and thus build up its bed known as aggrading type and while the bed is getting years to years then the river formed as a degrading type river (fig. 3). The same river may behave as aggrading or of degrading or even stable type. The silting may be due to various reasons, such as heavy sediment load, construction of an obstruction across the river such as dam. The scouring may be found either above a cut-off or below a dam becoming a degrading downstream. Stable type River does not change its alignment and slope. When a river flows in two or more channels around alluvial islands known as braided river (Garg, 1987).

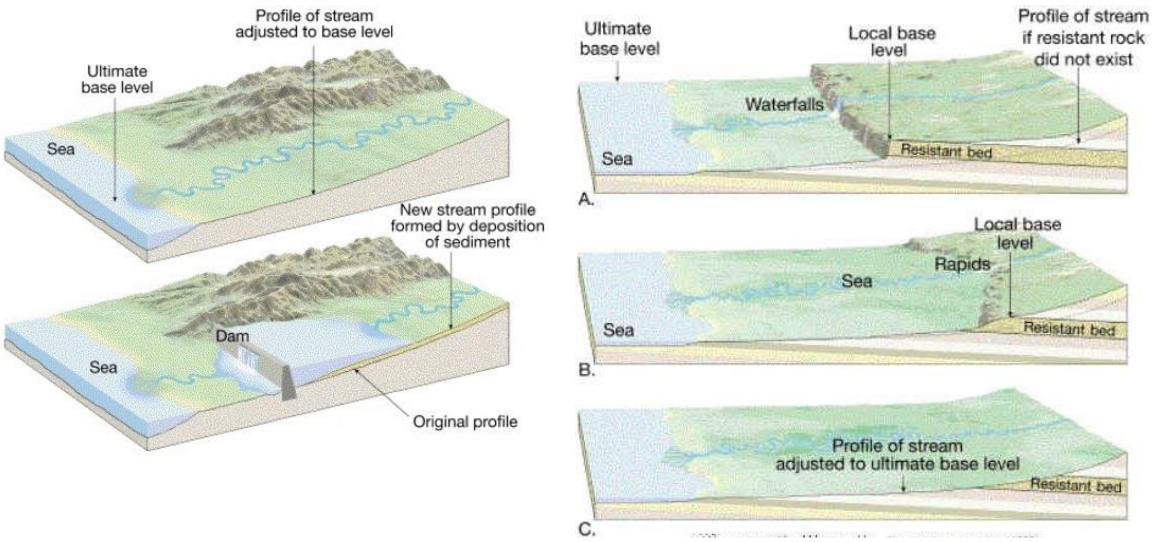

Fig.3: Aggrading river (left) and degrading river (right)

(Ahmed, 1989)

The chief characteristics of alluvial river reaches is the zig-zag fashion in which they flow called meandering (Garg, 1987). The process moves downstream by building up shoals on the convex side by means of secondary currents. The formation of shoals on the convex side, results in further shifting of the outer bank by erosion on the concave side (fig. 4). There was a belief meandering is caused due to the presence of an excessive bed slope in the river that the excess energy is dissipated by increasing the channels length by means of meandering (Garg, 1987). Meandering is also generated by the excess of river sediment during floods (Hossain, 1984).

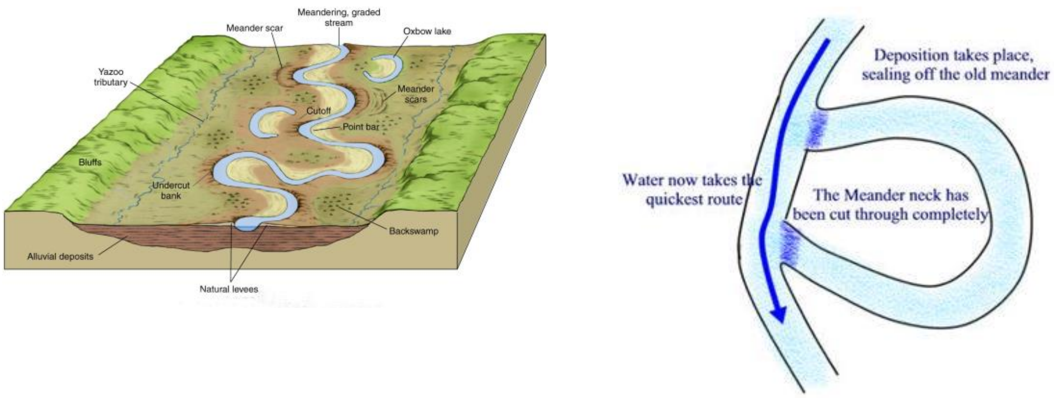

Fig. 4: Meandering of river (left) and cut-off of rivers (right)

(Fujita et al., 2000)

In an excessively meandering river, a particular bend may abandoned by the formation of a straighter and shorter channels and also developed itself is termed as cut-off (fig. 4).

DISCUSSION

Erosion and Deposition area at Karnafuli River

Data of different periods (from 1989-2013) were considered for erosion and deposition area calculation. All of the shoreline layers superimposed one by one maintaining sequential order, there after change detection was performed, demarcated and calculated the erosion and deposition of the study area from 1989 to 2013 .

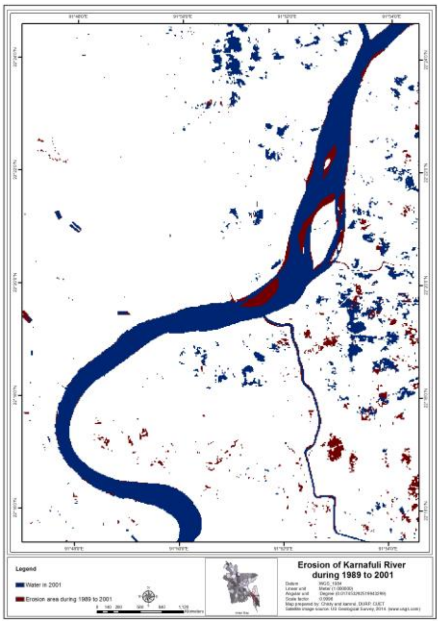

Fig 5: Erosion area during 1989 to 2001

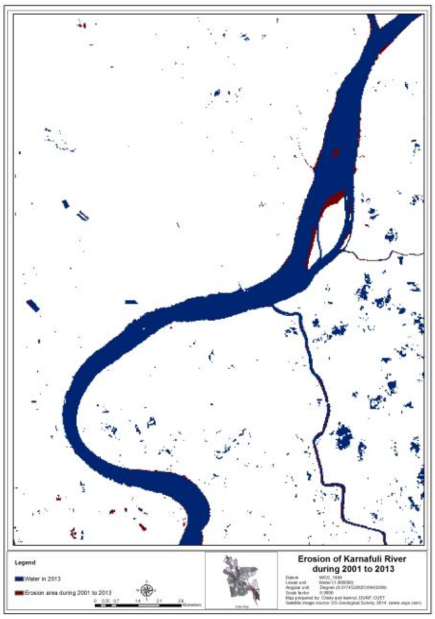

Fig 6: Erosion area during 2001 to 2013

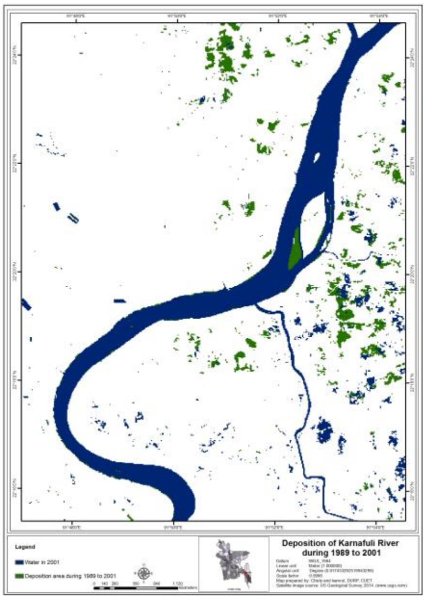

Fig 7: Deposition area during 1989 to 2001

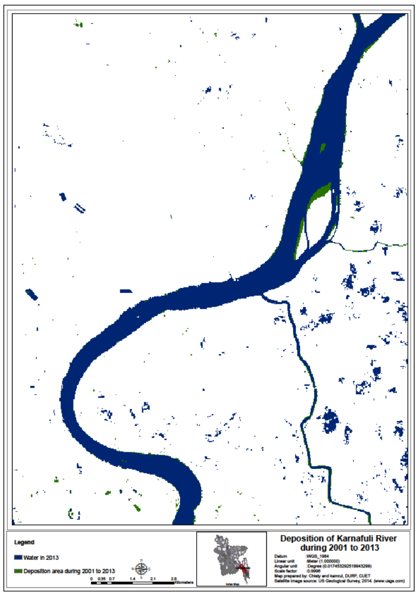

Fig 8: Deposition area during 2001 to 2013

In Chittagong the Karnafuli made the most significant change in its course from Kalurghat downwards. The change has been taking place for more than a century. Formerly, the river had a western and southwestern course from Kalurghat and flowed by Sampanghata, Suloop Bahar, Kapashgola, Chowk Bazar, Roomghata, Ghat Farhadbeg, Boxirhat, Patharghata on its right bank. But gradually it receded to the left throwing up vast and extensive alluvial lands along its right bank, now known as Char Bakalia, Chandgaon, Char Chaktai, etc. The above ghats and bazars which once dotted the right bank of the Karnafuli along the eastern limits of the town are now important localities in and outside the municipality, far away from the present course of the river. This fact is of much historical importance, so far as it helps locate the eastern bounds of the town during the Mughal and early British period.

Shore line deviation of Karnafuli river during 1989 to 2013

The shoreline change map was prepared and the superimposed techniques were applied layer by layer as a vector file base on imagery of 1989, 2011 and 2013. All of the layers superimposed one by one sequential order to detect the change. Previous shifting pattern of shorelines is also demarcated base on imagery of 1989. Satellite imagery shows how the coast has changed, how sandbanks have grown or decayed, how inlets have changed the course and how one shore type has displaced another or has not changed at all. Shore change is a natural process but, quite often, the impacts of man through shore hardening or inlet stabilization come to dominate a given shore reach. Fig 5 to Fig 8 are showing exact scenario of different erosion and deposition area such as Paschim Gomdondi. Others side also demarked through the shoreline changing such as Kaptai Lake which demarked highly shoreline changing zone.

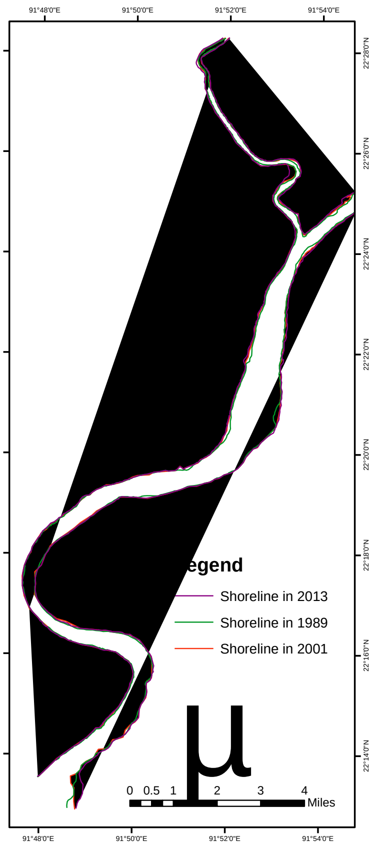

Fig 9: Deviation of shoreline of Karnafuli river

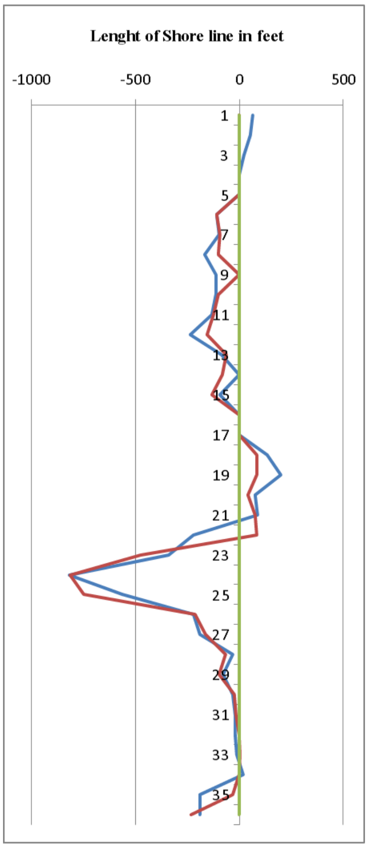

Fig 10: Deviation of left shoreline of Karnafuli river

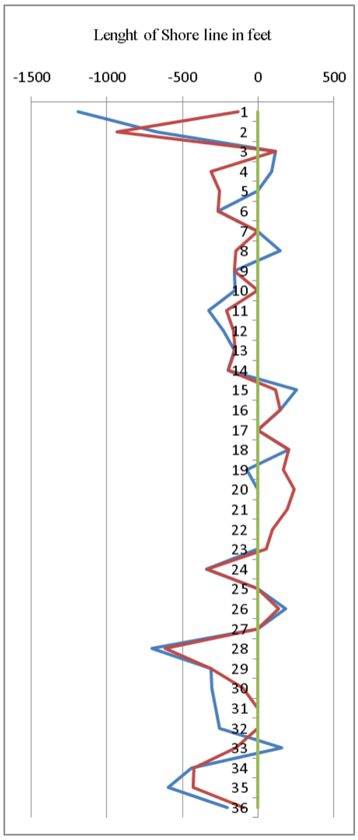

Fig 11: Deviation of right shoreline of Karnafuli river

Negative shoreline switching demark as erosion and positives shoreline switching demark as deposition. About 815 feet lenght shifting up at left bank (fig. 10) and about 1125 feet lenght shifting up at right bank (fig. 11).

CONCLUSION

Landsat satellite images observation predict shifting pattern of Karnafuli River was less demarcated. Landsat images of the last three decades from 1989 to 2001 showed average shoreline deviation rate was 154.059 feet and images from 2001 to 2013 showed average shoreline deviation rate was 108.81 feet only. Satellite imagery shows the coast or sandbank has changed at Boalkhali and Paschim gomdondi, Chadgaon location. The silting of river may not notice significantly due to construction of an obstruction across the Karnafuli River such as Kaptai dam. However, further research may be focused on shore hardening and its impact on urban people.

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BANGLADESH WATER DEVELOPMENT BOARD: A BANK OF HYDROLOGICAL DATA, ESSENTIAL FOR PLANNING AND DESIGN IN WATER SECTOR

Mohammad Mahtab Hossain[1]*, Anwar Zahid[2]

[1]*FFWC, BWDB, Dhaka-1000, Bangladesh.*
[2]*Ground Water Hydrology, 72 Green Road, Dhaka, Bangladesh.*
**Corresponding Author, <mahtabce82@gmail.com >*

ABSTRACT

Data with adequate quality is essential for design and planning in any sector whereas hydrological data collected by Bangladesh Water Development Board (BWDB) is prerequisite for design and planning in water sector. BWDB is mandated by Government of Bangladesh to collect hydrological data over the country which has been performing since 1959. All collected data is processed and stored in Processing and Flood Forecasting Circle (PFFC) at Green Road, Dhaka, under BWDB. Although BWDB has been officially collecting data since 1959 but at few river stations, hydrological data is available since 1919 at PFFC server. This writing up mainly presents an overview of the hydrological data collection network maintained by BWDB. A comparative study will be present here to show whether the resolution of data collection network is sufficient for hydrological modelling or not. This paper will also focus on the utilization of this hydrological data for design purpose in water sector. Finally, it will briefly discuss the drawbacks of BWDB to collect this data with its possible solution or remarks.

Keywords: Hydrological data, Hydrological Network, Hydrological Modelling, Resolution

INTRODUCTION

Water is a national resource. It is also a renewable resource which is renewed each year through the natural process of the Hydrologic Cycle. The large cycle of the hydrosphere includes the evaporation of water from the surfaces of the oceans and the continents into the atmosphere, its back to the land-surface in the form of precipitation, and the surface and subsurface runoff conveying the water back to the water bodies.

Hydrology is a branch of water science describing water transport and storage through the cycles and the interactions between the water and its environment. Considering the task of hydrology, it is proof that not only the natural processes have to be analyzed, but the modification of the water regime due to human intervention must also be considered. Data from both the natural hydrological cycle and the use of water in the social sphere should be collected and evaluated. The description of a water regime includes not only the determination of the quantities transported and stored, but also the qualitative properties of water. Therefore, the hydrological information systems must supply data about the instantaneous condition of and the expected changes in water quantity and quality (FAO,1984)

Water is one of our most essential natural resources. Although there is plenty of water on the earth but water with usable quality is limited by nature. It is not always in the right place, at the right time and in the right quality. This problem is intensifying due to human and chemical waste disposal into water bodies without appropriate treatment. So, adequate planning to secure the water bodies, hydrological information is essential. Hydrology is the science that encompasses the occurrence, distribution, movement and properties of the waters of the earth and their relationship with the environment within each phase of the hydrologic cycle. Hydrologists apply scientific knowledge and mathematical principles to solve water-related problems. Groundwater, extract from beneath the earth's surface, is normally cheaper, more convenient and less vulnerable to pollution than surface water. Thus, it is commonly used for public water supplies. But for balance extraction of ground water, hydrological information is necessary (DEQ, 2014).

The Hydrology of Bangladesh is largely governed by the monsoon originating in the Indian Ocean and the Bay of Bengal, the strategic location of the Himalayas that produces Orographic effect to the monsoon rainfall distribution pattern and the Himalayan glaciers governing the dry season flow, the three major rivers that drain the northern slope of the Himalayas in the China, the Assam valley in the Northeast India and Bhutan and the southern slope of the Himalayas that includes Northern India and Nepal.

After recurrence devastating flood of 1954 and 1955 “Crug Mission” was formed in 1957 under United Nations (UN) to grow up food productivity by reducing flood damage and water resources development & management in this region. As per mission’s recommendations, Bangladesh Water Development Board (BWDB) started its operation in 1959. Thus, hydrological data collection by BWDB under the chief engineer, hydrology has been started since 1959. Before that, PWD collects few hydrological data and therefore, data has been available since 1919 at few stations.

The prime objectives of this write up are:

- To provide a brief overview of hydrological data collection system and network maintained by BWDB.
- A comparison of network resolution with world standard to satisfy the modern requirement.
- Few remarks to improve the system and meet the present demand.

HYDROLOGICAL DATA COLLECTION AND DISTRIBUTION SYSTEM OF BWDB

Data with adequate quality is essential for design and planning in any sector whereas hydrological data collected by Bangladesh Water Development Board (BWDB) is prerequisite for water sector. BWDB is mandated by Government of Bangladesh to collect hydrological data over the country and it has been collecting data since 1959. Hydrology branch under BWDB, leading by a chief engineer is collecting the hydrological data all over Bangladesh. Under the leading of chief engineer hydrology, there are two superintending engineers (surface water and river morphology) and one director (ground water), who are collecting data at field levels. All the surface water related data (e.g. water level, rainfall, discharge, sedimentation etc) is collected under the supervision of superintending engineer of surface water hydrological circle. Morphological data (e.g. river cross section) is collected under the supervision of superintending engineer of river morphology and research circle. Ground water related hydrological data (e.g. ground water level, ground water storage etc) is collected under the supervision of a director (geology) of ground water hydrological circle. Another superintending engineer under chief engineer hydrology; namely “superintending engineer of Processing and Flood Forecasting Circle (PFFC)”, who is compiling and storing all the collected hydrological data at Green Road, Dhaka. Anyone who wants to get hydrological data for research or study purposes, he can collect it from PFFC at Green Road, Dhaka with showing the authenticity for it and paying nominal charge for it. BWDB has officially been collecting data since 1959 but data has been available since 1919 (e.g. Hardinge Bridge station) at PFFC server at few river stations.

STANDARD DENSITY OF HYDROMETRIC NETWORK

To get reliable and accurate data modern equipment should be used for Hydrological Survey and investigation. It is mentioned that, Hydrological survey works mainly dependent on the modern equipment and trained technical man power. In BWDB Hydrology all the Hydrological survey equipment has been procured during 1960-1970. After long 30-40 years survey work some equipment are damaged. A few numbers of equipments are in good condition. The major number of equipments are still using in survey works after repairing. As a result the accuracy of survey works has been decreased to a great extent.

Modern water resources survey activities very much dependent on the high-tech equipment. On the one hand satellite based survey equipment are being used for field measurement and on the other hand computer based data management are being used in office. Hence, to ensure more accurate and real time data procurement of modern Hydrological survey equipments and installation of mobile network are most essential. Modern technology, trained manpower as well as adequate network density are essential to ensure the quality of hydrological data.

Hydrological network is aimed at giving the hydrological information to be used for the following needs:

-Assessment of the regional or national surface water resources and of their trends (climatic and anthropogenic impacts) thus water resources planning for management and utilization.

-Estimation of environmental, economic and social impacts of current or planned management practices on WR and analysis and forecasting of extreme events (warning) : drought, exceptional floods

To achieve the goal from hydrological network, the density of network should meet the minimum requirement. Recommended minimum densities are presented below:

Table 1: Recommended minimum densities of precipitation stations

Physiographic Unit	Minimum densities per station (area in km^2 per station)	
	Non-recording	Recording
Coastal	900	9000
Mountainous	250	2500
Interior plains	575	5750
Hilly/undulating	575	5750
Small islands	25	250
Polar/arid	10000	100000

(WMO, 1994)

Table 2: Minimum density of evaporation stations

Physiographic Unit	Minimum densities per station (area in km^2 per station)
Coastal	50000
Mountainous	50000
Interior plains	50000
Hilly/undulating	50000
Small islands	50000
Polar/arid	100000

(WMO, 1994)

Table 3: Recommended minimum densities of steamflow stations

Physiographic Unit	Minimum densities per station (area in km^2 per station)
Coastal	2750
Mountainous	1000
Interior plains	1875
Hilly/undulating	1875
Small islands	300
Polar/arid	20000

(WMO, 1994)

Table 4: Recommended minimum densities for sediment stations

Physiographic Unit	Minimum densities per station (area in km^2 per station)
Coastal	18300
Mountainous	6700
Interior plains	12500
Hilly/undulating	12500
Small islands	2000
Polar/arid	200000

(WMO, 1994)

Table 5: Recommended minimum densities for water quality stations

Physiographic Unit	Minimum densities per station (area in km^2 per station)
Coastal	55000
Mountainous	20000
Interior plains	37500
Hilly/undulating	47500
Small islands	6000
Polar/arid	200000

(WMO, 1994)

Table 6: Minimum density of water level stations

Type of region	Range of norms for minimum network Area (km^2) per station
Flat regions of temperate, mediterranean and tropical zones	1000-2500
Mountainous regions of temperate, mediterranean and tropical zones	300-1000
Small mountainous islands with very irregular precipitation, very dense stream network	140-300
Arid and polar zones	5000-20000

(WMO, 1981)

The spacing of observation well in a network (density of groundwater observation network) designed for investigation of groundwater depends on (a) the size of the area (b) the hydro-geological complexity of the area (c) the objectives of the network and (d) the financial limitation. In general, one observation well should be installed every 5 to 20 km^2 spacing. For special case (e.g. intensive studies of ground water regime, large engineering projects like reservoir, mining areas, etc, intensive irrigation and drained area), the spacing should be increased to attain the desired level (WMO, 1981).

IMPORTANCE OF HYDROLOGICAL DATA

The Hydrologic Cycle in Bangladesh constituting the arrival and retreat of the monsoon, recharge and depletion of the soil moisture and the ground water storage, the process of storage and drainage of the large inland water bodies of haors, baors and beels, rise and fall of flood levels in the intricate river network and flood plain, the huge sediment flow along with the water discharge, the instability of the river courses and the consequent riverbank migration, the propagation of tide, salinity and cyclonic storm surges in the coastal waters affect the life of the people, their agriculture, trades and development activities of all sectors of the economy of the nation.

Besides this, for the last few decades, Ground Water has been used extensively as the main source of drinking and irrigation. But this water resource is also facing different problems including quality hazards due to presence of arsenic and the problem is compound by high concentrations of iron and chloride with depth, excessive use of chemical fertilizers, sea-water intrusion as well as continuous declining of water level in city areas. Sustainable use and development of this resource needs detail survey and investigation to avoid further quality and quantity degradation. Therefore, hydrological data with adequate quality and quantity is pre-requisite for the water resources management in a sustainable way.

Hydrology information can be useful to land managers, engineers, modelers and the public for a number of purposes which may include:

- Water is an important mechanism for transport of energy, nutrients and chemicals, and sediments. Thus, defining the hydrologic cycle of a watershed is an important step in understanding erosion processes, water nutrient dynamics, contaminant transport, thermal processes (e.g. ground water).
- Water quality and quantity data can allow for balance use of water among the stakeholders.
- Relative roles of groundwater and surface water components in the basin allow natural resource managers to better evaluate effects of proposed surface disturbing activities on fish habitat and water quality.
- Flood Forecasting/Flood Warning
- Flood Studies (Flood Insurance, etc)
- Flood Control Design (water management)
- Water/Wastewater Plant Siting
- Watershed Best Management Practices
- Environmental Impact Assessment/Abatement
- Hydraulic Structure Design
- Drinking Water Monitoring/Other Public Safety Issues
- Lake Management
- Weather and Climate Forecast Modelling
- Climate Diagnostics and Change Detection
- Water Availability Modelling
- Determination of the depth of water table
- Determination of ground water flow direction and speed (for contamination analysis)
- Calibration and validation for ground water modelling *(*Holmes et al., 2001; ABFC, 2002; FAO,1984)

METHODOLOGY

The water regime of the country and its different components are dynamic and are constantly changing from point to point spatially and temporally and as such a continuous monitoring system is indispensable for any country and also for Bangladesh. It is mentioned that from 1960 to 1995 a number of T. A. Projects has been implemented by UNDP and other Donor Organisations for improvement of National Hydrological services in Bangladesh. From the study a Nationwide Hydrological Station Network has been developed. BWDB Hydrology has now an operational network of Hydrological and Hydro-Geological stations covering Surface Water, Ground Water, River Morphology and Flood Forecasting activities all over Bangladesh.

The paper will be based on the following steps:

- Data collection network maintained by BWDB is collected from the office of chief engineer hydrology, PFFC and Surface Water Hydrology Circle.
- Data collection network will be analyzed and compared with the world standard resolution which can be found through literature review.
- Final remarks will be provided with respect to modern hydrological data requirements.

For the network analysis, it is better to separate the hydrometric network into coastal, mountainous, interior plains, hilly area, small islands and polar/arid regions. This study will try to do it with respect to available information. The country (Bangladesh) area has been considered here of 147,570 sq. km., coastal area (in terms of administrative consideration, 19 districts out of 64 are considered as coastal districts) has been used for this study of 47,201 km^2 and Mountain area has been considered (in terms of administrative consideration, 5 districts out of 64 under Chittagong division are considered as mountain districts) of 21,119 km^2 (BBS, 2011; MoEF, 2007).

SCENARIOS OF HYDROLOGICAL DATA MAINTAINED BY BWDB AND ITS ANALYSIS

Table 7: Presentation hydrological data network maintained by BWDB

Type of Data	Station Number	Resolution	Standard Max. Resolution (km^2)
Non-tidal Water Level	235	337	1000-2500
Tidal Water Level (Coastal)	77	613	2750
Water Level (Mountain)	28	754	300-1000
Non-tidal Discharge Station	92	861	1875
Tidal Discharge Station (Coastal)	3	15733	2750
Discharge Station (Mountain)	8	2640	300-1000
Surface Water quality	29	5088	37500
Salinity	100	472	2750
Sedimentation	15	9838	12500
Precipitation	184	431	575
Precipitation (Coastal)	71	665	900
Precipitation (Mountain)	19	1112	250
Climatology	3	49190	-

Evaporation	30	4919	50000
Morphological Cross Section	1852	80	100-2500
Ground Water Table (total)	1923	77	5-20
Ground Water Quality	917	161	5-20
Aquifer Test	297	497	5-20
Bore Hole Lithology	1166	127	5-20

Source: Office of the Chief Engineer, Hydrology, Green Road, Dhaka

RESULTS AND DISCUSSION

The hydrological network maintained by BWDB at plain land is densely located (high resolution) in compare to coastal or mountain area in Bangladesh. The resolution of water level/stage station satisfies the world standard resolution but the resolution of discharge/stream flow station at mountain area in Bangladesh does not meet the world standard resolution requirement. The number of precipitation station at mountain area is also insufficient with respect to WMO standard resolution.

Salinity data collected by BWDB meets the standard resolution but the distribution of stations over the coastal area is not uniform but consolidated at small region.

The resolution of ground water hydrological data is clearly insufficient. Plotting of high resoluted ground water contour map is not possible with the help of this resoluted data.

Presently, the number of water level stations maintained by BWDB is 340 whereas it was previously it was 343. That means, 3 stations are now out of order. Although the demand of high resoluted data is increasing but resolution of water level stations are decreasing. The number of discharge stions previously maintained by BWDB was 112 whereas presently it maintains the number of 103, which indicates that the resolution of discharge station is decreasing.

BWDB previously maintained 269 precipitation stations which is now increased at a number 274 but the number of evaporation pan is decreased from 39 to 30. BWDB is maintaining 3 climatological stations but 1 of these stations is presently not in service.

Table 8: Comparison of present and past hydrological data network

Data Type	Present Number	Previous Number
Water Level Station	340	343
Discharge	103	112
Precipitation	274	269
Morphological Cross Section	1852	1074
Ground Water Table	1903	1250
Aquifer Test	297	278
Borehole Lithology	1166	471

Source: Surface Water Hydrology Circle, Green Road, Dhaka

Overall, the numbers of hydrometric data stations are in decreasing trend whereas the demand of high resoluted qualified data is increasing. Although the number of ground water related hydrological data has been increased but it has still not met the world standard network resolution criteria. Therefore, more ground water data stations should be installed to plot high resoluted contour map and other purposes.

CONCLUSION

BWDB is responsible for Water Resources Development of the country but the basic qualified hydrological data is pre-requisite for the planning, design and operation of most development projects. Hydrological data collection network density must satisfy the standard resolution to ensure the adequacy of data. So, hydrometric data collection network density must be increased to meet the standard resolution which will help to get adequate data applicable for planning and design.

Bangladesh as also the whole World has become very conscious much more than ever before about the climate change and its impact on human civilization. A great part of Bangladesh being contiguous to the coast and a low lying is likely to be seriously affected by the rising sea level due to climate change. This has put another dimension for continuous and accurate monitoring of the coastal environment. For the research of climate change and its impact, accurate and adequate hydrological data with appropriate resolution is essential. Therefore, BWDB should focus on the hydrological data network resolution to ensure the adequacy of data for climate change research.

Bangladesh is a densely populated developing country. The natural resource base of the country is not yet fully developed. Discovery of arsenic pollution in ground water has shattered the Hand Tube Well based rural drinking water supply. Uncontrolled sewage and fast growing industrial waste disposal into the river system is not only affecting the life and environment of the people but also affecting the fish production for the nation. Deteriorating open water fisheries due to pollution are particularly affecting the poor fishermen and common people from the easy accessibility to the source of protein from the open water fisheries. So, authentic hydrological data is essential for adequate planning and design to preserve the ecological balance. Therefore, hydrology branch of BWDB should be well equipped e.g. automated system and data collecting human resources will be well trained to reduce the source of errors for getting the qualified data. Better the data quality, better the design/planning confidence prerequisite for sustainable development.

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FLOOD HAZARD MAPPING OF VARIOUS LAND TYPES IN A RIVERINE FLOOD PRONE AREA FOR DIFFERENT FLOOD MAGNITUDES

Sadequr Rahman Bhuiyan[1*] & G.M. Tarekul Islam [1]

[1] *Institute of Water and Flood Management, Bangladesh University of Engineering and Technology, Dhaka, 1000, Bangladesh.*
**Corresponding Author <ceesadeq@gmail.com>*

ABSTRACT

Khoksabari Union, under the Sirajganj Sadar Upazila is surrounded by a network of rivers namely the Jamuna, the Banglai and the Karatoa. Water level and discharge in all these rivers rise in the monsoon (June-September) due to seasonal rainfall and trans-boundary flow which eventually makes the area vulnerable to flooding. The study area is very much vulnerable to flooding because of having comparatively low lying areas and deltas and moreover breaching on Jamuna right embankment (JRE). In the present study, flood hazard mapping for different land types have been carried out for different flood magnitudes integrating Digital Elevation Model (DEM) and interpolation of water level height. To determine the water level of 2.33, 5, 10, 20, 50 and 100-year return periods flood frequency analysis are conducted based on observed water level data of Bangladesh Water Development Board. Landuse map has been generated from the LANDSAT satellite images by supervised classification algorithm. The supervised classification process is divided into two phases: a training phase, where the computer is ’trained’, by assigning for a limited number of pixels to what classes they belong to in this particular image, followed by the decision making phase, where the classification algorithm assigns a class label to all (other) image pixels, by looking for each pixel to which of the trained classes this pixel is most similar. To have training areas, ground truthing points have been collected for each landuse class from the field survey. In the present study, LANDSAT satellite image has been used for land use mapping with the help of Integrated Land and Water Information System (ILWIS 3.4) software. Agricultural land, rural settlement, water bodies, growth center, roads and bare soil coverage have been identified in the landuse map. The study shows that, high land (F0) which is 54.75% of the total study area is not much inundated in under normal monsoon flood and significant amount of inundation of F1,F2,F3 and F4 lands from 5-year to 100-year return period. This flood hazard map can be used for selecting the types of crops and area for cultivation during the monsoon period on the basis of magnitudes of inundation of different land types.

Keywords: Flood Hazard, Riverine, Flood Magnitude, Digital Elevation Model, Landuse

INTRODUCTION

Bangladesh is extremely vulnerable to flooding because of its geographical setting. It is a low-laying deltaic flat country with big inland water bodies, including some of the biggest rivers in the world. Flooding is an annual recurring event during monsoon and 80% of the annual rainfall occurs in monsoon. Due to intense rainfall during monsoon (June to September), about one-fifth to one-third of the country is annually flooded by overflowing rivers caused by heavy rainfall. Bangladesh is a flood prone country and very often experiences devastating flood during monsoon that causes damage to crops, settlement, fisheries, infrastructures and properties. This study assessed the flood hazard of various land types for different flood magnitude by integrating LANDSAT and SRTM digital elevation data with GIS and RS.

STUDY AREA

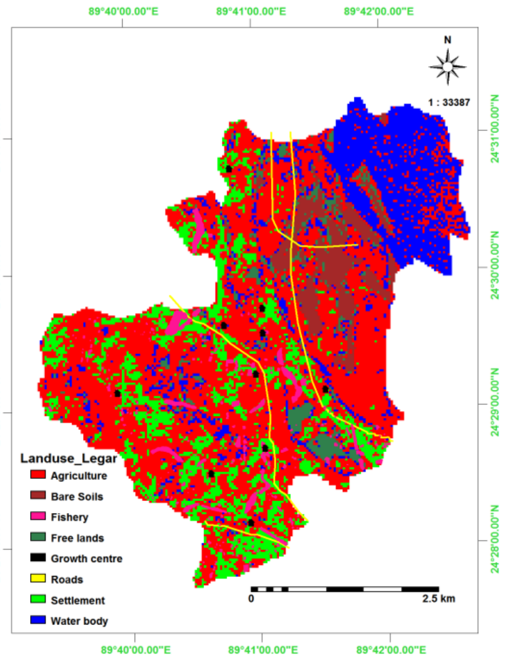
Khoksabari union of Sirajganj Sadar thana under Sirajganj district in Bangladesh has been selected as study area. It is located in the northern part of the country. Three major rivers bound Sirajganj sadar, Jamuna in the eastern side, the Banglai and the Karatoa in the western side of the study area. The main cause of flooding in the area is the trans-boundary inflow from upstream catchment carried by the Jamuna River. The study area is very much vulnerable to flooding because of having comparatively low lying areas and deltas and moreover breaching on Jamuna right embankment (JRE). In the Jamuna River, average recorded water level at Bahadurabad ghat, which is an upstream location of the study area is, 15.99 m (PWD). The highest water level recorded at this point is 20.61 m (PWD) on 30, August, 1988 (BWDB, 2009). Khoksabari union is situated between $24^0 27^/$ and $24^0 31^/$ north latitudes and between $89^0 39^/$ and $89^0 43^/$ east longitude. The area of Khoksabari union is 19.58 km^2. The population and homestead area are 32,455 and 1.87 km^2, respectively (BBS, 2001).

METHODS AND DATA

For better assessment of flood hazard, especially in low land areas like Bangladesh, DEM data, historical flood depth and water level data, inundation of flood water, landuse and other data of the study area are essential. For this study two types of data have been used. Topographic data which includes Digital Elevation Model (DEM), satellite image and hydrologic data is water level data. Satellite Images with the integration of Geographical Information System (GIS) are used for historic flood hazard analysis. National Oceanographic and Atmospheric Administration (NOAA) and Advanced Very High Resolution Radiometer (AVHRR) data were used to analysis Bangladesh's historical flood event of 1988, which sets a hundred - year record for the inundated areas, with severe damage occurring throughout this region (Islam & Sado, 2000). Water level data of the year 1983 to 2012 were collected from three gage stations SW49, SW66, SW11 of Bangladesh Water Development Board (BWDB) of Brahmaputra-Jamuna , Karatoya and Bangali River for flood frequency analysis. For flood frequency analysis yearly maximum levels of water were used. Digital Elevation Model (DEM) based flood extent with depth; an integral part of GIS can be adopted for flood hazard study. To get flood map of a study area, flood elevation generated from water level data, is subtracted from ground elevation data (Dewan, et al., 2004). For obtaining flood extent it is necessary to have both interpolated water level and land elevation surfaces as flooding is a continuous phenomenon and interpolation is the procedure of estimating the value of properties at unsampled points or areas using a limited number of sampled observations.In order to resolve the methodological gap, interpolation technique at GIS system has been applied using water level data of different stations in order to generate interpolated water level surface. Moving average interpolation techniques has been applied to water level surface generation using ILWIS 3.4 software.

For FFA, commonly used empirical distributions 2- Parameter Log normal (LN2), 3- Parameter Log normal (LN3) Pearson type 3 (P3), Log Pearson type 3 (LP3), Extreme value type 1 (EV1) have been adopted in this study. After that, using Probability Plot Correlation Coefficient (PPCC) and goodness – of – fit test, the best frequency analysis method has been selected for flood mapping in this study. In this study, observed peak water level data collected from Bangladesh Water Development Board (BWDB) have been used for FFA and estimated water level data from the FFA have been used as height source in moving average interpolation technique for water level surface generation for different flood magnitudes. Land use or land cover dataset was generated from the digital image classification of LANDSAT, ETM+ satellite images, downloaded from Global Land Cover Facility web site (Alaguraja et al. 2010). LandSat TM sensor provides several improvements over the MSS (Multispectral Scanner) including higher spatial resolution and radiometric resolution, finer spectral bonds with seven (as opposed to four in MSS) spectral bands. Supervised classification of LANDSAT images was done with of Integrated Land and Water Information System - ILWIS 3.4 Academic software to derive different land coverage in the existing study areas.

Samanta, et al., 2011, developed land use or land cover dataset from the digital image classification of LANDSAT satellite images. In this study, landuse map has been generated from the LANDSAT satellite images by supervised classification algorithm. The supervised classification process is divided into two phases: a training phase, where the computer is 'trained', by assigning for a limited number of pixels to what classes they belong to in this particular image, followed by the decision making phase, where the classification algorithm assigns a class label to all (other) image pixels, by looking for each pixel to which of the trained classes this pixel is most similar. To have training areas, ground truthing points have been collected for each landuse class from the field survey. In the present study, LANDSAT satellite image has been used for land use mapping with the help of Integrated Land and Water Information System (ILWIS 3.4) software. A supervised classification was performed on True color (3, 2, 1) composite image, false color composition of band 4, 3 and 2 and NDVI image into following land use and land cover classes: cultivated land, rural settlement, water bodies and others. Others include free land, bare soil, etc. which are not important elements for finding vulnerability. Roads of the study area were identified from Local Government Engineering Department (LGED) roads map digitiging using ILWIS editing tools. Growth centres location information were collected during field survey using Global Positioning System(GPS). Information of the ground truthing points were also used to assess the accuracy of classification.



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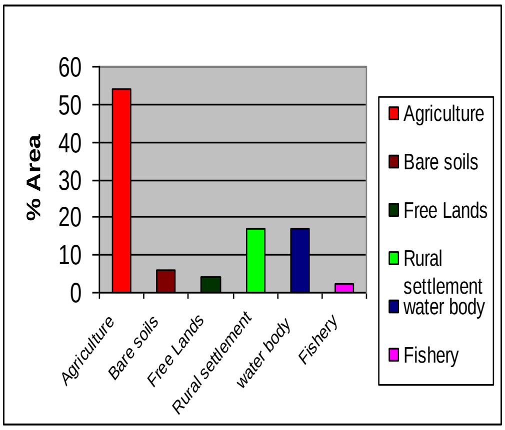

Figure 1 Existing land use pattern of the study area.

CLASSIFICATION OF LAND BASED ON ELEVATION

The land classification based on elevation or flood depth can be useful for flood management. The change of inundation area in terms of large depth to shallow depths are analyzed using F0, F1, F2, F3 and F4 types of land classification corresponding to depth of inundations, of 0-30 cm, 30-90 cm, 90-180 cm, 180-300cm and greater than 300 cm. F0, F1, F2, F3 and F4 lands classification of Bangladesh regarding flood depth are shown in Table 1.

Table 1 Classification of cultivated land by flood depth (*Source:* MPO, 1986)

Land Types	Flood depth (cm)	Description of land	Area (Mha)	%
F0	0 - 30	High land	4.20	29
F1	30 - 90	Medium high land	5.04	35
F2	90 - 180	Medium low land	1.18	12
F3	180 - 300	Low land	1.10	8
F4	over 300	Very low land	0.19	1

This classification is used in Bangladesh water resources planning. Based on this classification, present study area lands have been classified in Table 2. This table represents the land classification for the whole Union. Khoksabari union is the worst flood affected area compared to others ten unions of Sirajganj Sadar Upazila.

Table 2 Land classification is regarding flood inundation of the study area.

Land Types	Types of Land	Flood depth (cm)	Area (acres)	Area in percent
F0	High Land	0 - 30	2621.37	54.75
F1	Medium High Land	30 - 90	548.43	11.45
F2	Medium Low Land	90 - 180	568.44	11.87
F3	Low	180 - 300	276.88	5.78
F4	Very Low Land	over 300	773.04	16.14
	Total		4788.16	

The classification has been standardized from Bangladeshi farmers' own classification of land types in relation to 'normal' seasonal flooding. F0 lands are sustainable for HYV rice in wet season. F1 lands are suitable for Aus and T. Aman. B.Aman can be grown in F2 and F3 lands in wet season. But depth, duration and time of the flooding do not permit growing of B.Aman in F4 lands.

INUNDATION MAPPING

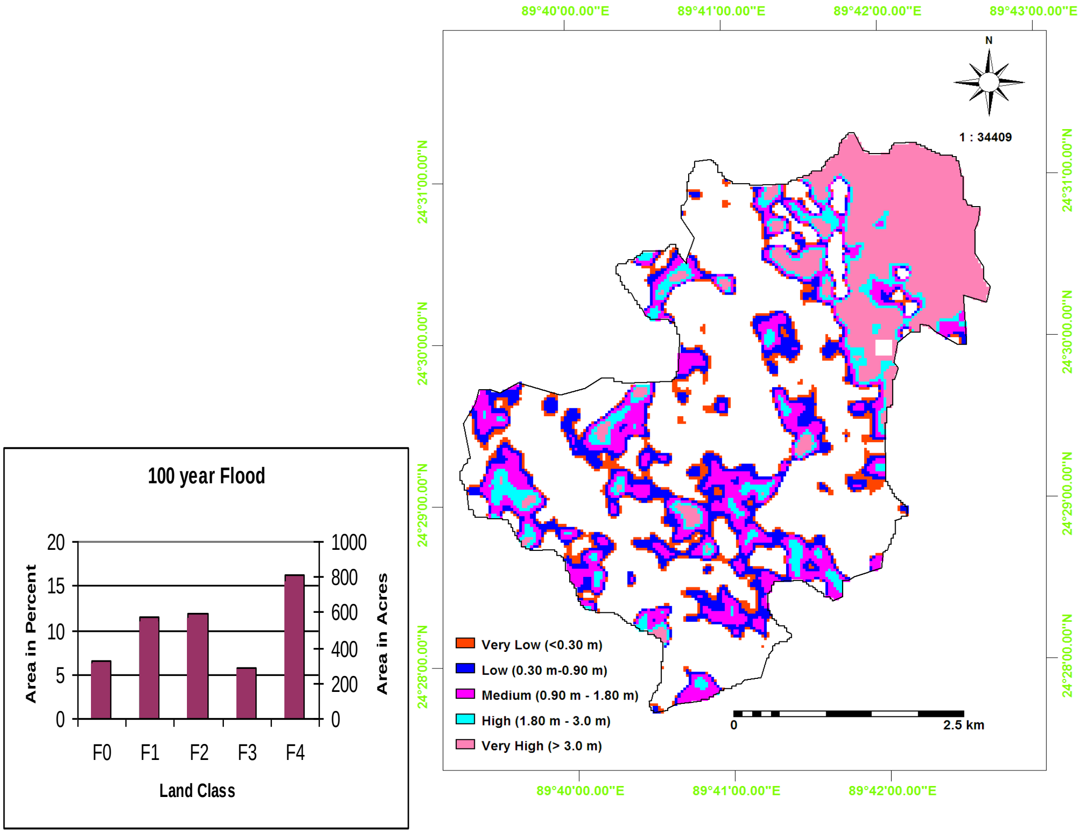

Figure 2 100-year return period flood inundation map and inundation statistics of study area

The flood water levels obtained from Flood Frequency Analysis (FFA) for various return periods have been used for moving average interpolation technique and overlain onto the land surface elevation of the study area. Different between water level interpolation and land elevation surfaces has been considered as a depth of inundation for each return period. In the study, inundated areas are defined into five qualitative hazard classes viz. Very Low Hazard(< 0.3m), Low Hazard (0.3m – 0.9 m),Medium Hazard (0.9m – 1.8 m), High Hazard (1.8m –3.0 m) and Very High Hazard (> 3.0 m) based on the inundation depth . Inundation area of different class increases with the increase of flood magnitudes. On the other hand, flood free zone decrease with the increase of return period. It is also noticeably that, for all land types, affected area increase with the increase of flood magnitude and flood depth. Very Low (F4) lands are more affected than other lands types. The study shows that, high land (F0) which is 54.75% of the total study area is not much inundated in under normal monsoon flood and significant amount of inundation of F1,F2,F3 and F4 lands from 5-year to 100-year return period. In 100-yr return period 51.71 percent land become inundated, in which F0, F1, F2, F3 and F4 inundation area are 6.47, 11.45, 11.87, 5.78 and 16.14, respectively. Study conducted by (Institute of Water Modeling, 2010) termed that 1998 flood is a return period of 75 to 100 year. The percentage area of flooded in Sirajganj sadar upazila was 54 as on September, 17,1998 (BWDB,2010). In this study, for 100 year return period, 51.71 percent flooded area has been found, which is quite close to BWDB study . Flood inundation map and the inundation statistics of F0, F1, F2, F3 and F4 land class of 100 year return period flood are depict in Figure 2.

CONCLUSION

Flood hazard mapping of different land types in a low laying area based on landuse map in conjunction with digital elevation model (DEM) analysis in the GIS atmosphere can be effectively accomplished. Significant amount of medium high land, medium low land and very low land have been affected with the increase of flood magnitude compare to other land types.

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SHELTER FRIENDLY EMBANKMENT TO SAVE FLOOD AFFECTED PEOPLE FOR LONG TIME FLOOD MANAGEMENT IN BANGLADESH

I. Hasan[1]*, S. Hossain[2], N. Arju[3] & M. S. Islam[4]

[1]*Disaster and Human Security Management, Bangladesh University of Professionals, Dhaka, Bangladesh*
[2] *Institute of Water Modelling, New DOHS, Mohakhali, Dhaka-1206, Bangladesh.*
[3]*Disaster Manangement in IDMVS, Dhaka University, Dhaka, Bangladesh*
4 Disaster Management, Dept. of Geography and Environment, Dhaka University, Bangladesh.
**Corresponding Author, <imrul143@yahoo.com >*

ABSTRACT

Construction of earthen embankments for flood control, irrigation and drainage has been the history of Bangladesh but this can't solve the flood problem, effectively and permanently because of their ease of vulnerability with rain splash and flow of flood water. Construction of earthen flood control embankments is an established practice in Bangladesh for protecting people's lives and homes, agriculture and infrastructures. The paper reports the options for multipurpose use of embankments in Bangladesh based on the field visits to embankments site, collected data and information on failure and on-service embankments during field visits, necessary data related to embankments construction practice obtained from available publications and newspaper information reported in the year 2007. It studies the basic features and characteristics of floods and flood hazards, and reviews the design & construction practices followed. Several cases of successful and unsuccessful river and flood control embankments are investigated and analyzed. Based on the results of analyses and discussion, it shows that the present method of embankments in Bangladesh, although requires huge amount of money for its construction and repair every year, fails to solve the flood problem effectively and permanently rather it brings many other new problems. It not only increases the siltation on the floodplains and river beds but also creates a risky situation for the inhabitants inside the boundary of embankments. On the basis of overall present situation in Bangladesh, the paper also suggests a proper design and construction method of embankments to control and minimize the extent of flood hazards in the long run and also concluded that to achieve consistent standards of management of flood embankments, and also to raise these standards to optimize their performance, require (a) better understanding and application of good practice, and (b) a range of research actions.

Keywords: Bangladesh, Flood, Flood Management, Shelter, Embankment

INTRODUCTION

A total of 5,695 km of embankments, including 3,433 km in the coastal areas, 1695 flood control/regulating structures, and 4,310 km of drainage canals have been constructed by the Bangladesh Water Development Board (Rahman and Chowdhury, 1998). Embankments and polders have reduced floodplain storage capacity during floods, leading to an increase in water levels and discharges in many rivers (Chowdhury, 1998). Embankments can also create a false sense of security among residents living within embanked areas. For example, breaching of the Gumti embankment at Etbarpur during the 1999 flood caused substantial damage to the environment and property. Earthen embankments can easily breach and can be damaged by riverbank erosion. Most of the embankments in Bangladesh have experienced breaching and erosion more than once since their completion. The study is an attempt to provide a comprehensive insight into the shelter friendly embankment to save flood affected people for long time flood management. The study is based on two research procedure: a literature review and an empirical study. In answering the research questions and seeking to achieve the objectives of this study, a qualitative empirical research design was followed by utilizing data from primary and secondary sources. This research procedure aimed to answer the questions raised via exploratory and descriptive research procedure (Shahjahan, M.). This research design made possible an in depth understanding of the causes of erosion of embankment and measures which can be reduced the failure or erosion of embankment to save flood affected people. It studies the basic features and characteristics of floods and flood hazards, and reviews the design & construction practices followed. The paper reports the options for multipurpose use of embankments in Bangladesh based on the field visits to embankments site, collected data and information on failure and on-service embankments during field visits, essential data related to embankments construction practice obtained from available publications and newspaper information.

LITERATURE REVIEW

Bangladesh is one of developing and heavily populated countries of the world. Most of the people directly or indirectly relay on agriculture for earning their livings. The annual growth rate of GDP is mainly dependent on agricultural production. For enhancing agricultural production, construction of the embankments and dykes, their repairing and rebuilding for irrigation, flood control and drainage have been the history of Bangladesh long since. Institutional steps for constructing embankments began with the formation of the East Pakistan Water and Power Development Authority (EPWAPDA) in 1959. After the liberation of Bangladesh, EPWAPDA was split in 1972 into two organizations, namely the Bangladesh Water Development Board (BWDB) and the Bangladesh Power Development Board (BPDB). Embankment, dykes and other similar structure fall under BWDB to save life and properties from natural disasters (e.g. flood, cyclone etc.) and to increase agricultural production. Over the last few decades, nearly 13,000 km of embankments have been constructed (The Bangladesh Observer) and BWDB.

- Over 4,000 km of coastal embankments along the coastline nearby the Bay of Bengal and offshore islands.
- Nearly 4,600 km of embankments along the bank of big rivers flowing across the country.
- Nearly 4,500 km of low-lying embankments along the small rivers, haors and canals

To develop an effective system of irrigation, flood control and drainage, a few thousand allied structures also form the vital part of embankments. 1,488 regulators/sluices, 108 bridges and 923 other structures have been constructed in 135 polders over 472 km of embankment to defend 1.09 million ha of land (CERP, JPC).River embankments protect lives and property from flood and overflow during the monsoon. Sea embankments of the offshore islands and coastal zones give safeguard against the intrusion of saline water and damage connected with frequent attacks of tidal surges and cyclonic storms. (Rashid. H.)

Causes of Erosion and Their Effects

Most embankments face light to moderate erosion troubles arising out of rainfall splash, animal actions and the nature of human uses and poor maintenance system. Some of the critically positioned submerged types of embankment in *haor* areas of the eastern part and river embankments of the main land are subjected to turbulent water currents and changes in river courses. The difficulty is acute in offshore islands and coastal belts where the embankments are in addition exposed to erosion by sea waves and tidal fluctuation of water levels. The estimate prepared by BWDB in 1984 shows that about 1,200 km of bank length of rivers were subjected to erosion, 565 km of which faced severe erosion problems. The instability in river regime coupled with huge discharge and sediment load cause erosion, scouring and also deposition, and thus a chain action proceeds. This is almost a habitual phenomenon.

According to the latest information available from BWDB, it is found that 441 projects/sub-projects are either fully or partially damaged due to the severe floods of 1998. The total estimated cost of the rehabilitation works listed below is about US$143.17 million as follows; (I) Embankment – 2,987 km (II) Irrigation/Drainage canal – 373 km (III) Water control structures – 1,031 (IV) Protection works – 187.

Natural Forces

The natural forces cause erosion of the embankments in the following way:

Rainfall impact (from both the regular monsoon rains and torrential rains): Mean annual rainfallvaries from about 1,500 mm in the northwest (Khulna district) to over 3,750 mm in the south (Cox's Bazar). The heaviest rainfall occurs in July and ranges from 350 mm to over 875 mm accordingly (Source: Bangladesh Meteorological Department). The main features of rainfall impact are:

- ✓ The embankment crest is mainly affected with the formation of *ghoghs* and initiation of piping action leading to collapses in combination with either.
- ✓ Surface runoff caused by rainfall results in sheet erosion.
- ✓ Flooding (monsoon/periodic floods and those created by storms/cyclones).
- ✓ The high head of water on the river side induces piping across the embankment, which may lead to breaching and collapse of the polder system.
- ✓ Monsoon flooding often gives rise to serious erosion of embankments by undermining due to current, vortex and wave forces; the entire embankment gets affected, beginning with the damage of shoulders and crest due to undermining, and gradually the overtopping causes a complete wash down.

Wave action (daily/periodic and created by constant wind)

- ✓ Tidal waves cause damage to the embankments located too near to the sea (the earthen embankments in the coastal zones should have adequate setback not allowing its exposure to wave actions).
- ✓ Cyclonic storms in the coastal zone (occurring repeatedly) act upon the water surface, causing it to advance towards the shore with enormous hydraulic loads.

Turbulent water currents (mainly in rivers and at coastlines)

- ✓ The high velocity flow of water associated with vortex motion in rivers and estuaries often causes erosion of the banks.
- ✓ At the mouth of a branch river or canal, especially in the surroundings of sluice gates, the turbulent water current erodes the banks and subsequently the embankments.
- ✓ The presence of continuous borrow-pits on a river or seaside induces undercutting of the embankment toes and slopes due to complete inundation of the riverbank or seashore during the monsoon.

Wind action: The slow and steady action of wind in the relatively sparse fields and coastlines blowsaway the topsoil of the embankments where it is sandy or a mixture of silt and sand. The embankment crest and bare surfaces are gradually eroded to leave patch holes and undulated surfaces for further decaying by rainfall splash, runoff or wave action. (FAP6)

Human Interference

The most commonly observed erosion problems out of the varied human uses are noted below:

Travel paths for men and cattle: The people and their cattle, while moving along the damaged crest, often tend to take a better alternative route along the shoulders, slopes and even toes. Gradually the shoulders and slopes are also affected. Villagers frequently have to cross over the embankment to have direct access to the river or the sea to meet their various daily needs.

Homesteads and agricultural practices: Embankments often become the privileged sites for theconstruction of villages and isolated homesteads. This is mainly due to the fact that the embankments give the people a sentiment of security from flooding. The unmanaged cultivation of these species destabilizes the embankments. Agricultural practices on embankments are encouraged by: (FAP6)

- A high demographic pressure on the available land and accordingly a shortage of land for the rural population;
- Minimal land acquisition by the government brought about by high appropriation costs; as such, there is often no or insufficient provision to resettle people displaced by the construction of embankments; and
- A poor performance of routine maintenance activities for the embankments.

Cattle grazing: Cattle, mainly belonging to people living on the embankment, cause erosion byuncontrolled browsing of natural grasses. When the embankment is overgrazed, plant species and the vegetative cover, especially the grasses, show retarded growth, weaken and cannot continue to ensure adequate protection of the embankment.


Fig 01: Shelter on the embankment when the area is flooded and public cuts of embankment.

Public cuts: Public cuts and tubes linking a river or seaside with the country side of its embankment are frequently observed. These cuts weaken the embankments, exposing them to slow but continual erosive forces. During flood or cyclonic storm, breaching or major erosion occurs at those points. The people mainly cut the embankments to fulfil their purposes:

- To get rid of the poor and inadequate drainage conditions of the existing structures, they arrange quick removal of excess floodwater from the polder area to the river or the sea.
- They create temporary irrigation inlets for applying sweet river water to the cropping fields when there are prolonged droughts in the polder area.
- For short-term economic purposes yielding individual-level benefits, sometimes people allow river or seawater to penetrate inside the polder for shrimp cultivation or any other fishing requirement or salt panning. (FAP6)

Unplanned afforestation of embankment slopes: Afforestation without appropriate planning and management techniques destroys the undergrowth grass cover and becomes ineffective for erosion protection.

Uncontrolled animal activity: Other animals than grazing cattle also cause erosion of embankments. Burrowing animals such as rats often seek shelter on the embankment during floods. Rat burrows and holes, cavities, tunnels, etc, dug by other animals like earthworms often cause substantial weakening of the embankment.

Improper design and construction technique: In many cases the embankments are designed within sufficient setback, resulting in increased exposure to waves and current action. This may be due to the high costs involved in land acquisition.

METHODOLOGY

Measures Taken for Erosion Control

Efforts have been made at times to protect the embankments from erosion. In fact the embankments are regarded as very dynamic living microcosms. The multi-facetted problems facing the embankments in Bangladesh are fascinating and challenging. Light to moderate erosion of the reaches of embankments is common but only those with severe erosion (like turbulent water current, wave action, fluctuating water levels, etc) tending to near failure and destabilization are deemed worthy of protection. A 1984 BWDB report listed 305 such places (565 km) along riverbanks and 85 towns or villages experiencing severe erosion problems (Table 1).

Table 1: Main areas of erosion in Bangladesh

River	No. of location of bank/embankment erosion	Length of erosion (km)
Brahmaputra-Jamuna	41	162.60
Ganges-Padma	26	94.50
Meghna	8	72.00
Teesta	11	34.90
Minor river	112	92.30
Flashy river	75	23.00
Tidal river	32	85.80
Total	**305**	**565.10**

Source: Alam S.M. Zakiul & Faruque H.S. Mozadded, "Bank protection methods used in Bangladesh

Number of cities affected = 85

The project embankments of the Pabna Irrigation and Rural Development Project, the Meghna-Dhonagoda Irrigation Project, the Bhola Irrigation Project, the Coastal Embankment Project, the Chandpur Irrigation Project and the Brahmaputra Right Embankment executed by the Bangladesh Water Development Board (BWDB) are under erosion at different places **(Source: M. Zaman).**The protective measures of bank/embankment erosion at different locations fall under any of the following methods:

- Mattressing along the bank line and revetments either by boulders or by concrete/brick boulders.
- Permeable spurs Groynes and Guide bundle.
- Artificial loop cut and Porcupines.
- Retards (palisade fences), Gravel drains and Bandals.
- Biological treatment viz. afforestation in the foreshore and embankment
- Establish shelter friendly embankment and ensure the proper maintenance of embankment with awareness of local people. (Islam, A.)

In the bank/embankment protection works geo-textiles are often used to protect the subsoil from being washed away by hydraulic loads such as wave and turbulent currents. The geo-textile replaces the granular filter. Geo-textiles have also been used for bank protection works in Bangladesh. Severe erosion of riverbanks adjacent to the Meghna bridge on the Dhaka-Chittagong highway was corrected by using needle-punched, continuous filament, non-woven, mechanically bonded geo-textile placed on the bank slope both above and below the water level and by placing stones as revetment on the geo-textile below the lowest water level. (Alauddin, M. and Hasan. S.)

Social and Institutional Aspects

The erosion protection measures have never been successful without considering the real problems of the people who are directly or indirectly causing the embankments to erode. During catastrophic floods, the poor people who take shelter on the embankment along with the affected mass of villagers often tend to settle there permanently even after flood water has receded. The social and institutional context of the land-use approach deserves careful consideration to strengthen the broad-based sea-defence management of coastal embankments. (Khalequzzaman, Md.)

CONCLUSIONS

Formulating solutions to flooding problems requires a comprehensive understanding of the geologic settings of the region, and a better knowledge of hydrodynamic processes that are active in watersheds. Only solutions that take into account the underlying long-term factors contributing to flooding problems can prevail. Such contributing factors are as follows: unplanned urbanization, soil erosion, local relative sea-level rise, inadequate sediment accumulation, subsidence and compaction of sediments, riverbed aggradation, and deforestation.Structural solutions, such as the building of embankments along the rivers and polders in coastal regions in Bangladesh, will not solve the flooding problems, but will result in many adverse environmental, hydrologic, economic, ecological, and geologic consequences. Solutions to flooding problems can be achieved by adopting and exercising watershed-scale best management practices that include: floodplain zoning, planned urbanization, restoration of abundant channels and lakes, dredging rivers and streams, increased elevation of roads and village platforms, efficient storm sewer systems, establishing buffer zones along rivers, conservation tillage, controlled runoff at construction sites, good governance, indigenous adjustment of life-style and crop patterns, and improvement on flood warning/preparedness systems.

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FLOW MEASUREMENT OF HYDRAULIC OPEN CHANNEL WITH LASER-LDR FLOW METER

Liton Chandra[1*] & Md.Shamsul Alam[2]

[1]Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.
[2]Department of Electrical and Electronic Engineering, Chittagong University of Engineering and Technology, Chittagong-4349, Bangladesh.
**Corresponding Author, <litondas91@yahoo.com>*

Flow of an open channel may be laminar or turbulence flow. When one –dimensional flow occurs in a channel, flow is affected by various parameters of channel. It is difficult to measure the actual velocity based on flow patterns. Actual velocity as well as discharge can be determined by the present existing velocity through channel with a fixed existing area .When a light-weight ball immerses in flowing channel it will travel a distance both vertically and laterally before it settle down due to action of velocity .This action occurs due to the resultant velocity of the velocity of flow and settling velocity of ball. This settling action of ball is influenced by the force of gravity and buoyant force which is equivalent with drag force giving settling velocity. Appling this technique with determining the moving distance and serviceable time with the assistance of LASER-LDR pair between two points, velocity of flow can be measured. This paper depicts design and performance assessment of velocity measurement of flow with digital LASER-LDR flow meter by which actual existing velocity can be measured of an open channel at any depth. In this instrument, tow LASER and dependent resistor pairs are used to make such sensors in a common stand to detect the time of floating ball for a specified distance while microcontroller, liquid crystal display etc , used for executing program and displaying fluid velocity and flow with fixed area.

Keywords: Light-weight ball, Drag force, Buoyant force, LASER-LDR pair, Liquid crystal display, Microcontroller.

1. INTRODUCTION

Determination of exact flow of fluid like water, oil, diesel etc. in an open and closed channel is an important phenomenon in our day to day life. Knowing fluid's velocity having with fixed area, we can design any fluid related control system easily. Among various fluid flows, open channel flow or closed channel flow measurement is more challenging. Like many other applications of fluid flow measurements, water velocity at different depth in a river need to be determined for appropriate strengthen protection (by giving with light weight ball) at its' banks and sides to diminish hydraulic forces. The aforementioned flow measuring device meets these challenges very smoothly. This measuring device follows exact velocity of fluid with a fixed area. There are different types of flow measuring hydraulic devices such as Venturimeter, Orifice Meter, Pitot tube etc. which are based on Bernoulli's flow equation [1]. As the fluid flow is not uniform, it varies depth to depth; these flow measuring instruments don't give us exact velocity for any specific depth rather than mean resultant velocity. So, to determine velocity of any certain distance and at any depth of a channel, we design this one dimensional digital fluid flow meter. Moreover, a set of formulae need to be memorized to calculate fluid velocity through these types of hydraulic devices. But, in case of this digital fluid flow meter, there is nothing to be memorized as the necessary formulae are used in the internal software which automatically calculate and shows in the Liquid Crystal Display (LCD) unit. After calculating mean velocity using this new device we compare with the traditional ways where all about % errors are founds for water flow in an open channel with the mean velocity. In this device, two sensor made of LASER-LDR pair are placed at a specific distance in frame for laboratory purpose. When a very low weight floating ball passes two sensors consecutively, the microcontroller measures required time and execute other arrangement as per command. Using this measuring instrument, a series of laboratory hydraulic experimental study on different riverbank protection materials launched in Hydraulic Laboratory, Dept. of Civil Engineering of Chittagong University of Engineering & Technology (CUET) and hence, the device calibrated with the measured data. Mathematical Model for fluid velocity, Mathematical Model for falling ball's velocity, Device Overview, Mechanical and Electrical Design, Accuracy, Uses in Physical Life, Scope of Study are given in the following. By all the experiment the devices is found more efficient for certain cases. Moreover, the design simplicity and cost effectiveness make it more effective to use in the concerned arena.

2. MAHEMATICAL MODEL FOR FLUID VELOCITY

The motion of a fluid like that of solid is described quantitatively in terms of the characteristics known as velocity. However, in case of solids it is generally sufficient to measure the velocity of the body as whole, but in case of fluids, the motion of fluid may be quite different at different points of observation. Therefore the velocity V_f at any point of fluid mass is expressed as the relation between the displacement of fluid element along its path and the corresponding increment of time as the later approaches zero. A particular point, P (distance OP= r from the origin) in the space occupied by a fluid in motion is selected which may be denoted by coordinates *(x, y, z)* shown in Fig. 1. Since this point is fixed in space, the coordinates x, y, z and the time, *t*, are independent variables. At this point if *ds* is distance travelled by a fluid particle in time *dt* then the velocity V_f of the fluid particle at this point may be expressed as.

$$V_f = \lim_{dt \to 0} \frac{ds}{dt}$$

The velocity is a vector quantity and hence it has magnitude as well as direction. Therefore, the velocity V_f at any point in the fluid can be resolved into three components u, v and w along mutually perpendicular directions *x, y and z* respectably[Fig:1]**.** Each of these components can also be expressed as the limiting rate of displacement ds in the corresponding direction.

Thus if *dx*, *dy* and *dz* are the component of the displacement ds in x, y and z directions respectably, [2] then

$$u = \lim_{dt \to 0} \frac{dx}{dt}$$

$$v = \lim_{dt \to 0} \frac{dy}{dt}$$

$$w = \lim_{dt \to 0} \frac{dz}{dt}$$

$$V_f = iu + jv + kw$$

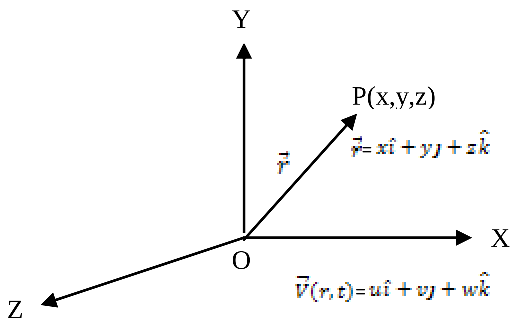


Fig1: Co-ordinates for a fluid particle.

Where *i*, *j* and *k* are the unit vectors parallel to the x, y z axis respectively. But, this velocity measuring device is implemented assuming steady as well as uniform flow up to 1 inch depth and so on till bottom point and hence, the quantity *v* and *w* is equal to zero. So, the velocity of the fluid is

$$V_f = u = \lim_{dt \to 0} \frac{dx}{dt} \text{------------}(1)$$

This 1 layer one dimensional velocity mean the exact velocity of fluids which can be determinate with the help of microcontroller and with the action of light weight falling ball.

3.M ATHEMATICAL MODEL FOR IMMERSED LITHT WEIGTH BALL.

If the fluid is not more concentrated with other particles which will not be close enough for velocity field interference to occur are termed as dilute suspensions. When a light weight particle or ball get immersed through flowing fluid, if the particle and ball does not change its size, shape and specific gravity during flowing its call discrete particle.[3]

A table tennis ball as a discrete particle immerse in flowing diluted fluid , then three types of forces are acted on this ball.

(1) The force of gravity: Due to weight of ball this force act downward by such parameters

$$f_g = \rho_p g V_p \text{..............................}(2)$$

(2) Buoyant force : Due to action of buoyancy this force act upward with such parameters

$$f_b = \rho_f g V_p \text{.....................}(3)$$

For this two force net force is, $f_{net} = f_g - f_b = (\rho_p - \rho_f)\, g V_p$ ……………..(4)

This net force becomes the driving force for acceleration.

(3) Drag Force : This force is created due to viscous friction with such parameters

$$f_d = C_d A_p \rho_f \frac{v^2}{2} \text{..........................}(5)$$

Where C_d is the co-efficient of drag force, A_p is cross-sectional area, , perpendicular of the direction of movement V_p volume of ball and *v* is the velocity of the particle .

This drag force acts in opposite direction to the driving force and increase as the square of the velocity, acceleration occurs at a decreasing rate until a steady velocity is reached at a point where the drag force equals the driving force ,

$$(\rho_p - \rho_f)gV_p = C_d A_p \rho_f \frac{v^2}{2}$$

For spherical particle $\frac{V_p}{A_p} = \frac{2}{3}d$, substituting this value we get,

$$v^2 = \frac{4}{3}\, g \frac{(\rho_p - \rho_l)d}{C_d \;\rho_l} \text{.....................(6)}$$

Expression for C_d characteristics of different flow regimes .Flows are verified with Reynolds number .

Reynolds number can be calculated by $R_e = \frac{\emptyset\, v \rho_l d}{\mu}$ and in this equation μ is viscoucity, For this

1. When R_e <1, the flow is laminar flow and for laminar ,$C_d = \frac{24}{R_e}$
2. When R_e =1 to 10^4 , the flow is transitional flow and transitional , $C_d = \frac{24}{R_e} + \frac{3}{R_e^{(\frac{2}{3})}}$ +0.34
3. When, $R_e > 10^4$, the flow is turbulent flow and for turbulent $C_d = 0.4$

For the spherical shape of ball, shape factor is 1.0 . When the flow is considered in open or closed channel then due to effect of inertia force flow may be characterized by depth and flow patterns be classified as critical , subcritical, supercritical [4] . But we have chosen a small distance and small depth for setting up the LASER-LDR pair . In open channel and pipe flow the pattern in small distance and small depth for settling the ball is considered to be as laminar flow. For this flow ball can travel as no disturbing object and itself settling velocity.

For laminar flow substituting $C_D = \frac{24}{R_e}$,$R_e = \frac{\emptyset\, v \rho_l d}{\mu}$ and $\phi = 1.0$ (for spherical) in equation (6) we get ,

$$v_t = \frac{g(\rho_p - \rho_l)d^2}{18\mu} \text{..........................(7)}$$

When a flow occurs in open or close channel, then the velocity of light weight immersed ball in a particular distance is resultant velocity [Fig-3] of fluid particle and settling velocity of ball.

$$\vec{v} = \vec{v}_f + \vec{v}_t$$

$$v^2 = {v_t}^2 + {v_f}^2 \text{....................................(8)}$$

If the travelling distance of ball in the LASER action zone is s (cm) and time is t (millisecond) then the resultant velocity of ball,

$$v = \frac{s}{t} \text{ (Cm/millisecond)}$$

Now, from equation (7) and (8) we ge ,

$$v_f = \sqrt{\left[\frac{s}{t}\right]^2 - \left[\frac{g(\rho_p - \rho_l)d^2}{18\mu}\right]^2} \text{......................(9)}$$

This equation promotes the exact velocity of the flow. Channel section of flow may rectangular, trapezoidal, circular, triangular etc .If the fixed area of the cross section is A (cm^2),then the flow of the section is

$Q = Av_f$

$$Q = A\sqrt{\left[\frac{s}{t}\right]^2 - \left[\frac{g(\rho_p-\rho_f)d^2}{18\mu}\right]^2} \quad \ldots\ldots\ldots\ldots(10)$$

Based on equation (10) a program is loaded into microcontroller which execute a series of codes and display one dimensional fluid flow at LCD display.

4. SYSTEM OVERVIEW

Actually this is a mechatronics and hydraulic project where three electrical , hydraulic and mechanical conceptions are combined. The mechanical part mainly a frame in which two LASER-LDR pairs are spaced keeping at least 20 centimeters between them.

Figure-1 shows the arrangement of the system by means of block diagram.

Fig2: Block diagram of the system.

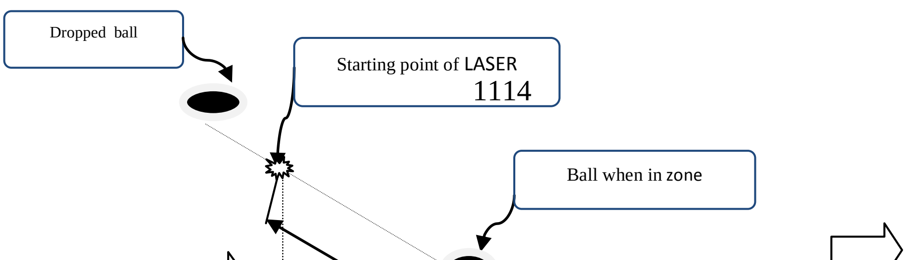
Two sensors which are placed at a certain distance defined the states of the circuit whether it starts counting or stop. When an object e.g. table tennis ball passes through the first sensor microcontroller flag bit starts counting and continue its operation until it passes through the second sensor [6][7]. Once the object passes through second sensor and counting stops and performs further actions to calculate the velocity of that object in the fluid. By the nature of fluid flow may varied . Giving input in the parameters of fluid such as viscosity ,density and diameter and density of particles result of flow of fluid is displayed. It is more significant side for this project is that by this measuring instrument table tennis ball moving velocity can easily be determined and with the parameters gives the flow of fluids.

Moreover it mitigate the exaggerate complexity in design procedure and implementation. The sensor LASER-LDR which has been used here has high speed [8], high resolution and highly accuracy in detection. Laser sensor allows easy position setup and alignment with its visible laser spot [9]. By a complete flow chart the total algorithm has depicted in the following figure.

5.DESIGN AND IMPLEMENTATION

5.1 Mechanical Section:

During design and implementation a sensitive mechanical arrangement is set up to measure the fluid particle movement with the help of nearly weightless object which is passed through two LASER-LDR made sensors and hence the time consumed to pass those sensors is measured by software arrangement.



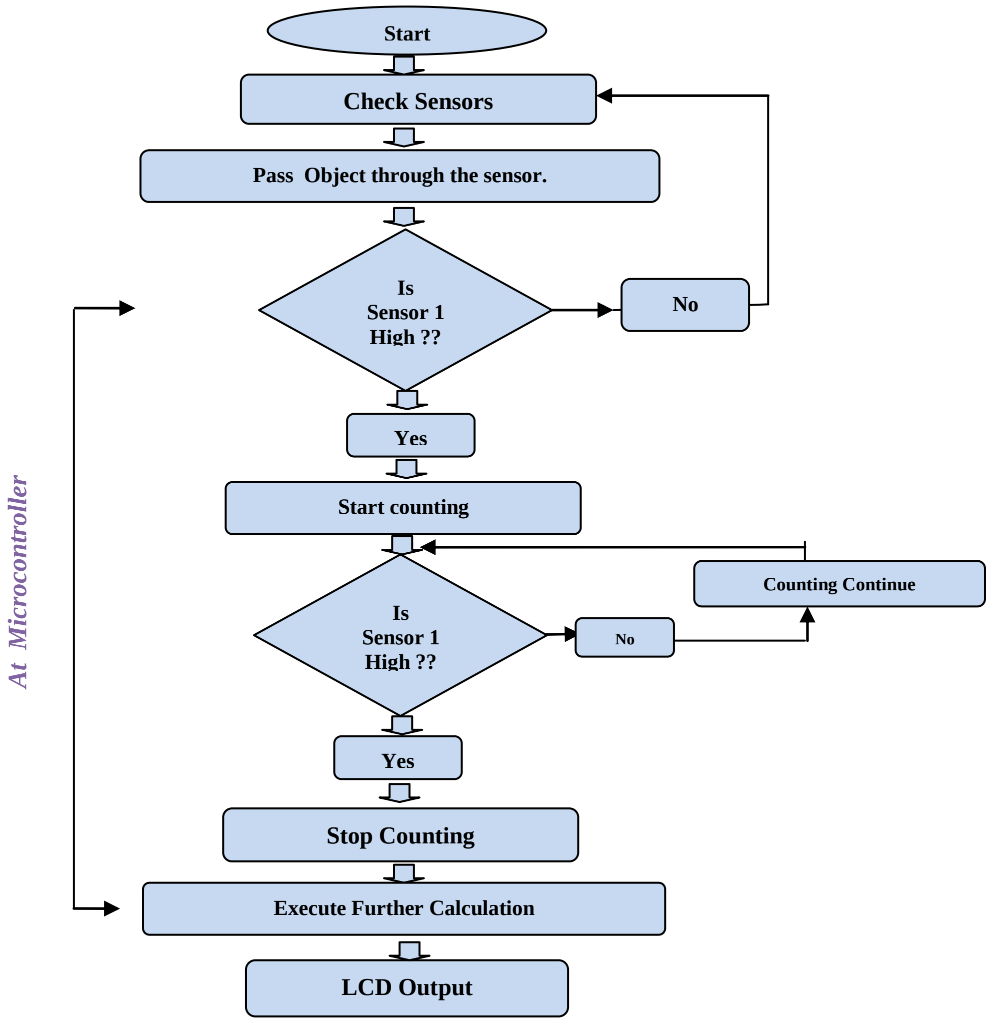


Fig4: Flow chart of the system.

It demands the following arrangement: a stand with two LASER sensors at least 15 centimeter apart each other in one side and two LDRs on the other side as like [fig-3] and[Fig-4]. The stand should be portable and sensitive whose LASER ray must fall on the LDR kept on the other side. So, to implement such kind of stand we use wood as stand material for portability and easy access [10][11]. The sensors are places at the bottom of the stand and 5 holes are kept in the upper side of the stand to detect the floating particle at different water level or depth. The whole arrangement is done only for the testing purpose for the flow measurement of fluid flowing in C4 Tilting Flume [5]. The stand used for sensors placing can move vertically and horizontally in the lab test fluid channel. By changing the vertical position of the stand hence the sensor, flow for a fixed area at different level of the channel can easily be determined as well as changing its horizontal position, velocity at different position of the channel in same level can measure.

5.2 Electrical Section:

In the flow meter circuit arrangement LDRs are used to detect the variation of light intensity of LASER ray. When there is no object or interrupt in between LASER and LDR contact, the whole LASER ray fall on the LDR. As a result LDR shows low resistance but when an interrupt occur means a particle moves through the ray, the state of the resistance of LDR changes to high. The resistance variation is stretched to microcontroller via a high gain operational amplifier [12]. The circuit arrangement is done such a way that when no light fall on the resistor, the microcontroller input bit is low and for light fall, this is vice versa. When the object is passing the sensor and when not these both condition circuit arrangement and output condition are depicted in the following diagram.

In circuit arrangement of microcontroller consists of five I/O ports are recognized by Port A (RA0 to RA5), Port B (RB0 to RB7), Port C (RC0 to RC7), Port D (RD0 to R7) and Port E (RE0 to RE2) [14]. According to loaded program at MCU, its tim0 bit starts counting like a stop watch with higher accuracy [13]. When sensor 2 high (i.e. object passes through the 2nd LESER ray & LDR resistance changes) tim0 ends counting. Total distance is known as two sensors are placed at a known distance & total time is counted. Inputting values of other data, according to the formula of velocity v_f can be easily calculated and displayed in the display.

5.3 SOFTWARE CODING:

The software function of this digital velocity meter design is achieved by Assembly Language. The program basically highlights data acquisition from the sensor and then processes the data in MCU and display real-time data in the display.

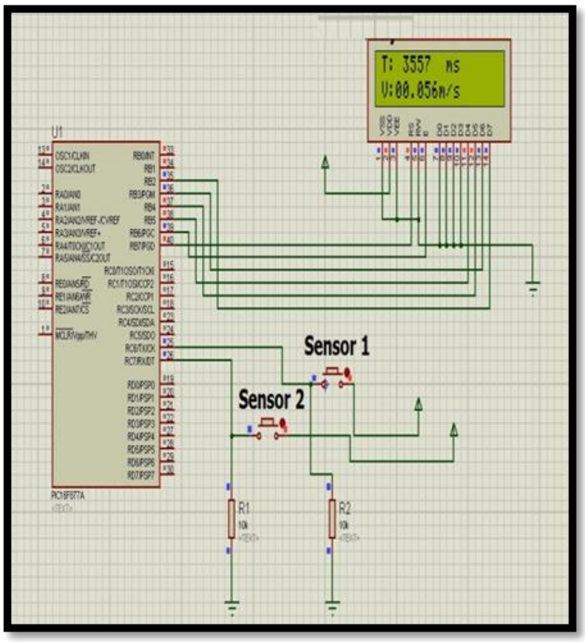


Fig4:
Circuit arrangement and output condition when object not passed through the sensors.

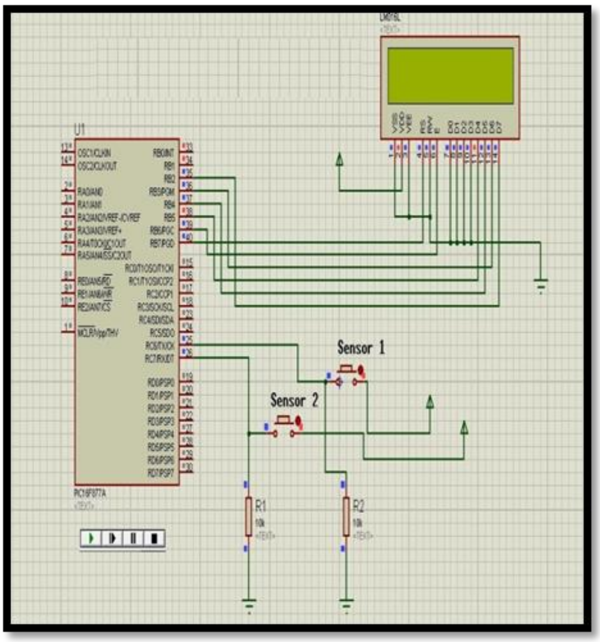


Fig5:
Circuit arrangement and output condition when object crossed two sensor consequently.

5. CONCLUSION

As a flow measuring devices the digital fluid flow meter has a huge application in practical life. It can be used in channel flow measurement and industrial sectors as well as the laboratory experiment very conveniently. By sensing the velocity of any fluid at certain time any machine can make a decision that what to do in certain velocity level such as in case fluid measurement in open channel, pipe, chemical measurement in reaction plant and sand monitoring system in industry. But according to the practical and control capability the ratings of sensors and other parameter will be changed. The sensor input signal of this flow meter is analog which is converted finally into digital by an ADC which is used always in integer. So where there is a very precise flow measurement like biomedical engineering and sophisticated chemical plant there it will give a little bit trouble and the total design should be made in different way.

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AN INVESTIGATION ON THE CAUSES OF FLOODING IN SIRAJGANJ TOWN USING 2D HYDRODYNAMIC MODEL

M.Z.Hasan[1*], M.R.Karim[1] & M.M.Hossain[2]

[1] *Department of Civil Engineering, Dhaka University of Engineering &Technology, Gazipur-1700, Bangladesh.*
[2] *Institute of Water Modelling, Mohakhali, Dhaka-1206, Bangladesh.*
**Corresponding Author, <zakirbuet05@yahoo.com>*

ABSTRACT

The Jamuna is one of the major rivers of Bangladesh having different unpredictable characteristics. As Sirajganj is the adjacent town of the Jamuna, town flooding of Sirajganj is a major concern that mainly depends on the behavior of the Jamuna River. Severe damages have been observed in previous flood years. The objective of the study is to analyze the situation of flooding of Sirajganj town and surrounding areas. To achieve this, a 2D hydrodynamic model of 50 km river reach of the Jamuna (30 km upstream and 20 km downstream of Jamuna Bridge) has been set up with topographic map of Sirajganj town including the Jamuna River produced with the digital elevation model. The results as obtained from 2D model analysis reveal that if the embankment stands safely, flood is not major concern in Sirajganj town. As the topography of Sirajganj town (10-15 m PWD) is lower than Brahmaputra Right Embankment (BRE) (20 m PWD) and the peak flood level of the Jamuna (13.35 m PWD), failure of BRE causes flooding in Sirajganj town and the area around it and damages to lives and properties. In 2007 flood events, the average inundation depth was 0.6-1.0 m. Maximum inundation depth in few areas of town was around 2.0 m. Average velocity of flood water in town area was 0.3-0.4 m/s. Therefore, 2D Hydrodynamic model could be used as an useful tool to determine the scenario of flooding in any area like Sirajganj town.

Keywords: Flooding, 2D Hydrodynamic model, Formation of breaches, Flow pattern

INTRODUCTION

Sirajganj is the entrance district of the north Bengal having the great importance in the field of trade & commerce and transportation. The Jamuna is the main river adjacent to Sirajganj town and it is one of the widest river carrying major runoff throughout the country. It is a braided river where each of channel (anabranches) having most of meandering characteristics. Bureau of Research, Testing and Consultation (BRTC) of Bangladesh University of Engineering and Technology (BUET) identified that severe bank erosion, bed aggradation - degradation, annual flood due to the lowering channel carrying capacity, variation of upcoming wash load due to the change of land use pattern, bed scouring and sedimentation based on seasonal flow variation and sediment carrying capacity, lateral instability causing meandering activities, local scour due to the existing structural intervention are the most common scenario of this river on location and topography basis. As sediment load is proportional to the discharge passing through the channel, due to the insufficient supply of sediment to the channel, it mostly covers the residual amount of sediment pick up rather from river bed or from river bank. Thus river bank erosion is the most common phenomenon for Jamuna and as subsequent triggering to the lateral shifting of the river (BRTC, 2010). This lateral migration in many ocassion damage to the constructed embankment especially on Brahmaputra right embankment (BRE). And there are so many evidence of breaching of embankment during severe flood year of 1988, 1998, 2004 and 2007. This causes flooding of the Sirajganj town. The report of national development programme (NDP) shows that the devastating flood of 2007 seriously disrupted usual life of the people of Sirajganj town. There were severe damages of crops, roads, bridges, culverts etc. More than 2 lakh people were affected in the Sirajganj upazila (NDP, 2007). Therefore, to investigate the main causes of flood, a model based analysis has been executed. The analysis was performed using a two-dimensional unsteady flow (TUFLOW) model based on surface water modeling system as graphic user interface. Topographic map of Sirajganj town including the Jamuna river produced with the digital elevation model was set as input data into the model. The model has been expanded to determine the scenario of inundation depth, inundation area and velocity of flood flow.

MATERIALS AND METHODS

Description of model

TUFLOW is a computer program developed by WBM Pty Ltd and the University of Queensland. It simulates flooding in major rivers including complex overland and piped urban flows, estuarine and coastal tide hydraulics and inundation from storm tides. It solves the depth averaged 2D shallow water equations (SWE). The SWE are the equations of fluid motion used for modelling long waves such as floods, ocean tides and storm surges. All computations are done based on grid. It can also simulate wetting and drying, bridges and culverts, weirs, embankment breaches etc.

Required data for modelling

Hydrodynamic modelling requires various input data like bathymetry, discharge, water level, satellite image etc. Those data were collected from different organizations. The river bathymetric data of the Jamuna River reach adjacent to the Sirajagnj town was collected from Institute of Water Modelling (IWM). This data covers 50 km of river reach (30 km upstream and 20 km downstream of the Jamuna Bridge). Originally this data was collected from field survey. The raw bathymetric data was interpolated with a cell size of 20 m to obtain a continuous river bed. Bangladesh Water Development Board (BWDB) regularly measures daily discharge data at Bahadurabad (BWDB station id: 46.9L) on the Jamuna river. This data was also collected from IWM which cover a time frame of year 2005 to 2009. This discharge data has been used as the upstream boundary condition of the model. Like discharge data, water level data is also needed for the boundary conditions and calibrating the model. The daily water level data was collected at the location of Sirajganj hard point (station id: 49). Downstream boundary of the model is 20 km far from the Jamuna Bridge. At the downstream boundary of the model, water level data has been generated by IWM based on the observed water level data at Sirajganj hard point, Jamuna bridge site and Aricha because there was no observed water level data because of lack of water level station. The generated water level data was collected from IWM with a time frame of 2005 to 2009. The digital elevation model (DEM) of Sirajganj was collected from IWM and was merged with the river bathymetry to get a complete bathymetric plot of the Jamuna River and Sirajganj.

Model setup

Flooding of the Sirajganj is mainly due to the breach of Brahmaputra Right Embankment. So a cut model concept has been adopted for the preparation of flood inundated data in and around Sirajganj town. A two-dimensional hydrodynamic model has been setup for the Jamuna River [Fig. 1] and has been extended for the purpose. For the purpose of the present analysis, this model domain is sufficient enough to avoid the boundary effect at the area of interest because the model covers 50 km reach of Jamuna River from 30 km upstream of the Jamuna Bridge up to 20 km downstream of the bridge (IWM, 2009). The right embankment of the river has been breached to represent the flood scenario, flow type, flow pattern, flow network in a reliable and realistic manner. Information about the breach was collected from IWM. For 2007 year flood event, the right embankment was breached at 3 locations (Songachha, Kholishakura and Khokshabari). A square grid size of 100 m × 100 m has been used for the simulation of the model. Observed daily discharge data has been used for the upstream boundary condition and daily water level data has been used as the downstream boundary condition. Manning's roughness value has been considered as calibration parameter. Different roughness values were used for the river and the town side. The model has been simulated for the flood event of 2007. Water level and discharge data of the year 2007 have been used for the simulation. It has been simulated with 30 seconds time-step for the water level, velocity, area of inundation and depth of inundation.

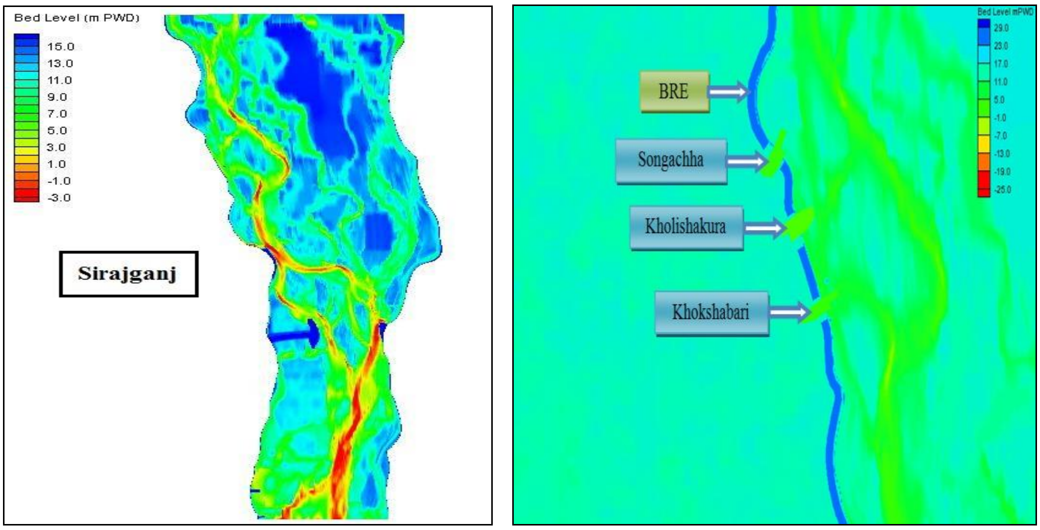
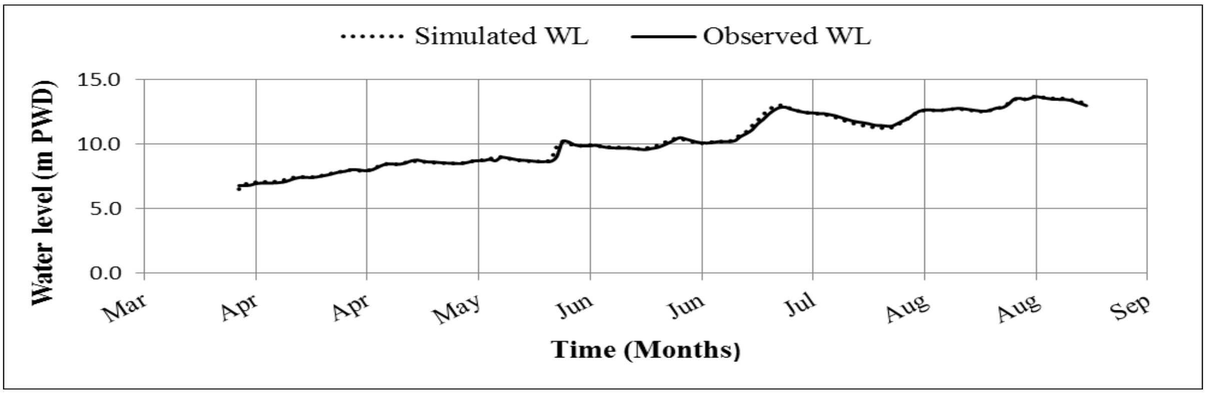


Fig. 1: Model setup of Jamuna River adjacent to Sirajganj with breaches at BRE

Calibration of model

Prior to simulation of the model for existing condition, it is necessary to test the performance of the model. It provides an impression about the degree of accuracy of the model in reproducing river process. This process is known as calibration of the model (Ahmed, 2013). The model has been calibrated using observed daily water level data of the Jamuna river at Sirajganj hard point for the year 2007 (April-August). The graphical representation of the observed and simulated water level is shown in [Fig. 2]. The calibration shows satisfactory result for both high water flow and low water flow.



[Fig. 2]: Comparison of observed and simulated water level at Sirajganj

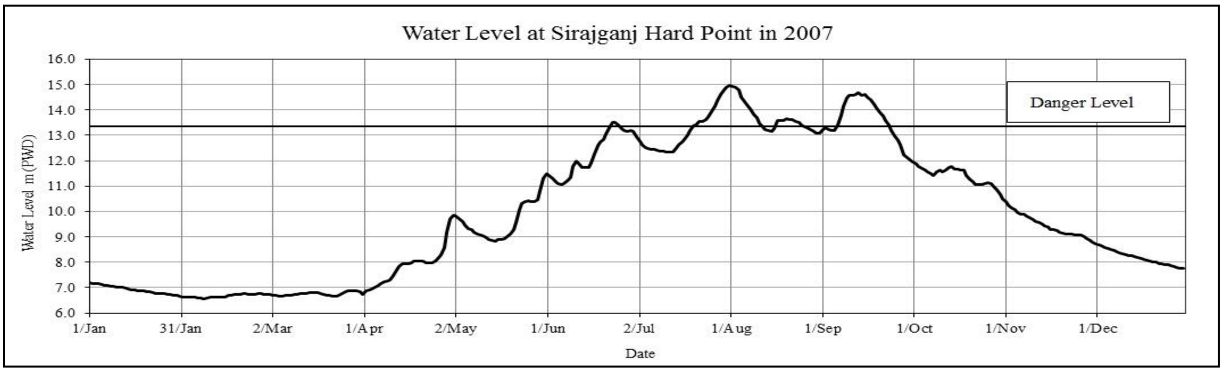
RESULTS AND DISCUSSIONS

Causes of flooding

The DEM shows that the elevation of the Brahmaputra Right Embankment (BRE) is around 20 m PWD. The topography of Sirajganj town shows that the elevation ranges between 10-15 m PWD and the danger Water Level is 13.35 m PWD. Model analysis shows that flooding in and around Sirajganj town occurs when river water enters into the land area. If the embankment stands safely, flood is not major concern in Sirajganj town. When the embankment fails in any place, flood water enters into the town area causing flood in the town and the area around it. In 2007, there were three breaches in the embankment at the upstream of Sirajganj town and heavy flood occurred at Soilabari as well as Sirajganj town. This may be due to the primary and secondary vortices are present into the scour hole at the toe of the embankment. These vortices act as a vacuum cleaner. The amplified shear velocity is also present into the scour hole. The bed materials are washed out from the scour hole. The developed deep scour hole is the main cause of failure of a bank protection structure (Uddin, 2012). In addition, Some flow slides occurred during the fast scouring process due to excessive mica content. The apron materials could not get sufficient time for its settlement resulting the failure of the embankment and hence the flood water enters into the town area and causes great damages (Ahmed, 2013).

Water level

Flood in 2007 was most dangerous for Sirajganj Town and caused severe damages to lives and properties. [Fig. 3] shows that the water level crossed the danger level in 20th July and then continued to rise rapidly. Maximum flood level was 15.00 m PWD, which is 1.65 m above the danger level. During the first peak, the water level was above the danger level for 32 days. Second peak crossed the danger level in 6th September. Maximum water level of this peak was 14.68 m PWD, 1.33 m above danger level. Duration of the second peak was 14 days. The high flood in combination with the breach of BRE caused large damage.



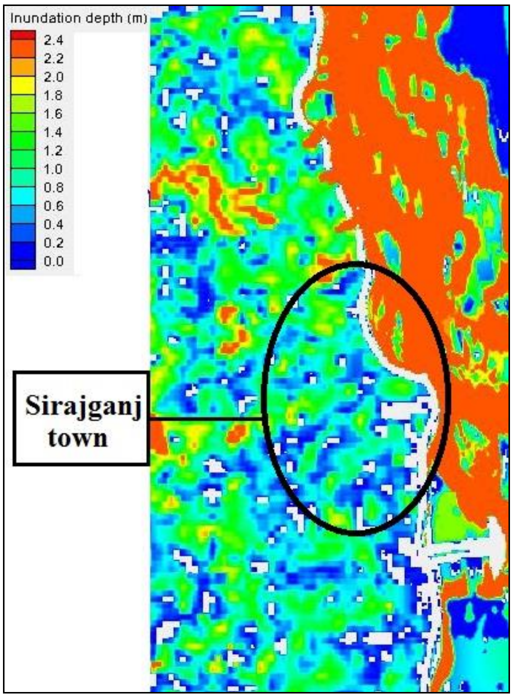
[Fig. 3]: Water level at Sirajganj Hard Point in 2007

Inundation depth

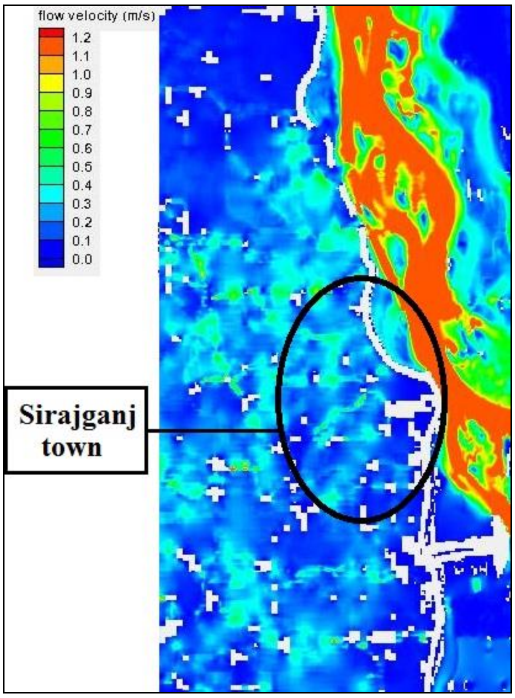
Model analysis shows that the values of inundation depth in Sirajganj town and the area around it due to the flood water entered into the area breaching the embankment at the upstream of Soilabari in 2007. [Fig. 4] shows the model output for the inundation depth of Sirajganj town and surrounding areas on 10th August, 2007. More than 60% of the town area was inundated. The average inundation depth was 0.6-1.0 m. Maximum inundation depth in few town areas was around 2.0 m. In maximum place, the average depth of water was 0.6-0.8 m. The average depth of inundation was around 0.8 m. The maximum depth was almost 1.8-2.0 m.

Flow velocity

Flow velocity of flood water is very important factor as well as inundation depth and duration of flood. Damages due to flood of any area depend on these factors. From the scenario of model results for the Sirajganj town, the maximum average velocity of flood water in town area was 0.3-0.4 m/s [Fig. 5]. The average velocity in the town was around 0.2 m/s. Almost 0.4 m/s was the velocity in most of the inundated areas.



[Fig. 4]: Inundation depth at Sirajganj town in 2007



[Fig. 5]: Flow velocity at Sirajganj town in 2007

CONCLUSION

A 2D hydrodynamic model has been set up to study the causes of flood in and around sirajganj town, inundation depth, flow velocity of flood water and water level. When the right embankment of Jamuna River stands safely, flood is not major concern in Sirajganj town. If the embankment fails in any place, flood water enters into the town area causing flood in the town and the area around it. For 2007 flood, more than 60% area of Sirajganj town was affected with average inundation depth of 0.6-0.8 m and flood water velocity of 0.3-0.4 m/s. Using this 2D hydrodynamic model, the prediction information can be gathered about the effects of flood of Sirajganj town for breaching the BRE at any point.

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ONE DIMENSIONAL HYDRODYNAMIC MODELING OF THE JAMUNA RIVER FLOW

M. M. Rahman[1]*, S. R. Bhuiyan[2]

[1] *Department of Water Resources Engineering, Bangladesh University of Engineering & Technology, Dhaka, 1000, Bangladesh.*
[2] *Institute of Water and Flood Management, Bangladesh University of Engineering and Technology, Dhaka, 1000, Bangladesh.*
**Corresponding Author, <mostafizsust@gmail.com >*

ABSTRACT

Bangladesh is formed by a delta plain at the confluence of the Ganges, Jamuna, and Meghna rivers and their tributaries and distributaries. The Jamuna river is one of the three main rivers of Bangladesh. A one-dimensional (1D) hydrodynamic model can be used to simulate the river flows. The HEC-RAS is a one-dimensional (1D) hydrodynamic model which can perform one-dimensional steady flow, unsteady flow simulation. In this study, a one-dimensional (1D) hydrodynamic model, like HEC-RAS is used to simulate the Jamuna river flow. Different analytical frequency analysis has been carried out to estimate maximum floods of different return periods, i.e., 2, 5, 10, 20, 50 and 100 years base on observed historical discharge data. After that, using Probability Plot Correlation Coefficient (PPCC) and goodness – of – fit test, the best frequency analysis method has been selected for determination of flood level in different flood magnitudes. In 50 and 100 years return periods flood flows are 93289.92 and 96191.60 m3/sec at Bahadurabad point, respectively. The HEC-RAS model boundary condition has been established from the observed upstream discharge data at the Bahadurabad station and downstream water level data at the Sirajgang station. The model calibration and validation have been done by using the observed discharge data. River roughness is the most sensitive parameter in the development of hydraulic model. Hence, in the present study calibration of the channel roughness coefficient (Manning's "n" value) along the river Jamuna has been attempted to develop flood level in different flood magnitudes of the study area. From the model simulation for different return periods flood flow, it is found that water level is increased with return period.

Keywords: Hydrodynamic Model, Frequency Analysis, Water Level, Return Period, Calibration, and Simulation.

INTRODUCTION

Various hydrodynamic models, based on hydraulic routing, have been developed and applied to different rivers in the past using computer technology and numerical techniques for flood forecasting, flood plane mapping and flood volume estimation (Parhi, 2013). The discharge, river stage and other hydraulic properties are interrelated and depend upon the characteristics of channel roughness (Timbadiya et al, 2011). HEC-RAS is the open source software which is user friendly, designed to perform one dimensional hydraulic calculation. With a geo-referenced modeling of river channel, it can be used for assessing the river flow of the Jamuna. Prafulkumar et al. (Parhi et al 2012) calibrated channel roughness for Lower Tapi River, India using HEC-RAS model. Channel roughness is highly variable which depends upon number of factors like surface roughness, vegetation cover, channel irregularities, channel alignment etc. It also depends on such factors as: bed material, vegetation, channel irregularity and alignment, scour and deposition, obstructions, channel size and shape, stage and discharge, seasonal changes, suspended material and bed load (Ramesh et al 2000). The Jamuna , one of the major river of Bangladesh, has experienced several historic floods which have caused huge loss to life and property. The Jamuna River's future flood scenario is studied in this research using one dimensional hydrodynamic HEC-RAS model.

Steady Reach

Bangladesh lies at the confluence of three world's major rivers, namely the Ganges, the Jamuna and the Meghna. The Brahmaputra-Jamuna River, draining the northern and eastern slopes of the Himalayas, is 3000 km long where in Bangladesh, reach length is 240 km (Jagers, 2003). The Brahmaputra-Jamuna inside Bangladesh is the study reach for this study (figure 1).

Figure 1: Location of Full Brahmaputra- Jamuna (left side) and Brahmaputra- Jamuna in Bangladesh (right side). Source: Google Earth https://maps.google.com

MATERIALS AND METHODS

Geometric and hydrologic data

The channel geometry, upstream and downstream boundary conditions and channel resistance are required for conducting flow simulation through HEC-RAS. The cross-section, discharge and water level data were collected from the Bangladesh Water Department Board (BWDB). The HEC-RAS model boundary condition were established from the observed upstream discharge data at the Bahadurabad station and downstream water level data at the Mathura station for 2011.The model validation were done using water level and discharge data of 2013.

Probability distribution functions used

Analysis of how often particular flood intensity is likely to occur termed as Flood Frequency Analysis (FFA), is an important concept in flood analysis. FFA is a technique of statistical examination of the frequency – magnitude relationship. It is an attempt to place a probability on the likelihood of a certain event occurring (Davie, 2008). In FFA, return period (T) is used which have a statistical term meaning the chance of excedence once every T years over a long period. In this present study, yearly maximum discharge data of the Brahmaputra-Jamuna at Bahadurabad station has been used for FFA.

For FFA, commonly used empirical distributions 2-Parameter Log normal (LN2), 3-Parameter Log normal (LN3), Pearson type 3 (P3), Log Pearson type 3 (LP3), Extreme value type 1 (EV1) have been adopted in this study. After that, using Probability Plot Correlation Coefficient (PPCC) and goodness – of – fit test, the best frequency analysis method was selected.

Model description

In the present study, unsteady, gradually varied flow simulation model i.e. HEC-RAS, which is dependent on finite difference solutions of the Saint-Venant equations (Equations (1)-(2)), has been used to simulate the flood in the Jamuna River.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA\frac{\partial H}{\partial x} + gA\left(S_0 - S_f\right) = 0 \qquad (2)$$

Here A = cross-sectional area normal to the flow; Q =discharge; g = acceleration due to gravity; H = elevation of the water surface above a specified datum, also called stage; S_0 = bed slope; S_f = energy slope; t = temporal coordinate and x = longitudinal coordinate. Equations (1) and (2) are solved using the well known four-point implicit box finite difference scheme (HEC-RAS, 2010). This numerical scheme has been shown to be completely non dissipative but marginally stable when run in a semi-implicit form, which corresponds to weighting factor (θ) of 0.6 for the unsteady flow simulation. In HEC-RAS, a default θ is1, however, it allows the users to specify any value between0.6 to 1. The box finite difference scheme is limited to its ability to handle transitions between subcritical and supercritical flow, since a different solution algorithm is required for different flow conditions. The said limitation is overcome in HEC-RAS by employing a mixed-flow routine to patch solution in sub reaches (HEC-RAS, 2010).

Model setup

The total length of the Jamuna river is almost 240 km that starting from Noonkhawa and ends at Aricha. The 1-D HEC-RAS model requires one upstream discharge boundary condition and one downstream water level boundary condition for unsteady flow simulation.

Model evaluation

The Nash-Sutcliffe efficiency (NSE) is used to assess the predictive power of hydrological models (Nash and Sutcliffe, 1970). NSE is computed as shown in equation 3.

$$\mathrm{NSE} = 1 - \frac{\sum_{i=0}^{n}(Q_i - S_i)}{\sum_{i=0}^{n}(O_i - \bar{Q})} \qquad (3)$$

Where O_i is the I th observation for the constituent being evaluated, S_i is the I th simulated value for the constituent being evaluated, $\bar{Q}$ is the mean of observed data for the constituent being evaluated, and n is the total number of observations. NSE ranges between −∞ and 1.0 (1 inclusive), with NSE =1 being the optimal value. Values between 0.0 and 1.0 are generally viewed as acceptable levels of performance, whereas values <0.0 indicates that the mean observed values a better predictor than the simulated value, which indicates unacceptable performance. Coefficient of determination R^2 describes the proportion of the variance in measured data explained by the model. R^2 ranges from 0 to 1, with higher values indicating less error variance, and typically values greater than 0.5 are considered acceptable (Santhi et al., 2001).

RESULTS AND DISCUSSIONS

Model calibration and validation

The data regarding to the floods year 2011 has been used for calibration of Manning's roughness coefficient "n". This study, effort has been made to calibrate Manning's roughness coefficient for single value using aforesaid data and, subsequently, different values have been used to justify their adequacy for simulation of flood in the study reach. Various single values (from 0.022 to 0.027 for main channel and 0.035 to 0.041 for flood plain) have been used to justify their adequacy for simulation of flood in the study reach along the channel for the year of 2011. The comparison of observed and simulated stage hydrograph at Sirajgonj gauging station for Manning's "n" value of 0.022 for main channel and 0.035 for flood plain is shown in Fig 2. Further, considering Manning's "n" value as 0.022 for main channel and 0.035 for flood plain, the flood peak and time to peak for the flood year 2011 is computed and it is observed that there is a close agreement between the observed and computed values.

It has been found from the simulation that Manning's "n" value of 0.022 yields the maximum Nash and Sutcliffe Efficiency of 0.92 and also Coefficient of Determination (R^2) of 0.95.

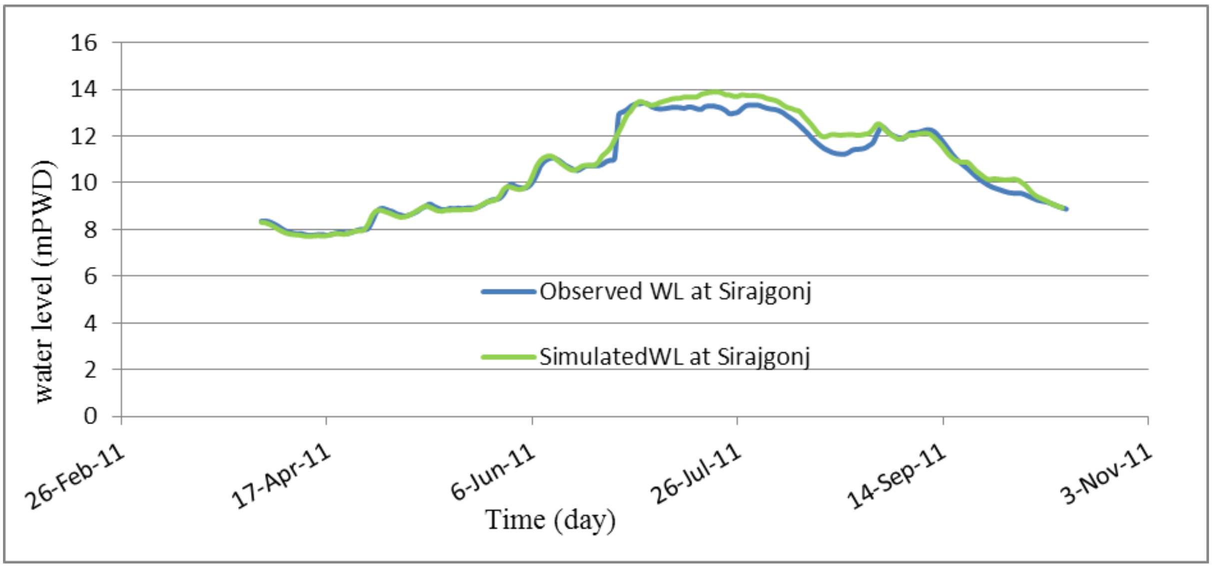

Figure 2: Observed and simulated stage hydrograph at Sirajgonj (calibration) for flood year 2011.

Typical value of Manning's roughness coefficient 'n' is 0.025 for river and 0.040 for flood plain (Chow, 1959). The calibrated HEC-RAS based model has been used to simulate the flood for year 2013. The comparison of observed and simulated flow hydrograph at Ssirajgonj gauging station is shown in Fig 3. Considering Manning's "n" value as 0.022 for river and 0.035 for flood plain, the flood peak and time to peak for the flood year 2013 is computed and it is observed that there is a close agreement between the observed and computed values.

Flood frequency analysis

The hydrologic data of historical discharge have been used to determine designed flood flow for several return periods (2, 5, 10, 20, 50 and 100 year floods) through flood frequency analysis. For

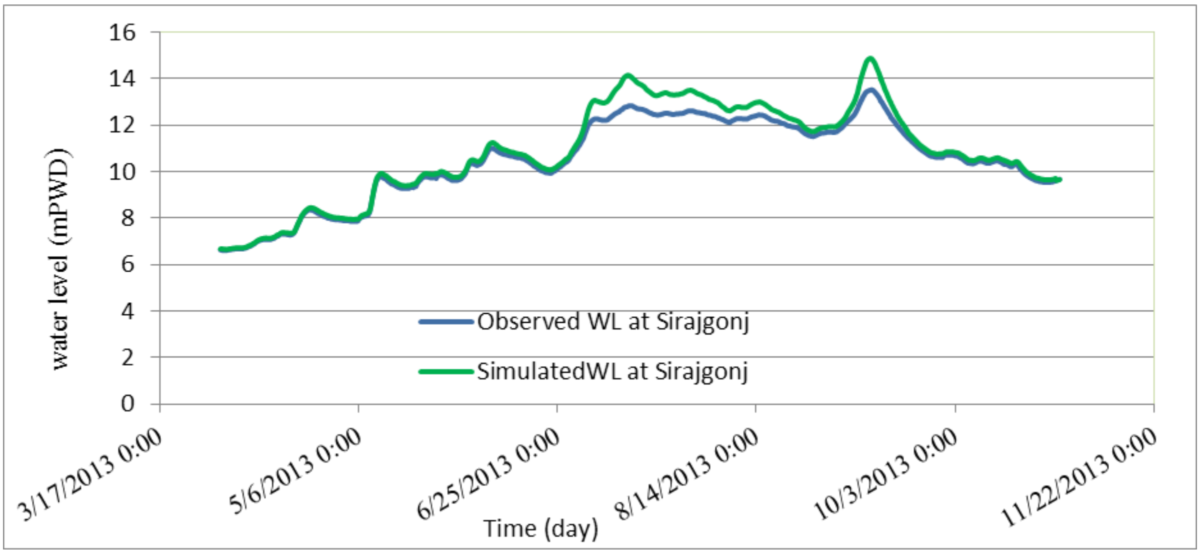

Figure3: Observed and simulated flow hydrograph at Sirajgonj (validation) for flood year 2013

selecting best fitted distribution, goodness-of-fit test was conducted. In the goodness-of-fit test, Probability Plot Correlation Coefficient (PPCC) was applied. The results of the test are shown in Table 1. Based on the result of the goodness-of-fit test, the best empirical distributions in the study area is Log Pearson type 3 (LP3).

Table 1: Flood frequency analysis of discharge data at Bahadurabad gauging station**.**

	Discharge (m^3/sec)							
	2 yr	5yr	10yr	20yr	50yr	100yr	PPCC	Rank
LN2	84536.29	87150.81	90150.82	120077.28	141446.04	157605.68	0.91403	5
LN3	**76267.1**	**78625.88**	**83625.18**	**88648.09**	**94289.92**	**98191.60**	0.95055	1
P3	76204.4	78561.24	82563.14	86981.85	90513.38	92582.55	0.94732	2
LP3	71864.08	74086.68	79066.65	85933.45	106783.08	133075.27	0.92201	3
EV1	74252.49	76548.96	86548.96	93947.42	104973.32	113235.69	0.92094	4

Table 1: Flood frequency analysis of discharge data at Bahadurabad gauging station**.**

Simulation of Flood Flow

After the model was successfully validation to the 2013 flood, the 2, 5, 10, 20, 50 and 100 year flood flow has been simulated. Boundary condition for Steady analysis has been chosen average slope of the Jamuna 0.00007 *(BWDB 2010). The resulting water surface profile reflects the flooding potential for the existing flood plain and account for future development. Water-surface elevations corresponding to these profiles also were calculated (table 2). The results indicate that for the 100 year recurrence interval, the flood profile was about 2.5 m higher.*

Table 2: Different return period flood at different gauging station of the Jamuna River

Rirver station	Water level (mPWD) corresponding to the return period of					
	2yr	5yr	10yr	20yr	50yr	100yr
Baharurabad	18.13	18.35	19.24	19.64	19.88	21.68
Sirajgonj	13.86	14.03	14.73	14.81	14.88	15.88
Mathura	12.57	12.72	13.37	13.668	13.84	14.66

CONCLUSION

Among the different flood frequency method, 3-parameter log normal (LN3) flood frequency analysis method has been best fitted to the historical daily discharge data of the Jamuna River at Bahadurabad gauging station. The most effectives Manning's roughness coefficient calibrated (on flood data of the year 2011) and validated (on flood data of the year 2013) for the Jamuna River comes out to be 0.022. The performance of calibrated model has been verified using Nash and Sutcliffe Efficiency and Coefficient of Determination (R^2) as well. A close agreement for NSE = 0.92 and R^2 = 0.095 have been seen between the simulated and observed flows at Bahadurabad gauging station. This model can be used for future flood inundation mapping.

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A STUDY ON MORPHOLOGICAL CHANGE OF MEGHNA RIVER DUE TO CLIMATE CHANGE BY USING GIS MAP ANALYSIS

S. Hossain[1]*, I. Jahan[2], S. A. Yeham[2]

[1]*Institute of Water Modelling (IWM), New DOHS, Mohakhali, Dhaka-1206,Bangladesh*
[2]*Dept. Of WRE, Bangladesh University of Engineering and Technology, Dhaka,Bangladesh.*
**Corresponding Author, <shlitonbuet@gmail.com>*

ABSTRACT

Geographic Information System [GIS] techniques are effectively being used in recent times as an important tool in determining the quantitative description of morphology of a basin. This technique characterizes very high accuracy of mapping and measurement of morphometric analysis. The aim of the present study is therefore to analyze the GIS based morphometric activities of the Meghna catchment. The means of assessment are field visit/visual inspection, data analysis, satellite image interpretation, analytical assessments and mathematical modeling. The approach in the present study has been to combine the results from the various means of assessment, in order to provide a ranking of sites. Stream velocity and cross-sectional area data, as well as GPS points were collected at four locations on the Meghna stream, and a segment of the stream was mapped using a GPS. The data collected was processed in ArcMap 9.3 software, and used to produce several maps of the study site, as well as some velocity profiles. The velocity profiles obtained from this study may be used to further our understanding of slope stability in order to predict future changes in the geomorphology of the stream. The study results provide latest and reliable information on the dynamic geomorphology of the Meghna River for designing and implementation of drainage development programs and erosion control schemes in the Meghna flood plain region of Bangladesh.

Keywords: Meghna River, River Morphology, Climate Change, GIS Maps

INTRODUCTION

The Rivers of Bangladesh are generally very active in morphological terms. Annual changes in river profiles as well as bank erosion rates have a scale that is far beyond those in low energy river systems, such as in Europe. The annual river floods, the slope of the river valleys, the sediment properties and the vast amount of upstream sediment supply are the main causes for this. Most of the rivers in Bangladesh are part of a large delta and represents as such an unstable state in the morphological development of river systems. Nevertheless, it has been attempted in the past to quantify river processes in Bangladesh by means of data analysis, analytical assessments and mathematical and physical modeling. The data basis for the Meghna River provides natural limits to both the understanding and prediction of the morphological river behavior in these rivers. Due to this fact, a morphological assessment can’t be made on basis of e.g. data analysis or modeling alone. The approach in the present study has therefore been to combine a number of assessment methods and evaluate the combined outcome to provide a ranking of the proposed water intake sites. The various means of determining the suitability in morphological terms are field visit/visual inspection, data analysis, satellite image interpretation, analytical assessments and mathematical modeling.

LITERATURE REVIEW

The following describes in short the key studies in relation to sediment transport and morphology which has been carried out in the past on Meghna Rivers. A basic morphological investigation was made on Meghna Rivers during the Feasibility Study from 2006. The investigation included a review of available data and reports, as well as some basic analysis of cross section development and satellite image interpretation.

FAP studies

A number of studies were carried out in the 1990'ies under the Flood Action Plan (FAP). In total 25 studies were carried out. The studies provided new bathymetry, hydraulic and sediment data within the rivers of Bangladesh, as well as analysis of sediment transport and morphology. Basically all main rivers and many secondary rivers of Bangladesh are covered in the FAP studies. The specialist reports of FAP24 and FAP6 are useful for the present feasibility study.

Hydraulic model study of Meghna Ghat

A hydraulic model study was undertaken by SWMC (now IWM) and DHI (Denmark) for a 450 MW Combined Cycle Power Plant at Meghna Ghat. As part of the study, new bathymetry measurements were made, as well as suspended sediments and bed samples collected in the area of Meghna Ferry Ghat. The data made available during this study is directly useful for the present feasibility study.

Bhairab bridge mathematical modelling

A number of studies were carried out in the late 1990'ies and early 2000's on the morphological behavior of the Meghna River at Bhairab Bazar due to construction of a new bridge. The studies did not provide new data on sediments, but gave insight into the morphological behavior of the river at that location. The study predicted bank erosion rates, and studied the effect of bank protection works. Although Bhairab is located far from the proposed intake sites, the studies contain some information which is useful for the present study.

Hydrological and morphological study of the Meghna River

A study was made on the hydrology and morphology of the Meghna River (BUET & JICA, 2004). The report is divided into a section describing floods and a section dealing with river morphology. The report is essential a series of papers produced by Japanese (through JICA) and Bengali (through BUET) researchers. Some papers on morphology are directly useful for the present study, as they contain data and information of the Meghna River between Meghna Bridge and Baidder Bazar. Other papers are almost entirely theoretical, and do not directly support the present study.

Meghna River bank protection - short term study

A study was carried out by Hasconing in 1990 for BWDB on the upper and middle Meghna river. Apart from a main report, the study included annexes for hydrology, river morphology and geomorphology, geotechnical investigations, scale model studies, mathematical model studies, river bank protection and EIA. Although the study is of older date, the sections on river morphology and geomorphology are useful as sediment survey campaigns are reported, and since plan form development has been analysed on basis of maps and satellite images.

METHODOLOGY

Field Visits to Meghna - Baidder Bazar

A field visit to Baidder Bazar was made on 27th of May 2013. The Meghna River was visited at Baidder Bazar at a reach of app. 1 km. Further a visit to Meghna Ferry Ghat was made. Figure 1 below shows the location visited, with a close up of the area around Baidder Bazar on figure 1.The route indicated was travelled by foot and at most places

Fig 1: ***Locations visited on the Meghna at Baidder Bazar.***

visited bank erosion was taking place.

At Baidder Bazar Ferry Ghat, bank protection works have been provided in recent years. The protection seemed to be sufficient. Further upstream of the river, the bank protection work provided did not seem to be sufficient, and river flows during high water is likely to deteriorate the protection further. At site 3, see figure 3.6, there was no bank protection, but the bank line was clearly eroding. The bank material consists of very fine material, most likely silt with some sand. The bank profile was vertical with some cases of an overhanging bank. According to the local villagers, the bank line has been eroding for quite a few years. The overall impression from the visit is that the entire reach of 1-1.5 upstream from Baidder Bazar is eroding.

Fig 2: *From field visits Left: Baidder Bazar Ferry Ghat and Right: Bank Erosion.*

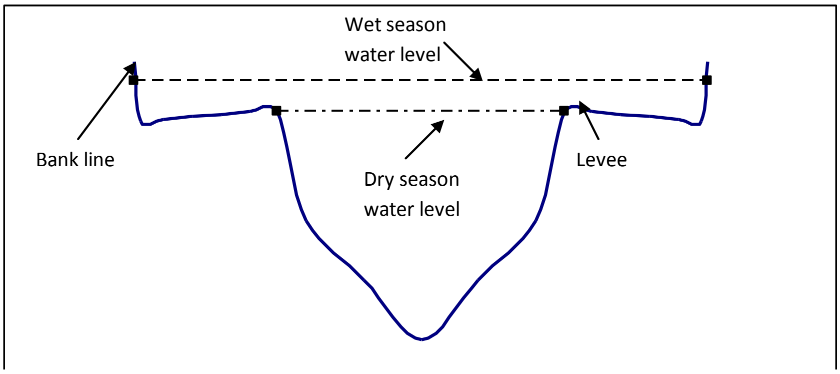
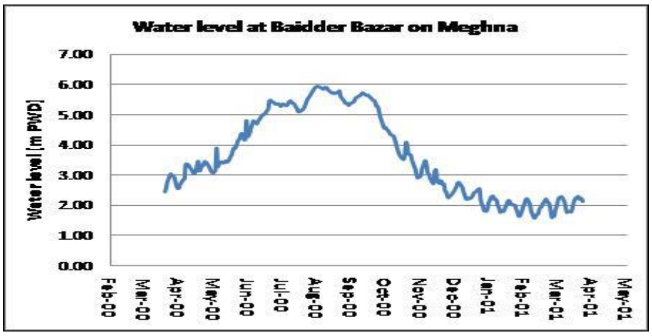


Fig 3: *Schematic view of the separation between a dry season and a wet season channel.*

Water level data exist for 4 stations on the Meghna. For all stations the period covered is 10 years of daily data. The details of the data are seen in table 1.

*Table 1 -**Water level data made available to DWSSDP***

River name	Station ID	Station name	Period
Meghna	SW276	Satnal	01.01.1999-31.10.2009
Meghna	SW275	Baidder Bazar	01.01.1999-31.10.2009
Meghna	SW275.5	Meghna Ferry Ghat	01.01.1999-31.10.2009
Meghna	SW274	Narsingdi	01.01.1999-31.10.2009



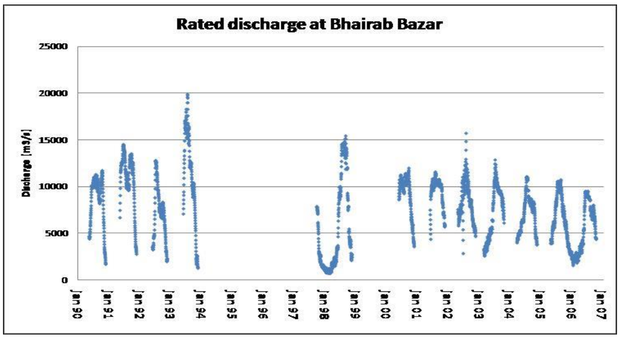


Table 2 -Sediment characteristics in the vicinity of Meghna Ferry Ghat (From SWMC 2001, DHI 2001, BUET & JICA 2004)

Parameter	Value / range
Suspended sediment concentration	Total: 22-123 mg/l Clay/silt: 21-120 mg/l Sand: 1-12 mg/l
Left bank sediment characteristics	99 % sand
Right bank sediment characteristics	10-27 % sand
Bank sediment grain size (d50)	0.13 mm (left bank) 0.026 mm (right bank)
River bed sediment grain size	0.12 mm
Sand bar grain size	0.12 m (d50) , 0.22 mm (d84), 0.035 mm (d16)

BATHYMETRY DATA

Bathymetry data in the form of river cross sections have been obtained from BWDB for various locations on the Meghna River. All together seven cross sections on the Meghna have been received from BWDB. These cross sections are from the years 1998-99 and 2000-01 respectively. Cross sections have likewise been retrieved from the General Model (MIKE11) of Bangladesh by IWM. These cross sections have originally been retrieved from BWDB. The cross sections cover the years 1997-98, 1999-00 in case of Meghna.A bathymetry survey was conducted in connection with the Meghna Power Plant study (DHI, 2001). The coverage of the bathymetry is from Baidder Bazar to Meghna Ferry Ghat, and therefore not of direct use for the present study. The study on Meghna River Bank Protection (BWDB, 1990) contains bank line information for various years derived from maps and satellite images. The report overlays the bank line information from different years in figures to interpret the changes in river plan forms.

HYDRAULIC DATA INTERPRETATION

Implication of the water level and discharge variation

The water levels in the Meghna River vary approximately 4-5 meters throughout the year. This has significant impact on the shape of the river channels and the stability hereof. The variation in water level in two distinct seasons, dry- and wet season means that a clear dry season and a wet season channel can be identified along the rivers one effect of this is that the bed shear stress in the vicinity of the banks during the wet season is larger than if the channel was u-shaped. Another effect is that the wet season banks are not as high as if the rivers have u-shaped cross sections. The water level variation of 4-5 meters results in river banks which become saturated during the monsoon, and gradually dry up during the dry season. In this process the banks may become unstable with bank collapse as a result. However, since the river profile is correspondingly larger in the Meghna.

Observations in relation to sediment data

During the field visits it was observed that the two rivers have relatively steep banks. This suggests that the banks have a certain content of silt and clay. The failure of such banks is difficult to predict as the mode of failure are many (shear erosion, tension cracks, rotational collapse etc.) and since the bank material is mixed cohesive (clay and silt) and non-cohesive (sand).

SATELLITE IMAGERY INTERPRETATIONS

Plan form changes are obtained by comparing bank line positions from different years. The bank line positions have been processed from the satellite images. The accuracy of the plan form changes is controlled by the resolution of the satellite images which ranges from 15 m to 57 m. Thus any difference in bank line position (from different years) which is less than or similar these figures will not be accurate. The acquired satellite images cover a period of 30 years (1980 to 2010). This is a relatively short period when considering time scales of morphological processes in river systems. However, the data available provides information on which reaches of the river have been most active in recent time. The satellite data period is of the same order of magnitude as the design life time of the project. Thus the observed plan form changes provide an idea of the order of magnitude of changes which can be expected throughout the life time of the project. For the Meghna River the bank line positions from different years are plotted on the same maps. The focus is on the reaches which have been selected as potential water intake sites.

PLAN FORM CHANGES OF MEGHNA RIVER

The bank line positions from the years 1980, 1989 and 2000 have been plotted together with the satellite image from 2010 in fig 4. The figure enables on a gross scale to detect the plan form development over decades since 1980. A close up of the area around Baidder Bazar is seen in fig 4. In this figure the direction of the bank retreat or accretion is shown with arrows. From the figure it is seen that the island upstream from Baidder Bazar is eroding on the eastern side, whereas it is accreting on the southern side. This development deflects the flow further southwards resulting in a severe erosion of the opposite bank of the Meghna. Due to the erosion of the opposite bank the flow is forced more directly into the left bank at Baidder Bazar. Further the flow is forced to follow a more curved path. This curvature enhances the secondary flow and leads together with the direct flow attack to increased erosion of the left bank. This is the most likely reason for the observed bank retreat at Baidder Bazar.

Fig 4 Bank lines from 1980, 1989 and 2000 superimposed on satellite images from 2010. Area covers from downstream of Meghna Ferry Ghat to upstream of Baidder Bazar.

CONCLUSIONS

Significant research has been made in the past to identify the stability conditions for various types of rivers and to identify the conditions for which the form or shape of a particular river would change according to the climate change and other effects from environment. There has been much debate on both the validity and the usability of those findings. The present study has attempted to apply the principles for meander stability in order to analyse whether the Meghna River is likely to be in a transition from one type to another. However, the conclusion when applied to the Meghna River remains unchanged even if suspended load is included. However true straight rivers do not exist in nature, hence the diagram should be considered as a continuum of shape conditions. The cross sections as well as the model results (from the General Model) have been inspected and parameters derived for the Meghna River.

RECOMMENDATIONS

According to the results in terms of the parameter variation it is seen that the Meghna River remains in a more clear meandering state due to climate change and erosion accretion process. The results confirm that Meghna is in a meandering stage for the range of flow and depth condition that it is subjected to. This finding is not surprising when analysing the river course on satellite images. These types of branches like Meghna River are usually more stable than typical river braids in e.g. the Jamuna River. The present morphological assessment is concerned with the suitability of climatically changes of river courses along the Meghna River. The main concern for the study was whether the river reaches with a future changes to remain a stable river.

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CHANGES OF RAINFALL IN THE FUTURE OVER BANGLADESH SIMULATED BY A HIGH RESOLUTION REGIONAL CLIMATE MODEL CONSIDERING THE RCP SCENARIOS

Mohammad Alfi Hasan[1*] ,Supria Paul[1], A.K.M. Saiful Islam[1], G.M.Tarekul Islam[1], Sujit Kumar Bala[1], Mutasim Billah[1], Md. Mustafizur Rahman[1], Md. Golam Rabbani Fahad & Muha Abdullah Al Pavel

[1]*Institute of Water and Flood Management, Bangladesh University of Engineering and Technology (BUET), Dhaka, 1000, Bangladesh,*
**Corresponding Author, <mdalfihasan19@gmail.com>*

ABSTRACT

Research related to the changes of rainfall events due to global warming is considered to be one of the challenging fields of research throughout the globe. Extreme rainfall events are responsible for the occurrence of floods and water logging in low-lying areas. On the other hand, lack of rainfall events is a major cause of agricultural droughts and deficiency in water management systems. Bangladesh is one of the countries of the world most vulnerable to natural disasters such as floods, cyclones and storm surges as well as climate change due to its least capacity to adapt these problems. Thus, assessment of change in variability of rainfall and their patterns are very important for the agriculture as well as the economy of the country. To analyze the future changes of rainfall patterns, the Fifth Assessment Report (AR5) of Intergovernmental Panel on Climate Change (IPCC) suggests a new set of emission scenarios which is known as "Representative Concentration Pathways (RCPs)". Climate data used in the study are from the global climate model ECHAM5 developed by the EC-Earth consortium. These global data were downscaled dynamically for the period of 1951 to 2100. Three emission scenarios were used namely RCP2.6, RCP4.5 and RCP8.6. Simulation were made over the large CORDEX domain with spatial resolution of 50km. Downscaled data are used in this study to evaluate precipitation change for the three future time slices: from 2011 to 2040 as the early-21st century, from 2041 to 2070 as the mid-21st century and from 2071 to 2100 as the end of the 21st century with respect to base line (from 1971 to 2000). Possible changes of rainfall events over Bangladesh in the future are presented both in temporally and spatially. Based on the analysis of all the RCP scenarios, significant increase of rainfall has been observed in the pre-monsoon season than that of any other seasons of the year. Climate predictions using the RCP scenarios for the historical periods have less bias than the earlier scenarios used in the Fourth Assessment Report (e.g., A1B, A2, B1 and B2).

Keywords: Climate change, CIMP5, CORDEX, IPCC, RCP Scenarios, rainfall extreme

INTRODUCTION

The observed warming of the world over the past century has produced a significant impact on society, economy, environment and the ecosystems. According to IPCC (2007), the world population will become much more vulnerable due to the climatic changes resulted from continuous global warming. The ongoing climate change and its consequent negative impacts of the future, have been taken increased attention by the researchers and policy makers around the globe. Due to high population density and geographic location, the climate vulnerable country like Bangladesh is also facing many challenges and difficulties to address the issue. Being a monsoon-dominated country, precipitation plays an important role on the country'seconomy as its livelihood is mainly depend on the agriculture and ecosystems services. Thus, any change in the pattern of precipitation cycle will surely affect the economic, society and livelihood of the people. Shahid (2010b, 2011) has already found that the annual and pre-monsoon precipitation has increased and provided some negative impacts on the socioeconomic state of the country. Several other literaturesalso showed concern for the current and upcoming danger related to the change in precipitation patterns over this region under prevailing climate change(Ali, 1999; Haque et al., 2012; Islam and Hasan, 2012; Murshed et al., 2011; Shahid, 2010a).However, all of these studies usedthe SRES emission scenarios and predictions based on models developed before the Fourth Assessment Report. Therefore, it is necessary to undertake initiatives to identify possible changes of precipitation over this region using the new sets of RCP scenarios.

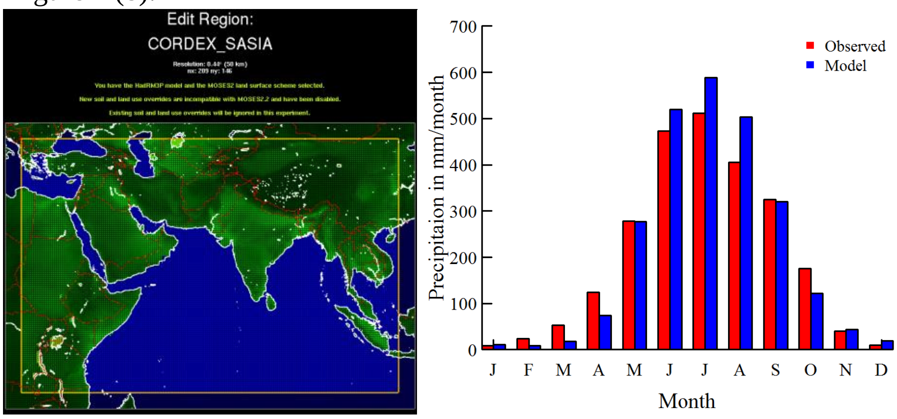
Regional Climate Models (RCMs) using the boundary data of GlobalClimate Models (GCMs) developed under CMIP3 and CMIP5 initiativesare capable of realistically simulating many aspects of the climate and the primary tools for developing projections of future climate change. However, only a few studies have taken over Bangladesh to examine the future climate change based on simulations of the fine scale regional climatemodels.Results obtained from the analysis of climate projections from RCMs provided some valuable information onthe probable future precipitation of the country. From the multi-model ensemble of 17 CMIP3 model simulations, Hasan et al. (2013b) found that the pre-monsoon precipitation is more likely to increase about 15% during 21st century. Nowreen et al. (2014)showed that high intensity precipitation will be less frequent in the north-eastern part of the country in the future. Other similar studies also indicated that the annual precipitation over Bangladesh will be increased in future years(Islam and Hasan, 2012; Islam et al., 2008; Rahman et al., 2012; Rajib et al., 2011). As climate extremes are more sensitive to the global climate change, changes of various indicators of precipitation extremes have also assessed. Based on the multi-ensemble regional climate modelling results, Hasan et al. (2013a) documented a decrease of consecutive rainy days with an increase in intensity of precipitation. However, these previous studiesare done either by PRECIS or by RegCM3 regional model which consist some biases in the control run simulation and are based on CMIP3 simulations(Rahman et al., 2012).But, the ,new sets of climate model outputs become available for the IPCC Fifth Assessment Report (AR5) which are also known as the “CMIP5 multi model dataset”(Stocker et al., 2013). CIMP5 dataset comprise a new set future climate scenarios, called Representative Concentration Pathways or RCP scenarios. Global Climate Models (GCMs) in CMIP5 arebetter in the sense that they represent more of the relevant climate processesin more detail than CMIP3 models. Moreover, they have a wider range of projections which will be very useful to capture wide range of model uncertainties (Knutti and Sedláček, 2013). However, no study on the changes of precipitation over Bangladesh has been conducted using these new sets of CIMP5 Global Climate Model projections. Therefore, this study has been taken an novel initiative to investigate the changes of precipitation over Bangladesh using CIMP5 models projections. Global Climate Models of CIMP5 has provided results in a course scale grid with horizontal resolutions more than 100 km, which is not sufficient for the study of climate change for asmall country like Bangladesh. Thus, GCM outputare dynamically downscalledby a Swedish Meteorological and Hydrological Institute (SMHI)regional climate model (RCM) to produce 50km fine scale climate change information over the South Asia region.Figure 1(a) shows the domain of the regional climate modelling experiments. Using these downscaled outputs, the future precipitation pattern of the country is investigated under new RCP scenarios. The changes of precipitation in the futureare presented in both monthly and seasonal scales.

METHODOLOGY

In this study, EARTH climate model wasemployedto generate precipitation for the historical (1971-2005) and future periods (2005-2100). EC-EARTH model is a global integrated climate model developed by the EC-Earth consortium and also one of the models of CMIP5. The model has various usability in atmospheric and geographic research(Weaver et al., 2001).Fine scale regional climate information was generated by driving RCM using the output of EC-EARTH GCM as a lateral boundary condition. South Asian CORDEX domain was used as model domain for driving RCM model as shown in Figure 1(a). The regional climate modelling experiments werecarried out by SMHI model based on the three representative concentration pathways (RCPs) scenarios: RCP2.6, RCP4.5, and RCP8.5 scenarios.In contrast to the SRES greenhouse gas emission scenarios, the RCPs are the radiativeforcing trajectories adapted by AR5. The scenarios can reflect various possiblecombinations of economic, technological, demographic,policy developments and are not associated with predefined storylines(Van Vuuren et al., 2007).The fine scale (50km)climate projectionsare used which are generated by the EC-EARTH CIMP5 climate model consideringthe three RCP scenarios. Four time slices areconsider to represent the precipitation changes over Bangladesh. Thesetime slices are baseline (1971-2000), early era (2011-2040), mid era (2041-2070), long term era (2071-2100). Comparison of the simulated data with gage-base point observation has been made to assess the model performance in simulating past climatology. The observed rainfall data has been collected from Bangladesh Meteorological Department (BMD). The annual and seasonal changes of precipitation are assessed for the above three selectedtimeslicesin the future. Results also presented as spatial distribution over the whole country to determine the spatial patterns of the future precipitation.

RESULT AND DISCUSSION

A comparison on the mean month precipitation generatedby the model and observation are made and shown in the Figure 1 (b).



(a) (b)

Figure 1: (a) South Asia CORDEX domain. .(b)Comparison of model generated precipitation with the observed precipitation for the historical time slices (1971-2005).

It has found that model is performing well in the spre-monsoon and post –monsoon seasons. It is also revealed that the model in general overestimate the observed precipitation throughout the monsoon. The values are significant during the peak rainfall months i.e. in July and August. The CIMP5 GCM showed better performance than the earlier CIMP3 models as studied by Hasan et al.(2013c) and Islam et al.(2008). While precipitation CMIP5 GCMs are overestimated the CMIP3 GCMsas the previous regional climate model simulations underestimated the monsoon rainfall of the country.

Figure 2shows the projected annual precipitation by the end of the 21st century under the RCP2.6, RCP4.5 and RCP8.5 scenario with respect to the present-day simulation (RCP Historical simulation). It is clear that the annual precipitation isprojected to increase in the whole of Bangladesh, with a largerincrease in RCP8.5 at the end of the century. Under RCP 2.6 and RCP 4.5, the annual precipitation over Bangladeshalso shows incremental trends.

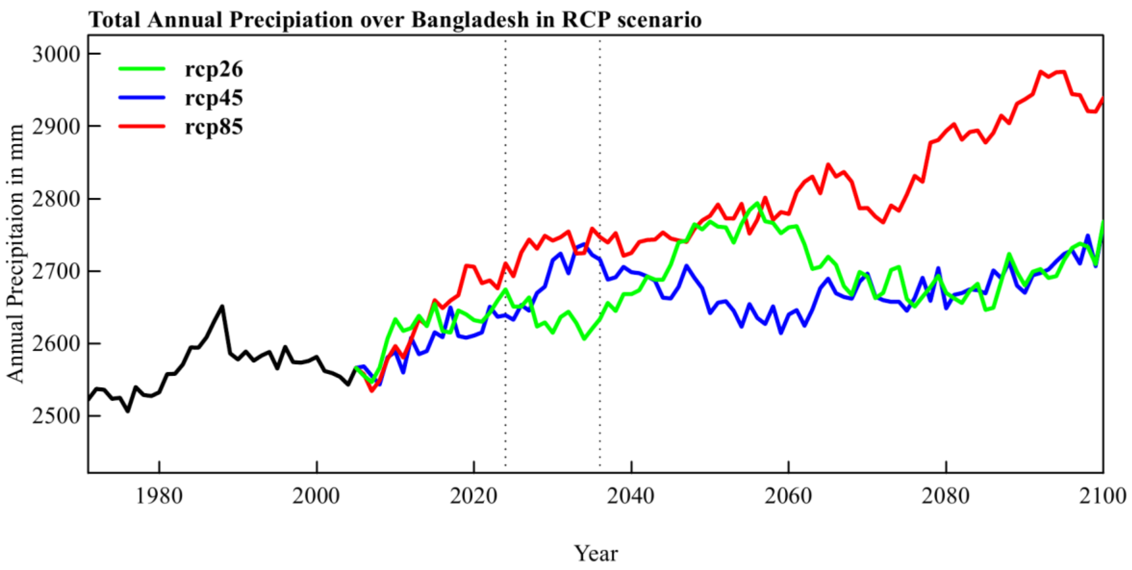


Figure 2: Annual precipitation over Bangladesh under RCP scenarios from 1970 to 2100.

Seasons plays a considerable role in the patterns of precipitation on which economy, development and culture of Bangladesh depends. Seasons in Bangladesh can be classified as Winter (December to February); Pre-monsoon (March to May); Monsoon (June to September) and Post-monsoon (October to November) from hydro-meteorological point of view. To understand the impacts of climate change on the seasonal precipitation pattern, this study made an attempt of using new RCP emission scenarios.Seasonal changes of precipitation for three future time slices are determined under the RCP 2.6, RCP 4.5 and RCP 8.5. A plot of the changes of seasonal precipitation over Bangladesh is shown in the Figure 3. The likely range of increase in the annual precipitation is projected to be 5%– 50% during pre-monsoon and 10% for both monsoon and post-monsoon seasons under the RCP8.5 scenarioin the 2080s era. However, upto 45% reduction of precipitation will be observed during winter in 2050s under the RCP 8.5 scenario. Other two RCP scenarios show higher spatial variability of precipitation comparing to that of the RCP 8.5 scenarios. On the other hand, change of monsoon precipitationis less comparing the other three seasons.

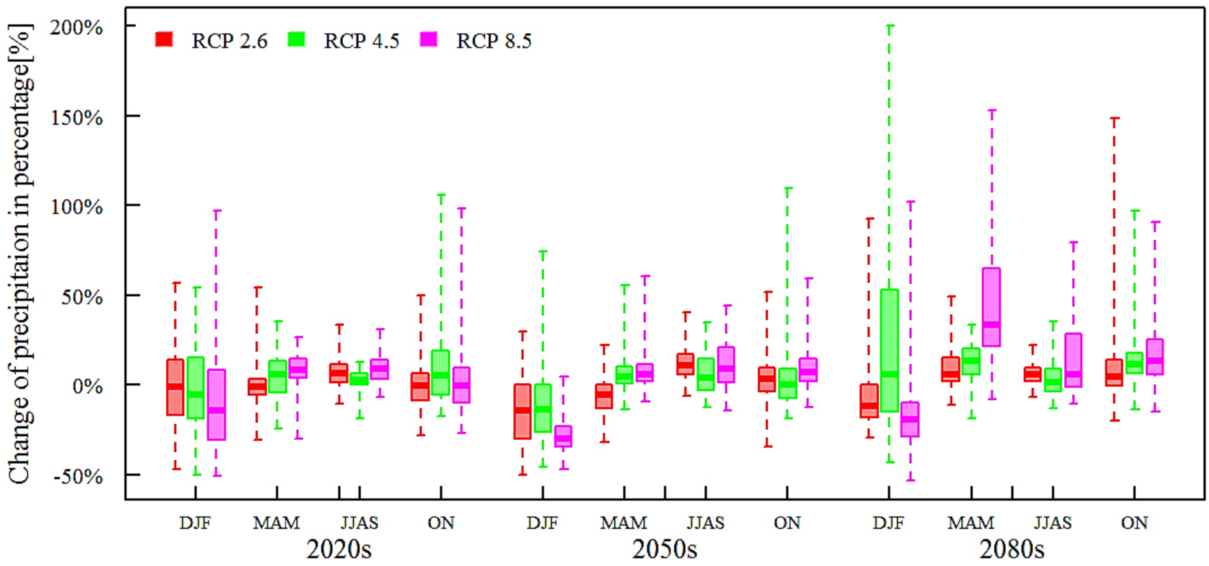


Figure 3: Change of Seasonal Precipitation under RCP scenarios over Bangladesh.

CONCLUSION

A study on the changes of precipitation over Bangladesh have been carried out consideringto determine the inter-annual and inter-seasonal variability. Regional precipitation predictions are generated using theEC-EARTH boundary data for the RCP emission scenarios. Analysisrevealed that regional climate model simulated by EC-EARTH boundary datacan able to reproduce Indian monsoon which was found challenging for the previous modelling studies. Global models forced by the new RCP scenarios showed a considerable improvement in capturingthe past climate of Bangladesh. Under the RCP8.5, the amount of precipitation is likely to be increased during pre-mosoonseasons at the end of century. In the future, precipitation will decrease almost all parts of the country during winter season. Under the RCP2.6 and RCP4.5 scenario, amount of precipitation will be increased during the early parts of the 21st century but became steady in the latter part the century. Climate predictions based on the new RCP scenarios showed improvement in capturing the present day climate. However, further analysis of thechanges of the extreme climate analysis is needed. Moreover, it is also felt that multi model climate predictions will be able to capture wide range of uncertainties of the changes of precipitations over this region.

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ENGINEERING FEATURES OF GANGA BARRAGE PROJECT AND ITS COMMAND AREA

M. A. Matin[1], M. M. Islam[2]*, A. Siddique[2] & M. M. Islam[3]

[1] BWDB, Dhaka 1205, Bangladesh.
[2] Department of Civil Engineering, AUST, Dhaka 1208, Bangladesh.
[3] RHD, Dhaka 1208, Bangladesh.
*Corresponding Author, <mashfiq7777@gmail.com>

ABSTRACT

Of the co-basin countries of the Ganga, Bangladesh is the lowermost riparian and allows the passage of the entire flood flow of the Ganga basin to the sea through its rivers, distributaries and estuaries but the development in upstream in India and particularly the Farakka Barrage has caused immense damage to Bangladesh in terms of availability of dry season freshwater thereby badly affecting agriculture particularly by propagation of sea salinity from the bay, environmental degradation, fisheries etc. Long and protracted negotiations with India a sharing of dry season flow of the Ganga have been arrived. A feasibility and design studies have been done to construct Ganga Barrage and utilize the Bangladesh share of Ganga water for alleviating suffering of the people of the Ganga dependent area. The project covering needs of irrigation, navigation, fighting salinity intrusion is now waiting for a financing plan and implementation. The main barrage on the Ganga in Bangladesh will be 2.1 km long and as by product will have 76.4 MW hydroelectric power and a second barrage on the off take of the Gorai will have 36.6 MW. This paper intends to present salient feature of the Ganga Barrage Project and describe how the people of Bangladesh suffering due to diversion of flows at Farakka may get over the present difficulties of irrigation, navigation, fisheries pushing salinity down so as to get more salinity free area for cultivation.

Keywords: Ganga Barrage Project, Ganga basin, Ganga, Salinity, Freshwater.

INTRODUCTION

The Ganga basin extends over more than 1 million square kilometres and encompasses parts of India (about 80% of the total basin area), Nepal, China and Bangladesh (Fig. 1a). The length of the main channel is some 2,525 km, while altitude ranges from 8,848m in the high Himalayas, to sea level in the coastal deltas of India and Bangladesh. Hydrology in Bangladesh is dominated by the monsoon. Based on this, the Seasons are defined as Monsoon Season: June-September, Post Monsoon Season: October-November, Winter Season: December-February and Summer or Pre-monsoon Season: March-May. In the context of Ganges Water Treaty 1996, the two defined seasons have received prominence. These are Dry Season: January–May and Critical period: March 11 to May 10. Hydrographical System in Bangladesh is dominated among others by the Ganga, Brahmaputra and Meghna river (GBM) system sometimes referred to as GBM system and catchment of the GBM System is presented below. Salient features of hydrographical system in Bangladesh are as follows:

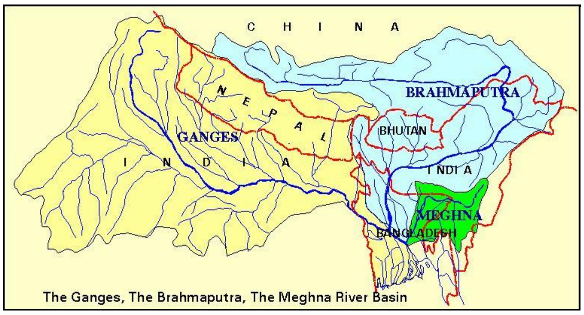
a. Three major river systems, i.e. i) The Ganga-Padma, ii) The Brahmaputra-Jamuna, iii) The Surma-Meghna draining a total of 1,660,000 km^2 lying in China, Nepal, Bhutan and India and confluence of these three major rivers within Bangladesh.
b. Fifty seven trans-boundary river (includes the three major rivers and their tributaries and distributaries) system.
c. Seven independent rivers draining Chittagong and Chittagong Hill Tracts directly into the Bay of Bengal.
d. Inland water bodies of haor, baors and beels.
e. A coastline of over 700 km and large estuarine waters in the coastal districts.

f. Low topography of coastal districts and propagation of tide and salinity from the bay into the inland.
g. Occurrence of coastal cyclones and propagation of associated storm surges causing great losses to life and properties.

Definition of Regional Rivers as a standard terminology is not readily available in publications. BWDB (Bangladesh Water Development Board) has classified the rivers as (i) Major Rivers, (ii) Tributary, (iii) Distributaries, (iv) Independent Rivers and (v) Estuaries. Based on the course of the major rivers dividing the country into a few blocks and also considering few other characteristics, WARPO (Water Resources Planning Organisation) has divided the country into eight hydrological regions by natural boundaries of principal river systems (Fig. 1d), while the hilly areas of the eastern part of the country form another hydrological unit. Based on these principles, eight regions have been defined as the (i) Northwest (NW), (ii) the North Central (NC), (iii) the Northeast (NE), (iv) the Southeast (SE), (v) the South Central (SC), (vi) the Southwest (SW), (vii) the Eastern Hills (EH), and (viii) the active floodplains and char lands of the Main Rivers and Estuaries (RE). Of the three Major Rivers–The Ganga-Padma, the Brahmaputra-Jamuna and the Surma-Meghna, the course of the Ganga rises from the Gongotri glacier on the southern slope of the Himalyas at an elevation of over 8,848 m west of Nanda Devi range in Himachal Pradesh and northernmost Uttar Pradesh, west of Nepal. The river comes out of the Himalyan and Siwalik range near Dehradun and enters the plains at Hardwar. In the plains the Ganga has a easterly course and receives tributaries from the north in Nepal (Mahakali, Karnali Kamala, Gandak, Kosi etc) and from the south in Rajasthan and northern slope of Vindhya parbat (Tons, Sone, Punpun etc.).

The river enters Bangladesh from the west some 11 km east of the Farakka Barrage and flows another about 95 km by the India Bangladesh border before entering totally into Bangladesh. From this point the river flows in a south easterly direction for another 120 km and confluences with the Jamuna upstream of Aricha. The total length of the river up to Aricha is 2,200 km. The combined flow of Ganga and Jamuna downstream of Goalundo assumes the name of Padma and confluences with the upper Meghna upstream of Chandpur and is now called Lower Meghna which after travelling for another 80 km discharges into the Bay of Bengal through a number of estuaries. The greater districts of Kushtia, Jessore, Khulna, Faridpur, and Barisal on the right bank of the course of Ganga-Padma-Meghna is known as the Gangetic delta in Bangladesh. In Bangladesh the Mohananda left bank tributary flowing by the side of Chapai-Nawabganj confluences with the Ganga near Godagari. In its further downstream course the river throws out left bank distributaries the Baral in Natore, and the right bank distributaries the Mathabhanga, Kumar, Gorai and Chandana-Arakandi before confluencing with the Jamuna.

The water diversion structures that India has built up over the years on the Ganga including the Farraka Barrage, are the reasons for the dwindling dry season flow in the Ganga in Bangladesh. Ganga Barrage Project can contribute to a great extent to minimize the effects of lean season reduced water availability over a large part of the country. The perspective view of the proposed Ganga Barrage is shown in Fig. 1c. This project is also critically important to save a vast area which is almost one-third of the size of the country and which has been severely deprived of fresh water flows to it ever since the operation of the Farraka Barrage in India in the seventies. It will arrest land subsidence by reducing dependence on groundwater for irrigation. The barrage will facilitate irrigation of about 19 lakh hectares of arable land in greater Kushtia, Faridpur, Jessore, Khulna, Barisal, Pabna and Rajshahi districts. The project will also allow construction of a 118 to 160 MW hydroelectric power plant with the barrage. The augmented flow of the Padma and its tributaries and distributaries will push back saline intrusion in the lands and save the Sunderbans. Desertification and threat to the overall environment will be much reduced and fishing and related occupations will experience good times again.



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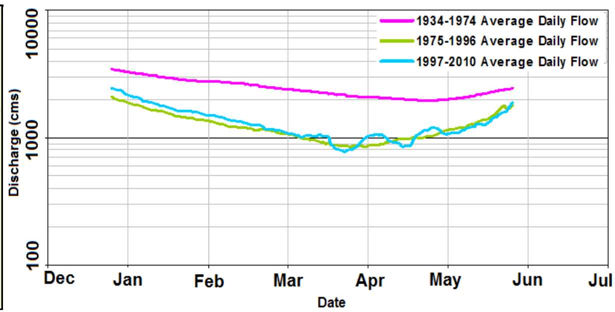

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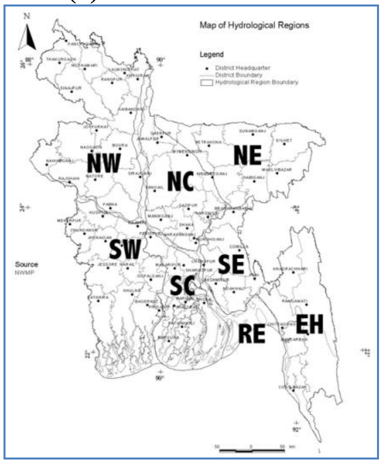


(d)

Fig. 1. (a) Catchment Area GBM System, (b) Superimposed daily average flow curve (dry season) for the three periods (1934-74, 1975-96, 1997-2010) at Hardinge Bridge site, (c) Perspective view of proposed Ganga Barrage (as per Report on Ganga Barrage, 2014), (d) Hydrological Regions.

BACKGROUND AND HISTORICAL GANGA FLOW

The Ganges Water Treaty (GWT 1996) signed on the 12 December, 1996 sets out the basis on which flows arriving at Farakka are shared between Bangladesh and India. The Government of Bangladesh (GOB) decided to examine and evaluate the options for effective utilization of waters available in the Ganges Dependent Area (GDA) because of the implementation of GWT 1996. Table 1 provides list of Ganga Depended Area (GDA) in Bangladesh. The National Water Management Plan (NWMP) strategy took into account these options (NWMP 2004). Accordingly a study entitled Options for the Ganga Dependent Area (OGDA) had been undertaken by the Water Resources Planning Organization (WARPO) under the Ministry of Water Resources. As a follow up of this, the feasibility study of Ganga Barrage in Bangladesh was undertaken under BWDB by a joint venture of Engineering Consultants with Design Development Consultant (DDC) as lead firm. The Joint Venture also utilized the technical support of BUET, RRI, IWM and CEGIS. The feasibility study finalized the location of the Barrage at Pangsha to create a reservoir 160 km long upstream and through a number of hydrologic, hydrodynamic and physical model studies fixed position of Barrage, Hydraulic structural features and points of tapping water from the reservoir (Fig. 2a and b). Discharge of the Ganga in Bangladesh is measured at Hardinge Bridge. Discharge data at this station is available since 1934-2010 (Fig. 1b). Farakka Barrage came into operation from 1975 and data for the period 1934-1974 is considered as historical flow. The flow during the period 1975-1996 was controlled by various short period agreements or absence of any agreement. Flow during the period 1997 onwards is controlled by GWT 1996. Summary of monthly flows for these three periods are presented in Table 2.

Table 1: Ganga Depended Area (GDA) in Bangladesh

Division	Sl. No.	District	Area (km^2)	Division	Sl. No.	District	Area (km^2)
Rajshahi	1	Nawabganj	1702	Dhaka	15	Faridpur	2072
	2	Rajshahi	2407		16	Rajbari	1119

	3	Natore	1895		17	Madaripur	1145
	4	Pabna	2371		18	Gopalganj	1490
Khulna	5	Kushtia	1620		19	Shariatpur	1182
	6	Chuadanga	1158	Barisal	20	Barisal	2790
	7	Meherpur	716		21	Barguna	1832
	8	Jessore	2567		22	Bhola	3403
	9	Jhenaidah	1961		23	Jhalokathi	758
	10	Magura	1049		24	Patuakhali	3205
	11	Narail	917		25	Pirojpur	1308
	12	Khulna	4317			Total	50,802
	13	Satkhira	3858				
	14	Bagerhat	3960				

Table 2: Historical flow (m^3/s) of the Ganga at Harding Bridge

Period	1934-74 (Pre-Farakka)											
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max	5910	5040	4080	3260	4330	20300	50800	72000	73200	57800	26500	11200
Average	3088	2676	2305	2034	2151	4381	17817	38201	35990	17798	7076	4188
Min	1900	1800	1390	1190	1190	1680	3620	13600	12500	5190	3480	2410
Period	1975-96 (Post-Farakka)											
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max	3800	2690	2240	2180	5990	17300	46500	69200	75800	50300	23600	6570
Average	1693	1254	975	932	1335	3433	18664	37290	37718	15823	5401	2745
Min	842	374	261	267	393	622	2620	11600	7300	4190	2010	1520
Period	1997-2010 (Post-GWT 1996)											
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max	5950	2571	1606	1632	3539	22126	45384	67370	74278	52046	17753	28467
Average	1939	1346	945	934	1313	4101	21674	33793	31667	19076	7554	4824
Min	1040	739	212	330	581	713	1911	14034	16488	6340	854	1705

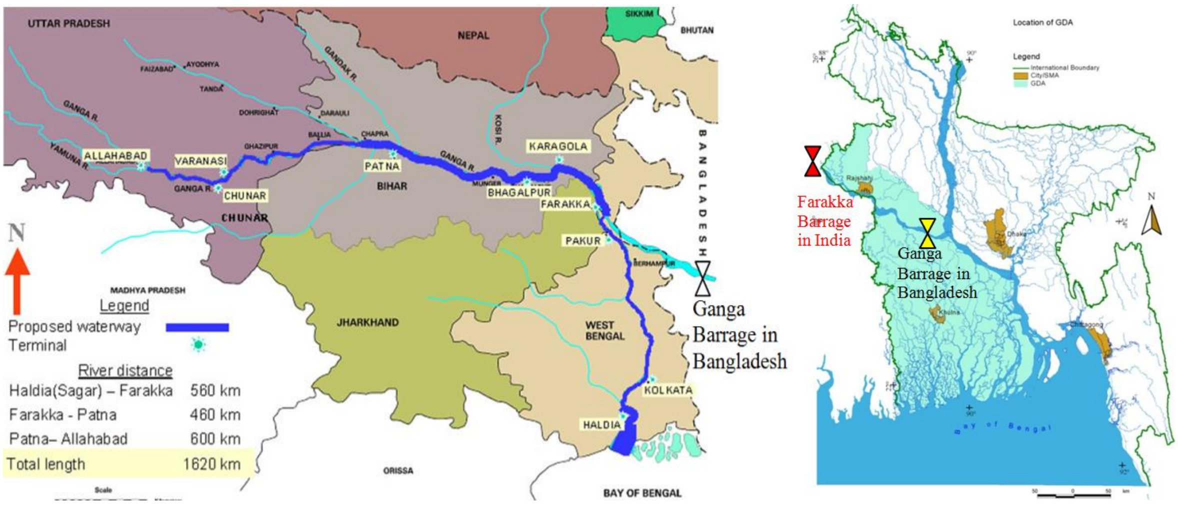
Source: JRC (B) website and Processing & FFWC, BWDB database and complied by Authors'.

SALIENT FEATURES OF FARAKKA BARRAGE

Fig. 2 shows the location of Farakka Barrage, the upstream tributaries coming from Nepal, Uttar Pradesh and Central India and the right bank feeder (diversion) canal leading the flow to the Bhagirathi and Hoogly rivers. The main objective of the barrage is to protect Calcutta port from siltation and to increase navigability of the Bhagirathi and Hoogly rivers. The main components of the barrage are the barrage on the Ganga at Farakka and on the Jangipur canal. A feeder canal from upstream of the barrage to Jangipur is made which is 38.3 km long, 150.88m wide and 6 metre deep. The construction of the canal was started in 1971 and completed in 1974. There is a head regulator on the feeder canal. The main barrage is 2,245 metre long with 109 gates of 23 metres high and 18 metres wide with a discharge capacity of 27,00,000 cfs (76,515 cms, i.e. cubic meter per second).

LOCATION AND ENGINEERING FEATURES OF GANGA BARRAGE

Fig. 2b shows the location of the proposed Ganga Barrage in Bangladesh, the GDA and also the location of Farakka Barrage 11 km beyond the border. From mid October to mid July Ganga upstream will be in reservoir condition, the design full supply level will be 12.50 m-PWD. From mid July to mid October all the gates of barrage will remain open and the river will remain in free flow condition. Fig. 3a shows the location of barrage site on plan of the river and various points of tapping water from the reservoir. Fig. 3b shows water level profile for Ganga River at Barrage location. Proposed diversion of water from Ganga Barrage is provided in Table 4.



(a) (b)

Fig. 2. (a) Location of Farakka Barrage and Feeder canal in India and Location of Ganga Barrage in Bangladesh, (b) Location of Ganga Barrage in Bangladesh and Farakka Barrage in India and GDA/Command area.

Table 3: Salient features of main barrage and allied structures

Sl. No.	Engineering Features	Size
1	Length of Barrage	2.1 km
2	Spillways	78 Gates: Each 18.0 m wide
3	Under sluices	18 Gates: Each 18.0 m wide
4	Navigation Locks	14.0 m wide
5	Fish Pass (2 No.)	Each 20.0 m wide
6	Hydro Power Plant (4 turbines)	Capacity 76.4 MW
7	Left Guide Bank	5.4 km
8	Right Guide Bank	6.0 km
9	Rail Bridge	2.1 km

Table 4: Proposed diversion of water in m^3/s from Ganga Barrage

Link Channel River Intake		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
Left Bank	Godagari Pump House for proposed NRIP	53	51	51	28	38	64	45	30	30	30	30	30
	Baral	27	24	26	18	21	22	15	20	20	20	20	20
	Ichamati & other 4 regulators of PIRDP	17	17	17	20	23	25	17	10	10	10	10	10
Right Bank	Hisna link includes 100 m^3/s for salinity control	400	300	231	234	232	242	184	300	400	500	500	400
	G-K Project Bheramara Pump House		88	90	103	127	136	76	100	100	100	100	100
	Gorai includes 150 m^3/s for salinity control	2500	1000	225	227	225	230	194	500	2500	7600	7600	2500
	Chandana link	300	200	46	57	77	80	44	50	200	300	300	300
	Total	3297	1680	686	687	743	799	575	1010	3260	8560	8560	3360

NRIP: North Rajshahi Irrigation Project, PIRDP: Pabna Integrated Rural Development Project, G-K Project: Ganga Kobadak project.

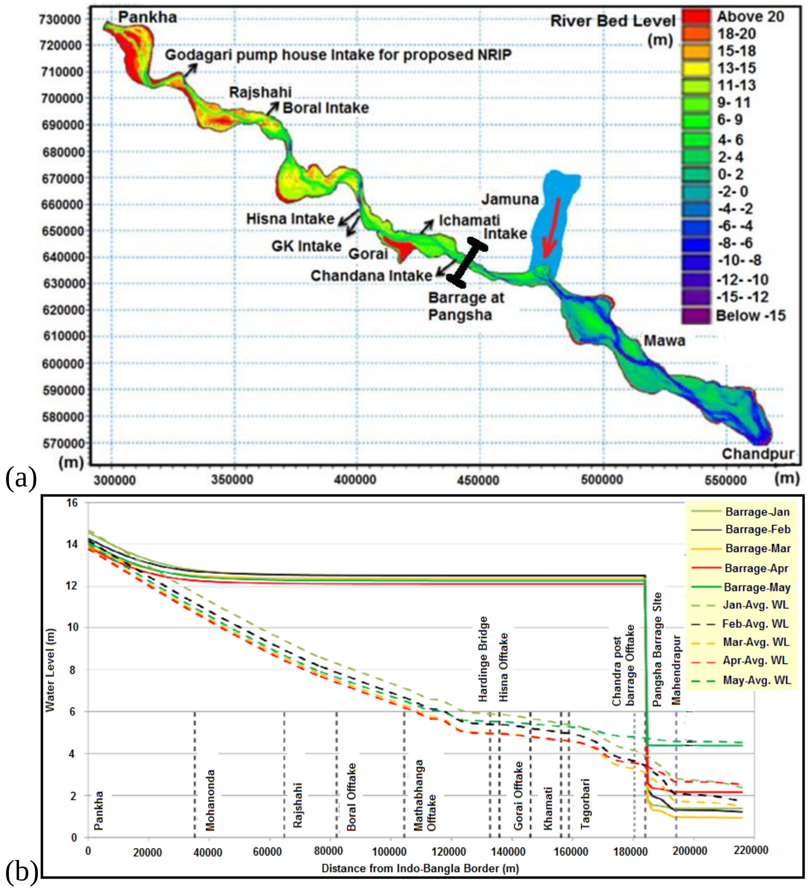


Fig. 3. (a) River Plan from Pankha to Barrage site then to Confluence of Jamuna and Chandpur, (b) Water level profile for Ganga River at Barrage Location: Pangsha, Hydrologic Condition: 12.5m pond level at barrage site.

CONCLUSION

By the implementation of Ganga Barrage, the optimum utilization of Ganga waters of Bangladesh share will be assured as well as GK Project (1,35,000 ha), Pabna Irrigation Project (1,85,000 ha), existing FCD/FCDI projects (12,00,000 ha) and 19,00,000 ha of SW area will be served by surface water Irrigation. The present pressure of underground water extraction for irrigation in NW area will be reduced. At the main barrage and at Gorai offtake structure 113 MW hydro-electricity will be generated almost as a byproduct. Increase of dry season flow in the rivers will push salinity front substantially downstream. In Sundarban about 33% of the high salinity area will be reduced to low salinity area. Proposed flood control interventions will make the project area flood free. There shall be 25,00,000 tons additional food productions and 2,40,000 tons of fish production. There will be substantial expansion of navigation and road transport facilities. Based on 2012 price, the cost of the project will be 31,414 crore taka with a FE component of 103 crore USD. The annual net incremental benefit will be 7,340 crore taka. After implementation of the project cost of the project will be returned from the benefit of the project within 5 years. The project is therefore a candidate of very high priority for implementation.

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PREDICTION OF WATER LOGGING IN CHITTAGONG CITY USING HYDROLOGICAL MODEL

S. A. Mohit[1*], M. A. H. Chowdhury[1] & A. Akter[1]

[1] Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong – 4349, Bangladesh.

**Corresponding Author, <samohit27@gmail.com>*

ABSTRACT

Most of the urbanized part of Bangladesh experiencing rainfall induced water logging due to lack of detailed knowledge on changed hydrological cycle. Human intervention and climate change affects the natural hydrological cycle, thus this study considered factors like hydrological condition, land use and topographic conditions. For modeling setup intensive field survey was carried out and there are total 13 most vulnerable locations were identified. Absence of proper literatures, researchers' needed to collect information through questionnaire survey. The aim of this study is to develop a numerical model for the Chittagong city to predict water logging and this was done in two steps: field survey and thus to provide information for model setup. Field survey indicated during water logging period i.e. May to July, the depth of logged water rises up to 0.6 - 2 meters above the road level during water logging. People living in this area face this kind of situation 3-10 times per year and the area remains inundated about an hour and sometimes even for 48 hours depending on the duration of rain. To evaluate these features GIS version 9.3 software was used for topographical and land use analysis and incorporated these information in HEC-HMS environment. Study showed that the main reason behind water logging is inadequate drainage facility for excess runoff disposing in some places. The outcome of this situation, the reported study is expected to be helpful for decision making on water logging reduction.

Keywords: Water logging, Topography, Hydrology, Model

INTRODUCTION

In last few years one of the main problems that the Chittagong city dwellers are facing is water logging. Rapid urbanization influences this issue and getting worse day by day (Li *et al.* 2012; Sanyal *et al.* 2014; Zhang *et al.* 2014; Su-qin *et al.* 2005). So, this problem has become one of the prime concerns for the decision support system to solve. Also the relevant associated factors for water logging in city areas should be taken into consideration while working on this issue. Naturally hydrological condition of an area comes first as it directly involve in water logging events (Zhang *et al.* 2014). The land use patterns of an area have influences over the hydrological condition while increasing urbanization reduces water body and natural streams. There is an increasing trend observed for land use change due to migrating people from rural parts and this has an adverse effect on the hydrological condition of city areas which sooner or later leads to water logging (Qin *et al.*2013; Djordjevic *et al.* 2011; Fu *et al.* 2014; Ali *et al.* 2011; Zhang *et al.* 2014; Li *et al.* 2012; Suriya *et al.* 2011; Sanyal *et al.* 2014). Water logging mitigation becomes harder, even impossible when the hydrological as well as the topographical condition of an area reach at its ultimate point. Then it becomes the main target to mitigate the loss due to water logging as far as possible. Prediction of water logging events could be a great help in this regard.

Chittagong the second largest and well known port city of Bangladesh started its journey as a city in 1863. Since then, the land use pattern of this city has been changing. A survey was conducted by District Fisheries Department in 1991 and it showed that there were 19250 water bodies in Chittagong city, unfortunately while the survey conducted by Chittagong Development Authority in 2006-2007 showed that the number of water body has decreased to 4523, i.e., the decreasing rate is 920 water bodies per year. In recent years water logging has become a common issue for this city during June-July (Majumder *et al.*2007; Hashemi 2006; Ashraf *et al.* 2009).

The Hydrologic Engineering Center- Hydrologic Modeling System (HEC-HMS) is designed to simulate the precipitation-runoff process of watershed system (Fleming 2010). The model features a completely integrated work environment including a database, data entry utilities, computation engine and results reporting tools. It is used widely as it is simple and works even when minimum data available (Zhang *et al.* 2014). GIS based HEC-HMS simulation output is usually used for design and management of drainage system (Markus *et al.* 2007; Halwatura *et al.* 2013; Ali *et al.*2011; Su-qin *et al.*2005; Zhang *et al.*2014). So far, there is no study covered such a hydrological model to feature water logging for Chittagong city. This paper focused on creating a hydrological model of Chittagong city to predict the water logging and thus reduce the hostile consequence.

MATERIALS AND METHODS

Chittagong city corporation area was selected as study area and study was conducted mainly in three steps [Fig. 1]. 1)Field survey, 2)Secondary data collection, and 3)Model setup.
Detailed questionnaire survey was carried out in Chittagong city for primary data. There were 12 questions in this questionnaire, among them three questions were focused on basic hydrology and rest on urban drainage issue [Appendix A]. The whole questionnaire considered 10% closed questions and 90% open questions [Appendix A]. The respondents for vulnerable locations were selected based on experiences and secondary information. Secondary data was collected from responsible organizations [Table 1]. Spatial data were collected from Chittagong Development Authority (CDA) and Chittagong City Corporation (CCC), this includes word boundary, contour, roads, different structures, water bodies, drains, open area, rivers etc. Figure 1 summarizes the whole work at a glance.

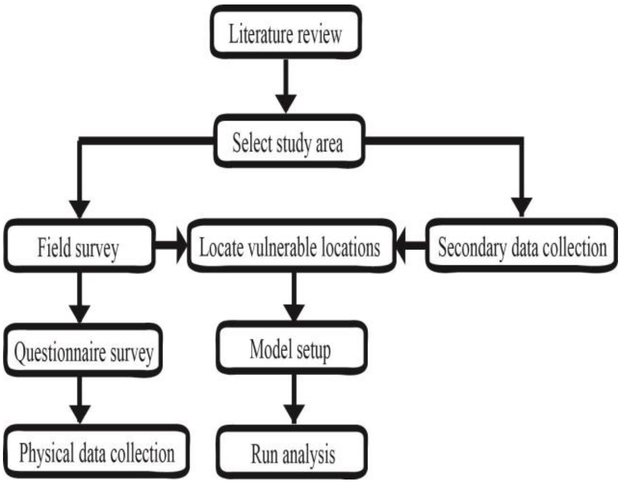

Figure 1: Flow chart for work

Table 1: Collected secondary data.

Parameter	Duration	Source
Precipitation	1982-2013	BMD
Land use	2006	CDA, Hashemi 2006
Soil profile	2012	Hasan *et al.*2012
Population	1991-2011	BMD
Water level, velocity, discharge (Karnafuli)	1998-2013	BWDB
Study area GIS maps	2006	CDA

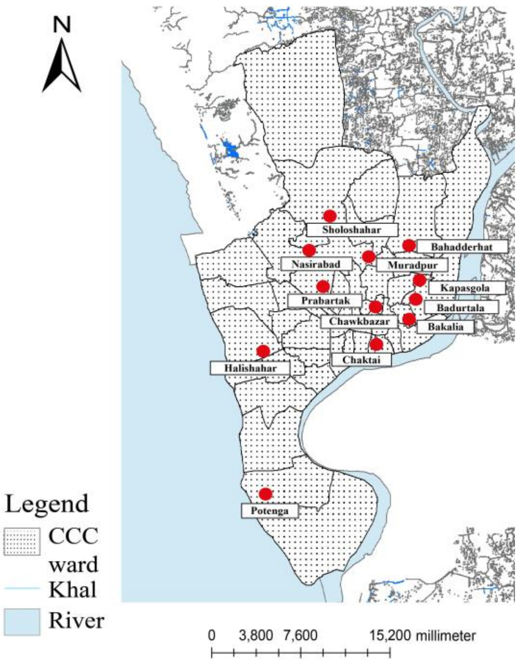

Figure 2: Identified vulnerable locations

PRESENT CONDITION OF WATER LOGGING IN CHITTAGONG CITY

Field survey was conducted over Chittagong city corporation area including questionnaire survey and physical survey. People living in a particular area for more than 40 years was involved in questionnaire survey. There were ten questions and major qustions were to collect informations on depth, duration, frequency, reason, losses, probable solution regarding water logging of that particular area [Appendix A]. Summary of the collected infromation from the questionnaire survey identified 13 areas as vulnerable to water logging [Fig.2]. These areas are Prabartak, Sholoshahar, Bakalia, Bahadderhat, Muradpur, Kapasgola, Badurtala, Agrabad, Nasirabad, Potenga, Halishahar, Chawkbazar and Chaktai. In those areas further physical survey was conducted to gather informations of existing drainage system. Table 2 shows the summary of questionnaire survey data of selected locations. Prabartak and Potenga are relatively developed area and the depth of logged water found here is also relatively low compare to Badurtala and Agrabad. Badurtala and Agrabad areas also face higher logged depth with longer duration. Muradpur, Kapasgola and Badurtala experienced more frequent water logging than others. On the other hand, Bakalia and Chawkbazar suffer longer duration though their logged depth is moderate. Every year water logging experienced these areas and causes heavy damages to residential, commercial and industrial places in many ways. People living in those areas have to suffer even in the post water logging period.

Table 2: Summary of questionnaire survey

Parameters*	**Prabart-ak**	**Sholoshah-ar**	**Bakalia**	**Bahadde-rhat**	**Muradp-ur**	**Kapasgo-la**
Depth(m)	0.3±0	0.76±0.1	1.22±0.2	0.91±0.23	0.91±0.3	0.76±0.4
Frequency(no/year)	1±0	2.5±0.52	4±0.72	4±0.9	11.5±2.3	12.5±2
Duration(hr)	1±0	1.5±0.52	25±16.19	5±1.8	5±2.2	12.5±9.7

Parameters*	**Badurta-la**	**Agrabad**	**Nasirab-ad**	**Pote-nga**	**Halisha-har**	**Chawk-bazar**	**Chak-tai**
Depth(m)	1.22±0.5	1.22±0.4	0.46±0.1	0.3±0	0.46±0.2	0.91±0.2	0.8±0.1
Frequency(no/year)	12.5±1.7	7.5±1.8	1.5±0.6	1±0	2.5±0.5	4±0.7	2.5±0.5
Duration(hr)	12.5±9.5	13±8.9	1.5±0.52	1±0	3.5±0.5	25±16.4	1.5±0.5

* values are represented by Mean ± Standard Deviation

MODEL SETUP

Model setup requires three basic components, i.e. basin model, meteorological model and control specifications. For basin model with the help of Arc-Hydro tool we have separated the city into natural catchments based on the prepared DEM data. Similarly, the natural streams within the city also found. We have created a map of natural catchments and streams of the study area and imported it as background in HEC-HMS v.3.5 environment [Fig.3]. For convenience reaches are denoted as R1 to R26. The Bay of Bengal was selected as the final outlet of the basin, i.e. as a sink.

For flow routing under transform method section 'SCS unit hydrograph' method was employed. In this regard, the required data is lag time (min) and this calculation is based on the following equation, Lag time,

$$t_{lag}(hour) = \frac{2.587 \times L^{0.8} \times \left(\frac{1000}{CN} - 9\right)^{0.7}}{1900 \times H^{0.5}} \quad \text{(Schwab 1993)} \quad \text{Eq. 1}$$

Where,

L (m) = Hydraulic watershed length= $110A^{0.6}$; A(ha)= Sub-basin area [table 1];

CN= Curve number [table 2];

H(%)= Average sub-basin land slope, 0.6% (calculated average)

For reach setup, lag time was calculated as:

$$\text{Lag time, } T_1(\text{hr.}) = \frac{l^{0.65}}{83.4} \quad \text{Eq. 2}$$

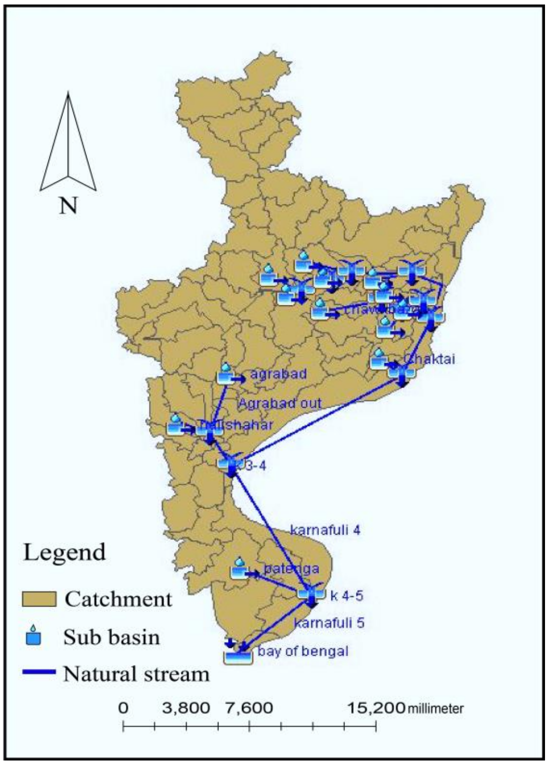

Figure 3: Chittagong basin model

Where, l(m)=largest hydraulic length and this was determined by clipping the sub-basins for natural streams using GIS. All the reaches were considered as lined during reach routing so loss or gain was ignored.

Daily precipitation time series data was provided for Potenga station (22.26˚N, 91.81˚E) for 2013. The derived hyetograph would be used in meteorological model. Under meteorological model, the Potenga rain gauge station has used and the 'Specified Hyetograph' were the input data and calculation ignored evapotranspiration and snowmelt issue.

RESULTS

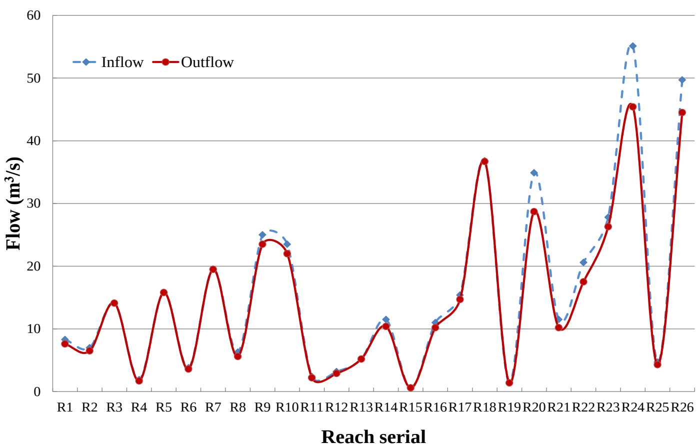

Figure 4: Peak discharges in reaches (04 May 2013)

The first phase of this study identified 13 vulnerable locations for the model set up. In second phase, the model simulation was done for the year 2013 with a time step of 1 day. The simulated peak discharge was observed for 04 May 2013, 28 June 2013 and 30 June 2013. Figure 4 shows the flow values for 04 May 2013, the 18 reaches i.e. R1 to R9, R11 to R17, R19 and R22 showed peak discharge compare to other 22 reaches. The second peak discharge observed in 8 reaches on 30 June 2103 and those are R10, R18, R20, R21 and R23 to R26. As the measured data for these reaches are absent, the model validation is rather difficult. However secondary data showed a severe water logging on 04 May 2013 due to 169.9 mm (BMD, 2013) precipitation and the reported logged water depth was 0.6 to 1.2 meter in

Bahadderhat, Muradpur, Bakalia, Chawkbazar, Agrabad, Halishahar, Kapasgola, Badurtala and Prabartak. Our simulated day 04 May 2013 showed quite reasonable performance in this regard.

CONCLUSION

Water logging in Chittagong city became unbearable issue for every rainy season and to overcome this situation there is a need for updated database. Due to rain induced water logging, 13 vulnerable locations were identified in the first phase of this study. Among these areas Muradpur, Kapasgola, Bakalia, Badurtala, Agrabad and Chawkbazar are facing severe logged water depth along with longer duration. The outcomes of this study also provide an idea on logged water depth, frequency as well as duration. Based on the field survey an attempt was made to setup a numerical model and presently the representative data seems impressive, however to validate this model there is an urgent need for flow data for all the reaches i.e. the city discharge data.

ACKNOWLEDGMENTS

The authors are grateful for the logistic supports of the department of Civil Engineering, Chittagong University of Engineering and Technology (CUET). Appreciation extended to Chittagong Development Authority (CDA) and Chittagong City Corporation (CCC) for their kind support during field survey and providing relevant information.

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TWO DIMENSIONAL MATHEMATICAL MODELLING TO SUPPORT PLANNING AND DESIGN OF THE PROPOSED GANGES BARRAGE IN BANGLADESH

F. Noor[1*], T. Afrin[1], M. Musfequzzaman[1] & S. Jahan[1]

[1]Institute of Water Modelling, Dhaka-1206, Bangladesh.
**Corresponding Author, <fnr@iwmbd.org>*

ABSTRACT

A barrage across the Ganges River near Pangsha site is proposed to meet up the diversion of the fresh water through the offtakes towards the south west region and to reduce salinity level. Two-dimensional mathematical model has been successfully used to reproduce and explore the governing river processes and critical mechanisms working in the vicinity of the barrage during monsoon season. Outputs of the two-dimensional modelling assisted the designer in finalizing hydraulic design variables of the barrage and its ancillary structures over the Ganges. In this regard different sets of two-dimensional models (using modelling software MIKE21C) having different domain lengths and grid sizes have been developed to suit different purposes and different outputs. Survey data of pre-monsoon 2010 has been used as the bathymetric data. In addition, other channel layouts like 1998 and 1980 have also been incorporated in the two-dimensional model considering different approach of flow towards the proposed barrage. The model has been calibrated for monsoon 2009 hydrological data. The developed model has been simulated for the base condition, barrage condition and barrage with proposed river training structures condition. Different analysis have been done based on the requirement such as, scouring around the guide bund, maximum velocity and maximum water level at the barrage and near the guide bund under different flow condition and different layout of channel. It has been found that maximum scour and velocity are generated near the left guide bund among all other location in the barrage area. The study has also come up with other important findings needed for the planning and design of barrage project.

Keywords: Ganges, barrage, two-dimensional, mathematical model, morphology.

INTRODUCTION

The Ganges Barrage Project is one of the most important projects in Bangladesh. Detailed mathematical model simulations are being undertaken to estimate the potential impacts and benefits of the Barrage. Following the success of application of two-dimensional models in other rivers of Bangladesh, two-dimensional mathematical model has been successfully used to reproduce and explore in detail the governing flow processes and parameters active in the vicinity of the barrage.

One of the major components of this study is hydraulic and morphological investigation of the Ganges in relation to the proposed barrage for diversion of fresh water towards the south west region. This investigation includes assessment of probable response of the Ganges, analysis and presentation of hydro-morphological behaviour of the Ganges and the Gorai River system in terms of channel condition, sedimentation, speed & water surface distribution, flow concentration, scour and bank erosion. Prior to this task, selection of suitable site between two locations, namely Pangsha and Tagorbari, for the proposed barrage was assessed from different morphological perspective, i.e. velocity, water level, channel changing tendency, erosion/deposition pattern, char movement etc. Finally, Pangsha site has been selected for the proposed barrage location. Figure 1 shows the proposed location

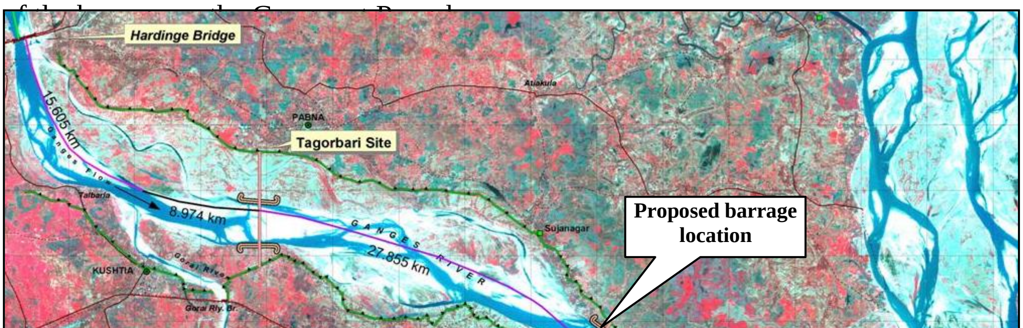

Fig 1: Study area for two-dimensional modelling and proposed barrage location at Pangsha

METHODOLOGY

Application of a fully dynamic two-dimensional morphological model for the Ganges-Gorai river system was adopted for the study. The mathematical modelling focussed on development of the two-dimensional model of the river system comprising the Ganges and the Gorai River, its calibration and verification, assessing suitability of proposed sites for the barrage, assessing the impact of the guide bund, determination of hydraulic design variables of guide bund, assessment of bank erosion prone areas and propose river training and bank protection works, effectiveness of proposed river training works and bank protection works, etc.

In developing two-dimensional model, MIKE21C, has been chosen from advanced mathematical modelling suite of MIKE series of DHI Water & Environment, Denmark. In order to develop the mathematical morphological model, relevant data like bathymetry, bank line, historical discharge, water level, sediment, satellite images and other information have been collected from different organizations and sources. Several steps have been followed to develop the model that includes domain selection, grid generation, bathymetry preparation, boundary generation, calibration and validation. For different purpose, outputs and limitations, different sets of two-dimensional models having different domain length have been developed.

Based on different domain length and purpose, different grids have been generated. Among those, the long model covers the whole length of the Ganges starting from Indo-Bangladesh border to Chandpur including part of major rivers, tributaries and distributaries of the Ganges. Length of this model is 357 km. The medium model stretches from Panka to Ganges-Jamuna confluence covering a length of 247 km of the Ganges. The short model covers the area from Hardinge Bridge to a distance of 5 km upstream of Ganges-Jamuna confluence. Length of this model in the Ganges is 70 km and from the offtake to 15 km downstream of the Gorai. The last one is the cut model, extent of which is the same as short model, except that at the downstream of the barrage location, it has been cut down. Among four different resolutions of model, in this study, different outputs have been generated using the short model. The generated grid of short model is shown in Figure 2.

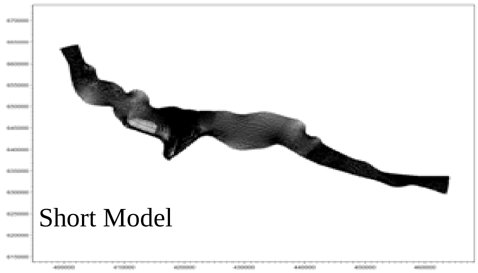

Fig 2: Generated curvilinear computational grids for different purpose

The initial bathymetry of short model has been reproduced with the data collected in the year 2009 and 2010 covering the reach from Hardinge Bridge to Pangsha and 15 km downstream of Gorai River. In addition, other channel layouts like 1998 and 1980 have also been incorporated in the two-dimensional model (Figure 3).

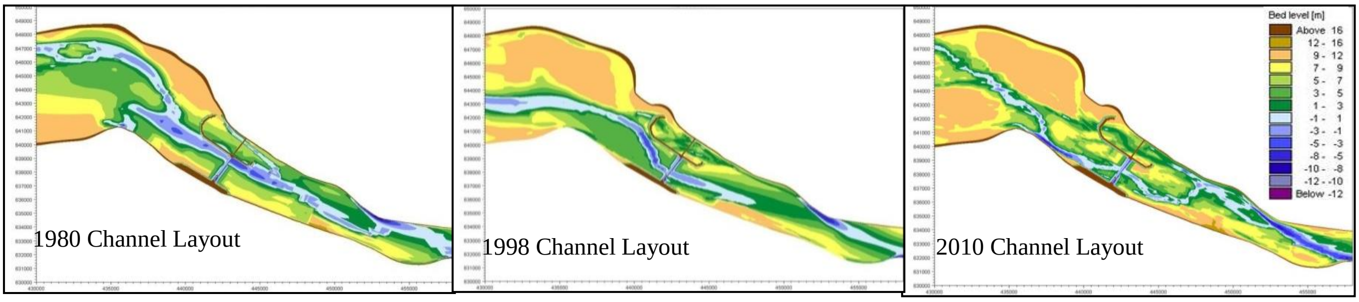

Fig 3: Bathymetry corresponding to the year of 1980, 1998 and 2010

The upstream boundary of short model is located at Hardinge Bridge where there is a BWDB gauge station for discharge and water level measurement. There are two downstream boundaries where water levels were applied, one at the downstream of the proposed barrage and the other at the downstream of Gorai Railway Bridge. Both the downstream boundaries have been collected from one dimensional model.

Steady state simulations have been made with different return periods for discharges and water levels. Four different flood events, e.g. 1 in 25, 1 in 100, 1 in 2.33 and bankfull discharge corresponding to the hydrological year of 1998, 1998 hydrological year with a scaling factor, 2001and 2002, respectively, have been chosen for covering ranges of different severity. Mainly two sets of boundary conditions have been applied; one set was used to calibrate the model while the other was made for application run. In order to calibrate the model, hydrological year of 2009 was used. During hydrodynamic calibration of the model, hydraulic resistance, C is adjusted in such a way so that the model generated water levels at known location can be matched with the observed data, whereas eddy viscosity was adjusted for comparison of velocity.

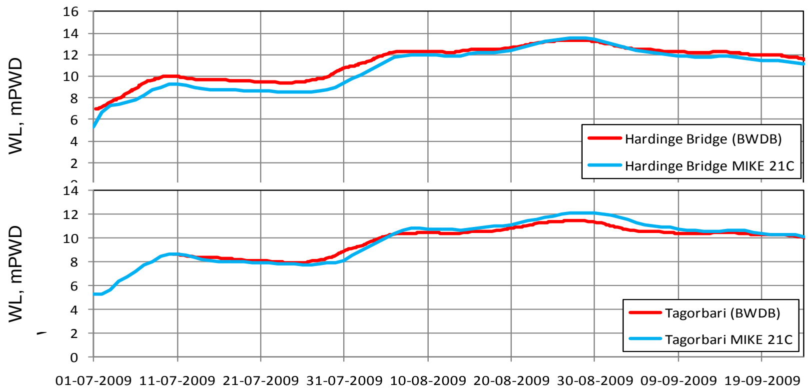

Fig 4: Comparison of observed and simulated water levels at two stations along the Ganges for model calibration, a) Hardinge Bridge, b) Tagorbari

Layout of Barrage

From the comparison of hydro-morphological condition obtained from model simulations, it was decided that it is a better option to have the barrage location at Pangsha. Figure 5 shows the final shape of the barrage which was actually conceived during several trials. The sill level is maintained at -1.5m (PWD) for undersluice and 0.0 m (PWD) for spillway. The number of gates for the undersluice and spillway are 18 and 78, respectively. In order to avoid inundation or spill over the guide bund, 16.5 m (PWD) has been considered for the crest level of the guide bund.

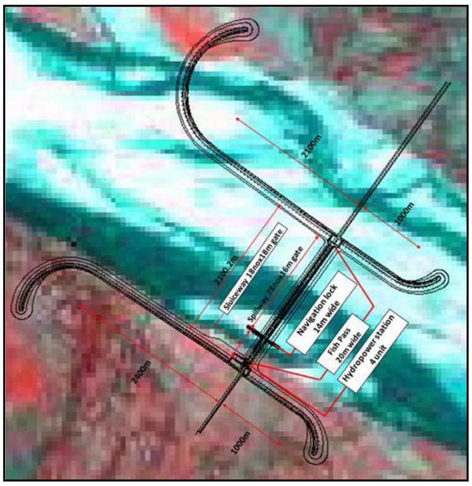

Fig 5: Selected barrage layout

Fig 6: Proposed river training structures with barrage

Layout of River Training Works (RTW) and Bank Protection Work (BPW)

Guide bunds in many cases are not stand-alone structures and depend on placement of additional river training works or hard points. Two T-groynes at the upstream of barrage along the left bank near Pabna have been proposed. These T-groynes are extended upto the existing earthen embankments located along left bank of the Ganges. Though the purpose of the groyne is generally to deflect the current away from the bank, in this case the function of these proposed groynes were be to act as closure dam by closing the left small channel. Such closing would ultimately close the possibilities of flowing the main channel along this path. Even if the main channel follows along the small left channel, then these groynes would diminish the possibility of outflanking of the channel from the left guide bund. A bank protection work at 10km downstream of the barrage location along the left bank and a river training structure at the immediate upstream of right guide bund are also proposed to support the barrage. The proposed river training structures with barrage layout is shown in Figure 6.

ANALYSIS OF RESULTS FROM TWO-DIMENSIONAL MODEL

Based on the flood events of different severity, 2-dimensional models have been simulated for the base condition, with barrage condition and with barrage and other RTW conditions incorporating the barrage layout and other river training structures in the bathymetry.

Scouring around the Guide Bund

One of the important hydraulic parameters for the barrage and its components is the lowest bed level that can undergo during its lifetime. Due to presence of the two guide bunds, constriction has been generated which is apt to cause bed erosion. Considering the safety of the guide bunds and the barrage, scour within this area has been determined. Figure 7 shows the maximum bed scour of the barrage area that can be generated within one monsoon. From Table 1, it is found that when channel alignment follows the path of 1998, highest scour takes place and the bed undergoes at -16 mPWD and the location of maximum bed scour takes place at the upstream termination bend of left guide bund. In case of 2010 channel condition, the flow exhibits a tendency to shift from right to middle of the channel whereas for 1998 and 1980 channel condition, the original flow which was at the left bank remains within the same bank. So, the left bank main channel being beside the left bank, causes more bed erosion and when it shifts, less erosion is observed.

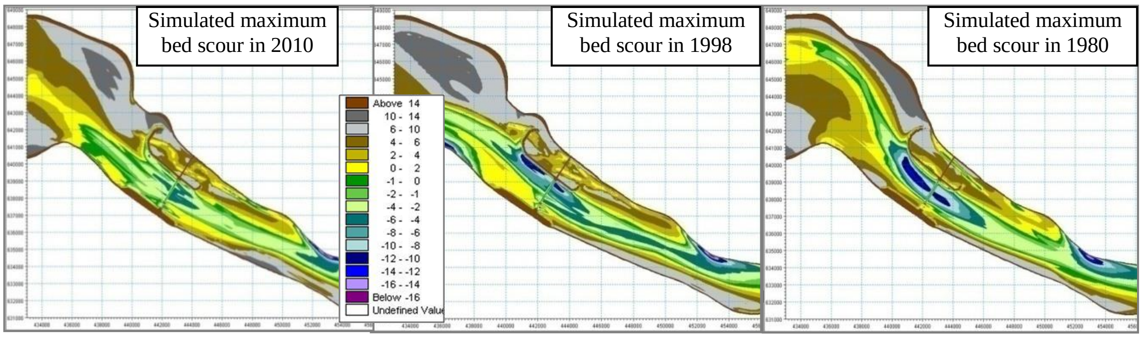

Table 1: Maximum bed scour (m) around the structures after simulation

Location	Channel layout			Maximum bed scour, in m
	2010	1998	1980	
Upstream of right guide bund	1.98	0.01	0.12	1.98
Straight Part of right guide bund	2.98	5.98	4.47	5.98
Downstream bend of right guide bund	4.42	2.08	2.12	4.42
Upstream of left guide bund	6.99	15.86	8.69	15.86
Straight Part of left guide bund	8.18	16.11	4.94	16.11
Downstream bend left guide bund	8.54	9.43	6.54	9.43
Spill bay	12.64	17.14	13.4	17.14
Under sluice	8.16	4.69	2.18	8.16

Bed Level

It is seen from results that the response of the channel in terms of bed level behaves almost the same for without T-groyne condition. This is because the groynes are placed at the left bank where only a small channel exists (in case of 2010 channel condition), and this channel is not so active and prominent that can cause changes or influence the morphological condition within the barrage (Figure 8). Observing the model results of channel condition in 2010 where negligible impacts of the T-groyne were found, it is decided to test the channel condition of 1980. Selection of this simulation is based on the fact that the main channel is along the left bank and the location of T-groyne might obstruct in diverting the flow towards the middle of the section. With this view, a simulation for extreme 1 in 100 year flood event has been done and the results in terms of bed level are presented in Figure 9. The result indicates that when the groynes were not in place, the main channel took a curved form and generated sharp bend at immediate upstream of left guide bund. Such curvature might exert pressure on both the guide bunds. In case of T-groyne condition, the channel shows development of diminishing of the bending tendency. Upon being obstructed from the groyne, the channel starts to flow away from the bank and thus exerts less thrust being parallel to the guide bund.

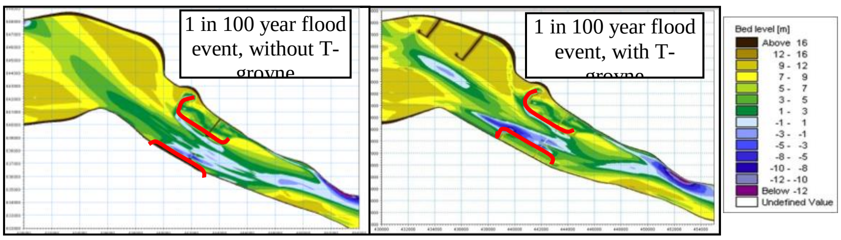

Fig 8: Simulated channel layout for 2010 indicating the impact of T-groyne

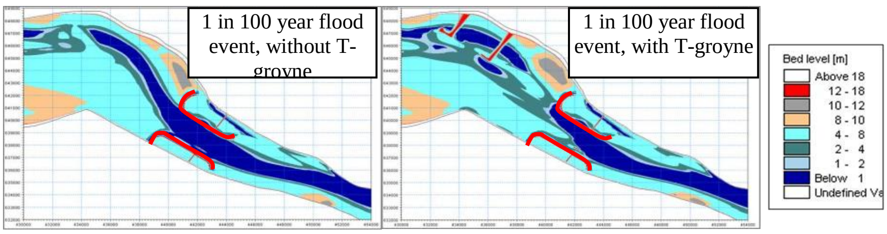

Fig 9: Bed contour for with and without T-groyne condition at the end of monsoon of 1in100 year flood event for 1980 channel layout

Velocity Distribution

In order to verify the design velocity for the barrage and the guide bund, model generated maximum velocity have been extracted and presented in Figure 10. These figures give a good idea on the spatial extent of the velocity contour. However, on designer's perspective, maximum local velocity is necessary. With this consideration, velocities near the barrage location have been presented in the Table 2 that corresponds to maximum of all simulations for different channel layouts.

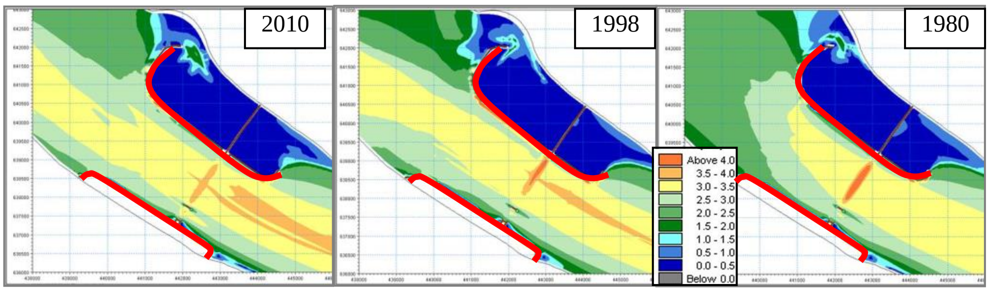

Fig10: Simulated maximum velocity in the vicinity of the barrage axis for different channel condition

Table 2: Maximum velocity (m/s) around the structures

Location	Channel layout			Maximum of maximums
	2010	1998	1980	
Upstream of right guide bund	2.76	2.20	2.29	2.76
Straight Part of right guide bund	2.72	2.19	2.54	2.72
Downstream bend of right guide bund	2.14	1.80	1.83	2.14
Upstream of left guide bund	3.49	3.30	3.53	3.53
Straight Part of left guide bund	2.96	3.28	2.79	3.28
Downstream bend left guide bund	3.80	4.15	3.78	4.15
Spill bay	3.90	4.76	5.12	5.12
Under sluice	3.56	3.12	3.09	3.56

Crest Level of Guide Bund

Figure 11 shows that the water levels within the barrage ranges from 14.1 to 14.5 mPWD and upstream of the guide bund, it varies from 14.5 to 14.9 mPWD which are well below of the design level, 16 w graphical represen te Table 3.

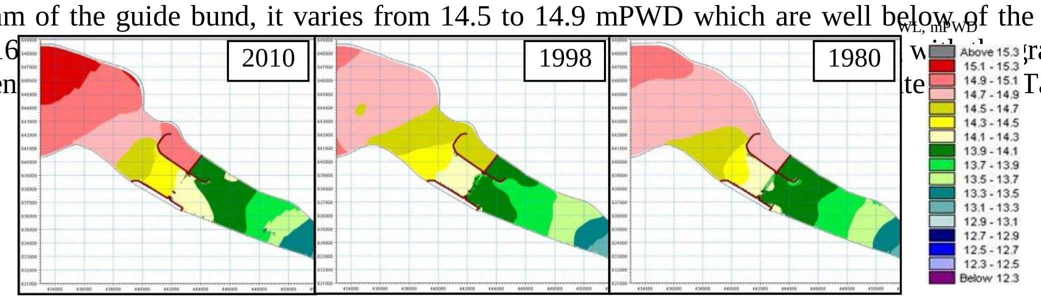

Fig11: Simulated maximum water level within the barrage area for different channel condition

Table 3: Maximum Water level (mPWD) around the structures

Location	Channel layout			Maximum of all events
	2010	1998	1980	
Upstream of right guide bund	14.74	14.44	14.59	14.74
Straight Part of right guide bund	14.57	14.27	14.42	14.57
Downstream bend of right guide bund	14.35	14.18	14.19	14.35
Upstream of left guide bund	14.83	14.40	14.80	14.83
Straight Part of left guide bund	14.54	14.30	14.20	14.54
Downstream bend left guide bund	14.07	13.88	14.07	14.07
Spill bay	14.47	14.27	14.29	14.47
Under sluice	14.47	14.27	14.29	14.47

CONCLUSION

The following concluding remarks have been made based on the study results:

Analysis of model results for all events indicates that the Pangsha site is more suitable than the Tagorbari location, as the velocity is low compared to Tagorbari site and an optimum waterway is found to achieve a minimum afflux.

The maximum scour takes place near the left guide bund for 1998 channel layout.

The location of T-Groyne is effective when the channel layout follows the 1980 channel condition.

Huge velocity is generated near left guide bund and spillway, though this velocity does not persist long.

The water level within the barrage ranges from 14.1 to 14.9 mPWD which are well below of the design level. Hence the risk of overtopping of water level can be ignored.

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The authors express their deepest gratitude to Dr. Ranjit Galappatti, Dr. Søren Tjerry, Development Design Consultant (DDC) and the Institute of Water Modelling (IWM) for the support to use their study findings in the paper.

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SUPPLY CHARACTERISTICS AND ALLOCATION OF WATER DEMAND FOR A DENSELY POPULATED URBAN AREA OF BANGLADESH

S.A. Sayeed[1*], S. Hossain[2] & N. Sharmin[3]

[1] 202/1/3, North Shamoli, Sher-e-Banglanagar, Dhaka, Bangladesh.
[2] Institute of Water Modelling, Mohakhali, Dhaka-1206, Bangladesh.
[3] House #96, Road # 4, Mohammadi Housing Limited, Mohammadpur, Dhaka-1207, Bangladesh.
**Corresponding Author, <saffat.sora@gmail.com>*

ABSTRACT

Dhaka is one of the most densely populated cities in the World which needs huge amount of water daily. Population and economic growth is increasing the demand for water, while climate change, pollution and over extraction of groundwater are decreasing the Dhaka's supply of fresh water. While technological and proper management reforms can increase supply, allocation for sustainable development calls for water conservation. This paper explores the potential of residential water allocation in Dhaka using proper water demand estimation. The increased reliance on demand-side management policies as an urban water consumption management tool has stimulated considerable debate among Engineers, water utility managers, regulators, consumer interest groups and policymakers. In turn, this has fostered an increasing volume of literature aimed at providing best-practice estimates of price and income elasticity, quantifying the impact of non-price water restrictions and gauging the impact of non-discretionary environmental factors affecting residential water demand. This paper provides an empirical residential water demand analyses conducted in the next 20 years. Both model specification and estimation and the outcomes of the analyses are discussed. To assess the potential for urban demand side management, we analyse the extent of residential demand and their distributional implications by type of household. Using detailed household-level panel data for DMA 315, the results suggest that the ultimate effects of reduction in aggregate demand and distribution of water savings among household classes depend both on the policy instrument selected and the composition of aggregate demand. Water demand in the Dhaka was simulated to year 2030 for two scenarios.

Keywords: Water Demand, Water Supply, Urban water management.

INTRODUCTION

The underground water level of the mega city Dhaka is decreasing in such a high rate that the water level may reach to a hazardous position in future. Importance has not yet been given in this vicinity even in geo-environmental research. The scope of this particular research provided a prospectus for proper and quality water supply, demand allocation and distribution system of Dhaka City on the basis of those important issues mentioned above. Besides other research work on Dhaka, this research also suggested careful monitoring especially the problems like ground depression, water logging of flood water and scarcity of water supply.

OBJECTIVES

The prime goal of the design of this water supply distribution system of study area is to satisfy consumer needs providing with reliable, continuous and 24 hour pressurized water supply system as well as minimizing cost for the implementation and operation of the system. For the rehabilitation of distribution network of study area, a model was prepared to check the performance of the system for existing (2013) and future (2030) scenario. Considering the rapidly depleting groundwater level and keeping consistency with future water supply plan of DWASA, surface water from transmission main was considered as future source whereas groundwater as existing source. Where transmission main exists it is considered jointly with groundwater as existing source. Incorporating present and possible future sources in model, final design was carried out. System was improved by minimizing head losses in pipes and utilizing the heads provided from sources. In general the design criteria of head loss gradient (5m/km) as mentioned in the tender document were followed, but for some larger diameter pipes of smaller length- considerations for cost were given preference over head loss criteria, as well as in the pumping zone. To minimize the overall construction costs, pipe diameters were determined keeping consistency as much as possible with the existing pattern of the system. To overcome the possible risks and uncertainties and ensure improvement of the integrated behaviour of system, several Inter-DMA Connections with meter chambers were considered. Every meter chamber was designed to allow water flow in both directions maintaining the required upstream pressure with the help of Pressure Sustaining Valve (PSV). Experiences of field surveys, effects of recent and possible future infrastructure development were incorporated in the model to get the best design of the system. Considering the head loss gradient criteria if possible and understanding implementation risks, pipes have been downsized to minimize the cost. For optimum operation, the system was checked for possible segment shut down to minimize the offline areas during maintenance work. Contingency plans were developed to satisfy consumer needs in the event of a key facility such as pump station and/or surface water interconnection fails.

LITERATURE REVIEW

DMA is a small discrete area with its own water supply system and distribution network for a community and isolated from remaining network without affecting supply system of other areas. Supply source for a DMA may be groundwater (PTW) and/or surface water. Water balance in a DMA can be accounted from its source capacity and consumption.

Benefits of DMA Concept

- ✓ up-to-date water balance information
- ✓ Minimized non-revenue water (NRW).
- ✓ Easy detection of leakages and illegal connections.
- ✓ Energy efficient system.
- ✓ pressurized system
- ✓ Improved water quality.
- ✓ Better customer satisfaction.
- ✓ Turning DWASA into a profitable organization.

Criteria for DMA Concept

Primary Criteria:

- ➢ Hydraulic Isolation
- ➢ DMA Size : Neither Too Big Nor Too Small
- ➢ At Least One Source/PTW
- ➢ At Least One External Connection for Emergencies
- ➢ All Connections and Sources must be metered.

Secondary Criteria:

- ➢ Consideration of Well Defined Roads
- ➢ Administrative Boundaries
- ➢ Land Use and Housing Pattern
- ➢ Future Developments

DMA Features

Most of the distribution pipes within a DMA are to be rehabilitated with new HDPE pipe and the distribution network of all DMAs will be designed for future. Also the network is designed for surface water scenario, all the zones are needed to be metered and there will be no illegal connection. In distribution pipe minimum pressure to be ensured within DMA at all-time is above 10m. Satisfying minimum pressure, surplus water of a DMA (if any) will be delivered to the transmission line or to nearby DMAs through inter-DMA pipes where exists negative water balance. The supply source will eventually shift from groundwater to surface-water.

DMA operations:

Major Control Parameters are

- Pump Operation, PSV and/or PRV Operation at Bulk Meter Chamber
- Bulk Meter ,Valve Efficiency and Domestic Meter and HC Characteristics

METHODOLOGY

Location of the Study Area

DMA ID	Area (sqr m from Shp)	Area (sqr m from Access)	% non-network Pop 2010	% non-network pop 2030	Population 2010	Population 2030	Demand 2010 (L/s)	Demand 2013 (L/s)	Demand 2030 (L/s)
315	200858	192634	15%	5%	12,808	21,851	42	47	55

Fig 1: The study area of DMA 315 of Dhanmondi area

Total demands will not be revised, but their allocation by junctions will be revised due to increased changes in geometry. Demands can be allocated to the junctions using various methods. Many methods are based on observed data, such as billing meters, known flows in certain points of the network, by land use or by population settled. Since all of this data are not available, other methods that compile geometry of network will be used. One of these is "proportional distribution by area". This method divides the lump-sum flow among the service polygons based upon one of two attributes of the service polygons-the area (in this case a total area). The greater the percentage of the lump-sum area that a service polygon contains, the greater is the percentage of total flow that will be assigned to that service polygon.

Other method available which can take into account allocation of residents both horizontally and vertically is billing meters aggregation. Billing Meter aggregation is the technique of assigning all meters within a service polygon to a specified demand node. Service polygons define the service area for each of the demand nodes. Since location and number of existing meters are not known, factious or virtual meters will be assigned to the buildings. Within every building the number of meters in a form of node which will be assigned is equal to the number of floors. This will be incorporated in GIS and imported in model as background layer together with polygon service area which will be generated using WaterGems "Thiessen polygon" tool. Flow distribution has allocated evenly for every node (meter).

Figure 2: Demand allocation method

On the figure 2 above, on the left side is the service polygon that closes larger area, and on the right side is the service polygon that closes smaller area. Dots represent number of the floors within buildings. On the left side total number of residences is much smaller comparing to the total number of residences on the right side. Summarizing what is said about both methods, proportional distribution by area is the method which counts that inside polygon population of consumers has evenly distributed, and billing meters aggregation is the method which counts that inside polygon population of consumers are concentrated distributed.

With billing meters method it will be taken horizontal distribution as well as vertical by floors. What can be the case is, within service polygon in some parts of DMA, it is possible that congregation of couple of tall buildings can give larger junction demand for the present scenario; while for the same service polygon distribution by proportional area can provide lesser junction demand. In this reason it is a bit safe to calculate for present scenario using billing meters aggregation as this method provides more realistic distribution of demands for current state. Also a small calculation has been done, for present scenario using both methods and results are showed in chart below. Results marked with blue columns are BMA (billing meter aggregation) and red are PDA (distribution by proportional area. It can be seen that pressures within network are larger for the first method.

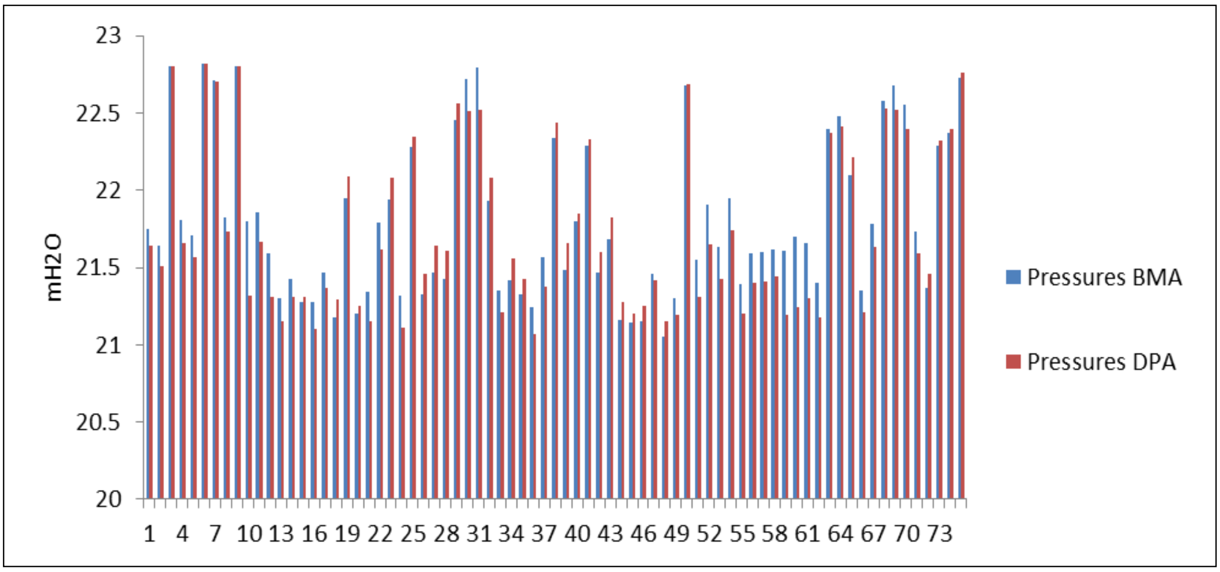

Figure 3: Comparison in pressures between two methods for demand allocation

We are on safe side if the calculation and dimensioning of the network would be done with methods that show higher pressures. Therefore for the present scenario billing meters aggregation will be used. For the future scenario, due to unpredictable conditions in Dhaka and rapidly increasing of population, it can be taken that inside service polygon population will be evenly distributed, and the second method, distribution by proportional area will be used. Assigning demands has been done using Load Builder tool. The Load Builder wizard assists you in the creation of a new load build template by stepping you through the procedure of creating a new load build template. Depending on the load build method you choose, the specific steps presented in the wizard will vary. Service area for every single junction is generated using Water GEMS tool to create Thiessen polygons.

RECOMMENDATIONS

Distribution Networks Model has been prepared & analysed for the different scenarios i.e. In order to distribute the total demands of DMA 315 to the network, first of all the Theissen Polygon Boundary has been prepared and the total demands have been distributed through the Thiessen Polygon generated by using the software. Considering the requirements as the Model, networks pipe sizing have been improved by giving several trials. In some cases the existing pipes have been increased to larger diameter and in many cases reduced to smaller diameter on the basis of hydraulic design in order to achieve the well balanced hydraulic model maintaining the Guaranteed Minimum System Pressure of 10 m (1 bar). Finally the design model has been tested by doing some sensitivity analysis e.g. if one PTW is shut-down; if one surface water source becomes inactive; if the peak demands exceeds the estimated peak factor of 1.25 and goes up to 1.80; if one primary &/or secondary distribution main is damaged suddenly and overall during the operation and maintenance of the system a considerable segment lengths has to be closed by operating the isolating valve, etc.

CONCLUSION

The network model of DMA 315 has been designed to fulfil all the requirements according to the Model as noted below:

- All nodal elevations are set at 1 m below the road surface.
- Total peak demand for 2013 has been assumed to satisfy for 43.95 L/sec, and peak demand for 2030 has been assumed to satisfy 55 L/sec.

- Head loss gradient in pipeline satisfy the value of 5 m/Km, with some exception for smaller lengths.
- Minimum cover of pipeline is about 1 m in the main road and about 0.6 m along the foot path.
- Minimum system pressure has been achieved 10 m (1 bar) at any place of the networks. Technical system loss should not more than 15%, with some exception in scenarios that are not primary designed to be conducted.
- Water meter installed in the PTW should be checked regularly and note if the production is less than expected.
- Model should be calibrated and updated in future.

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NUMERICAL APPROACHES IN SOLVING 1D FLOW INTERACTIONS BETWEEN SEA AND INLAND WATER

M. Salauddin[1*], R.M.R.M. Jayathilaka[2], C.A. Rey Velasco[3] & I. Popescu[3]

[1] Department of Civil Engineering, Chittagong University of Engineering & Technology, Chittagong-4349, Bangladesh.
[2] National Aquatic Resources Research and Development Agency, Colombo, Sri Lanka.
[3] UNESCO-IHE Institute for water Education, Delft, Netherlands.
**Corresponding Author, <mdsalauddin24@gmail.com>*

ABSTRACT

In the field of water science and structural engineering, designing requires description of phenomenon in terms of differential equation which can be Ordinary differential equation (ODEs) and partially differential equation (PDEs). However, in reality there are almost no exact solutions for these equations. Thus, the numerical methods are necessary to get approximate solutions of these types of differential equations. There are several kinds of numerical approaches that can be applied for the solution of numerical problems. Present paper, presents a critical comparison of the suitability of three selected numerical approaches; Euler first order; Range-Kutta Fourth order; and Adams Bashford, which are the most suitable for a specific selected case study and are of explicit nature. The selected problem to demonstrate the use of numerical schemes and their strength in solving water related problems is the one dimensional flow interaction between an inland reservoir and an ocean, that are connected through a canal. The non-linearity behaviors of the water level in the ocean-reservoir system make it hard to obtain the analytical solution by using classical methods. The purpose of this case study is to identify the best numerical approach among three selected approaches for solving one dimensional flow interactions between ocean and the reservoir. The outputs of the three selected numerical schemes are analyzed and compared with each other. The analysis shows that the fourth order Range-Kutta method gives better approximation compared to the first order Euler and Adams Bashford method. The accuracy and sensitivity of the first order Euler method can however be improved by choosing smaller time interval.

Keywords: numerical solutions, hydraulic modelling, differential equation.

INTRODUCTION

Most of the ODEs and PDEs that describes water movement in streams, aquifers, pipe networks, are very often complex (Popescu I, 2013). Thus, it is not always possible to find the analytical solution. Numerical methods are used to provide approximate solutions to the ODEs and PDEs. Numerical approaches can handle large systems of equations, non-linearity, complicated geometries that are common in engineering practice and for which analytical solution are not known. Although at present many numerical approaches are available to find approximate solution no numerical method is completely trouble free in all situations. No numerical method is optimal for all types of equation. Therefore, it is necessary to find the best numerical approaches for particular solution that means for particular case study.

CASE STUDY DESCRIPTION

In order to demonstrate the concepts presented in the Introduction section above, an ideal example case study is selected (Figure 1). The case study requires that a channel is designed to be constructed, linking a lake to a large water body (i.e. sea or ocean). The lake depth is defined as 10 m, with a surface area of 500.000 m2. The channel connecting the lake to the ocean has a Manning's n of 0.025 and a length of 2km. the depth of the channel is not known, needs to be designed, however the width has been selected, based on topography, to be 10 m. The ocean tide is diurnal (i.e. the period T is 24.8 hours) with 0.8 m amplitude.

The lake has inflows coming from a river. The inflow to the lake is available to the designers (Figure 2). The inflow is defined for 25 days (150 hours). At t = 0 hours, the water level in both the ocean and lake are at the same level, i.e. horizontal and the ocean starts rising (mean sea level has an elevation of 0 m).

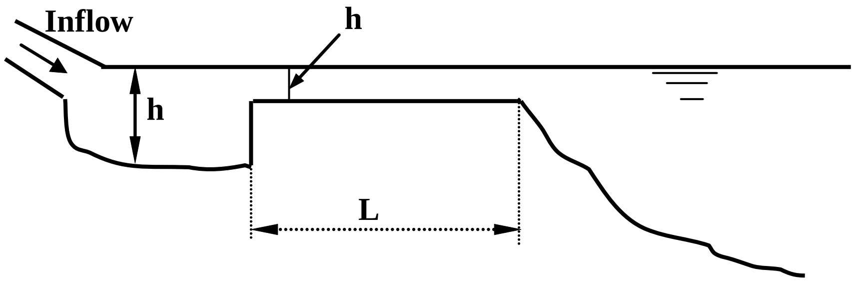

Figure.1: Lake-ocean system

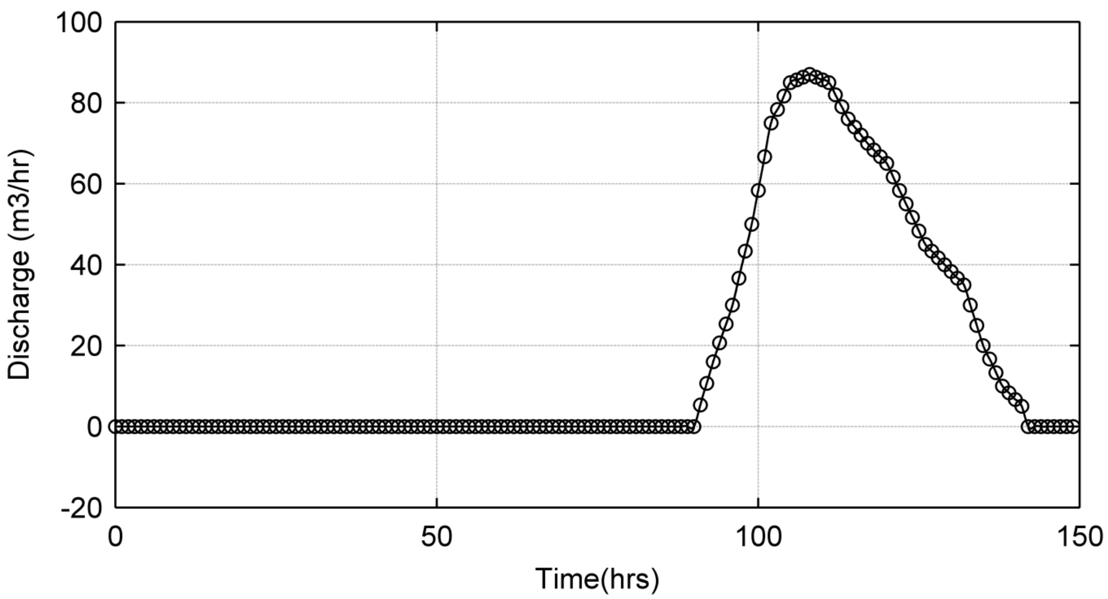

Figure.2: Inflow hydrograph

METHODOLOGY

Problem described in section 2, can be mathematically modelled using three consecutive equations; a continuity equation for the lake water level variation; a quasi-steady flow in the channel, governed by Manning's equation; and a tidal movement of the ocean, as follows:

Lake water level,

$$\mathbf{S_R \frac{dh}{dt} = Inflow - Outflow} \qquad \text{Equation 5}$$

In Which, h, S_R and Inflow are defined in Figure.1

- Flow in the channel, $Q = \frac{1}{n} A_c R_c^{2/3} \sqrt{I_f}$

Where, $\mathbf{R_c}$ = wetted perimeter, $\mathbf{A_c}$ = Area of channel Equation 6

- Ocean water level, $h_{ocean} = h_o + ASin(wt - \varphi)$ Equation 7

In Which, A=Amplitude, w=radian frequency and φ=phase shift

Present paper proposes to utilize three numerical schemes to investigate which one would be more appropriate to be used for the design of the channel width.
The three proposed numerical schemes are; Euler first order approach; Forth Order Runge-Kutta Method and Adams Bashford method

The three equations describing the phenomena are discretised according to the three equations as follows:

a. By Euler Method (Butcher, 2003):
The water level in the lake can be calculated by using an explicit scheme:

$h_r^{n+1} = h_r^n + (I^n - Q^n) * \frac{\Delta t}{Ar}$ Equation 8

Water level of the channel can be computed from water level of ocean and reservoir using following relationship,

$$h_c = \frac{1}{2}(h_{ocean} + h_r)$$

b. By Forth Order Runge-Kutta Method (Chapra C. and Canale P. 2010):
The approximation of reservoir (lake) water level by Four point Runge-Kutta Method (RK4)

$h_r^{n+1} = h_r^n + \frac{1}{6}(K_1 + 2K_2 + 2K_3 + K_4) \times \Delta t$ Equation 9

and $t^{n+1} = t^n + \Delta t$ where, Δt =size of the interval

$K_1 = f(t^n, h_r^n)$

$K_2 = f(t^n + \frac{\Delta t}{2}, h_r^n + \frac{\Delta t}{2} K_1)$

$K_3 = f(t^n + \frac{\Delta t}{2}, h_r^n + \frac{\Delta t}{2} K_2)$

$K_4 = f(t^n + \Delta t, h_r^n + K_3)$

c. By Adams bashford:
It is the multi steps explicit method, based on the numerical quadrature or numerical integration formulae. This method has the same order of accuracy as the implicit method (Isaacsosn E, 1994).
The expression used is a four step method, where, the first steps time needs a input of initial values to calculate the formulae. Thus, the initial value problem provides only one value, the other initial value guess, can be solve from one time step (h^{n+1}), computed by Euler's method or Runge Kutta.
The expression of this method is:

$$h_{n+4} = h_{n+3} + \frac{55}{24} dt * f(h_{n+3}, t_{n+3}) - \frac{59}{24} dt * f(h_{n+2}, t_{n+2}) + \frac{37}{24} dt * f(h_{n+1}, t_{n+1}) - \frac{3}{8} dt * f(h_n, t_n)$$

Equation10

Where the solution of the differential equation using Adams Bashford, it is:

$$h_{n+4} = h_{n+3} + \frac{55}{24} dt * \left(\frac{I^{n+3} - O^{n+3}}{S_r}\right) - \frac{59}{24} dt * \left(\frac{I^{n+2} - O^{n+2}}{S_r}\right) + \frac{37}{24} dt * \left(\frac{I^{n+1} - O^{n+1}}{S_r}\right) - \frac{3}{8} dt * \left(\frac{I^n - O^n}{S_r}\right)$$

Equation**11**

Numerical solutions were coded in MATLAB and tests for different time steps were selected. Results and discussion are presented in the following section of the paper.

RESULTS AND DISCUSSION

With the fixed surface area of the lake of 500,000 m2 and for the time step Δt=1hr, outflow in the channel and the reservoir water level have been found by using above mentioned three different approaches. The Figure 3 compares the inflow verses outflow found in three different approaches. The Figure 4 shows water levels at channel by three different schemes for the time step of Δt=1hr.

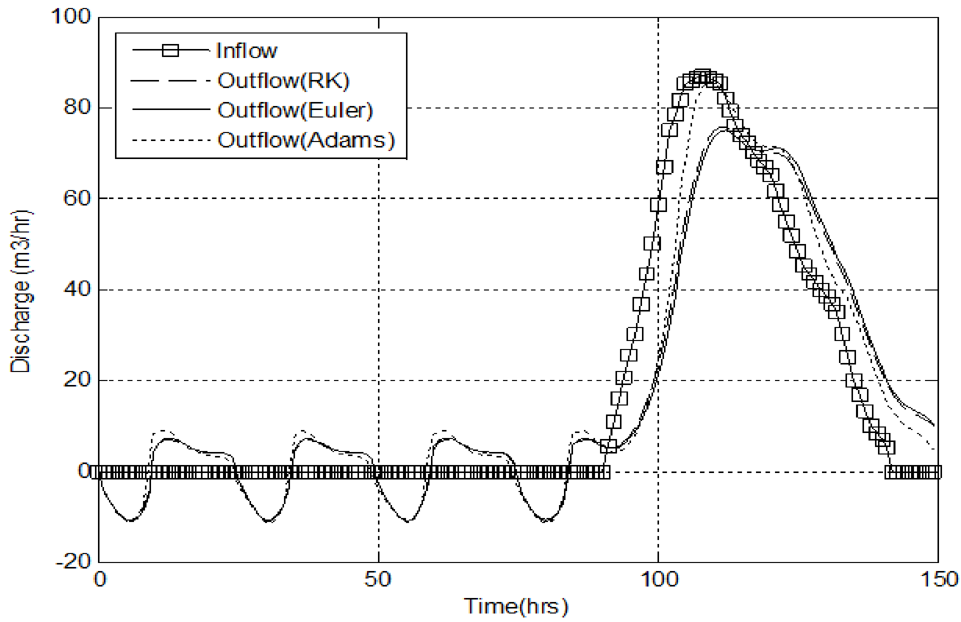

Figure.3: Comparison of outflow discharge by different schemes

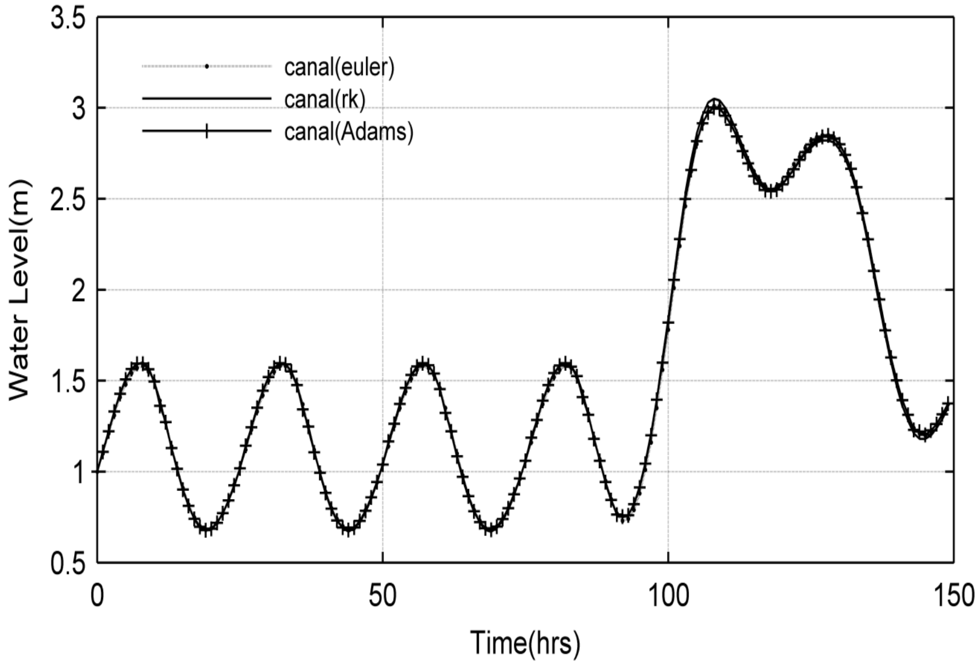

Figure.4: Comparison of water levels in canal and reservoir

Euler method is a first order method, Runge-Kutta method used in this modelling to approximate the water levels is a fourth order method and Adams method is a four step method. Thus, theoretically fourth order Runge-Kutta method and four steps Adams method should give better approximation than first order Euler method.From the outputs observed in Figure 3 and 4 are the outflow and the water levels in the canal at Δt=1hr, where three methods look almost exactly the same when the three lines are placed on the same line there is no bigger difference as expected.

The Figure 5 shows outflow discharge of channel by Adams Bashford method for different time intervals. In the sensitivity analysis of Adams method it is seen that outflow discharge reaches more unrealistic sharp pick with increasing time step whereas RK4 and Euler show more realistic results over different time intervals.

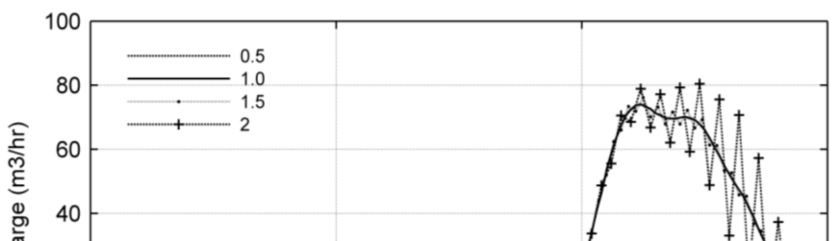

Figure.5: Sensitivity analysis of Adams Bashfords method with different Δt

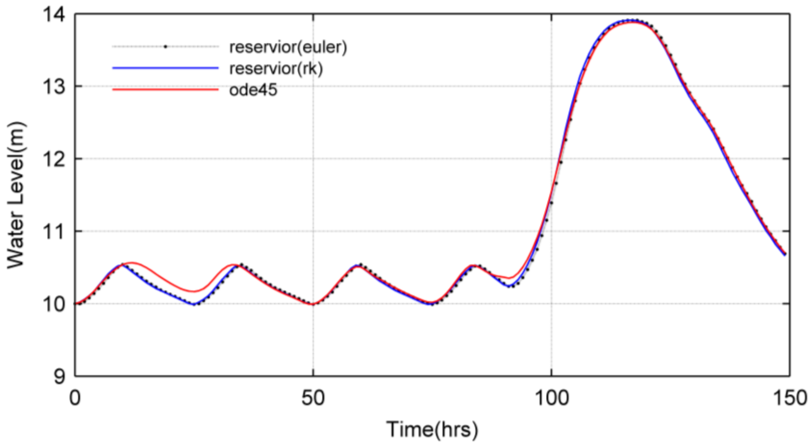

Figure.6: Comparison of water levels in reservoir with ode 45

In order to compare these methods MATLAB code has been generated. Using this code we were able to compare Euler, RK4, as well as the MATLAB solution to D.E.'s called ode45. The accuracy of the RK4 is just slightly better because of its programmed ability to match the curve of the D.E. between the set intervals as shown in the Figure 6. Also it is found that the root mean square error of RK4 is 0.072 whereas it is 0.078 for Euler method.
Thus, in the determination of channel width fourth order Runge-Kutta method (RK4) has been selected best one among described three approaches.

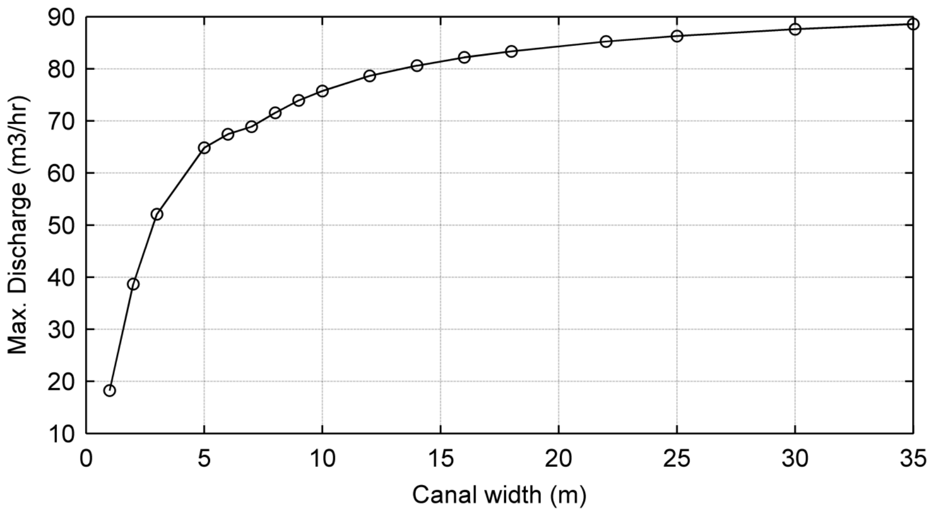

Figure.7: Relationship between outflow and channel width

The width of the channel can be computed from the outflow of channel discharge. The Figure 7 shows the channel width for different outflow discharge of the channel. The prescribed relationship between discharge (outflow) of the channel and channel width can be used in the design of a channel. For a particular outflow capacity of the channel the required value of channel width can be obtained from this relationship. After determination of the channel width for a particular outflow, the maximum water level also can be estimated from the relationship between maximum water level and width of the channel.

CONCLUSION

The numerical methods are useful to solve water science engineering problems and get a close approximation of the phenomena, comparing the sensibility of it. Based on, the sensibility analysis for one dimensional flow interactions between ocean and inland water that is reservoir, canal; the best numerical approach is found Runge Kuta forth order Method among prescribed three. However the accuracy of it depends directly of the time step small would be. Besides, from this research it is concluded the numerical method can be used as design procedure, once the data set is available.

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