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IRRIGATION WATER REQUIREMENT FOR MAIZE CROP CULTIVATION

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Abstract

Agriculture is one of the most important driving factors for the growth of economy of Bangladesh. So, in order to achieve a profitable, sustainable, and growing as well as environment friendly agricultural framework it is a must to ensure the food security for the people of Bangladesh. Alongside it is a huge challenging task for the Government to tackle the food demand of the huge population of Bangladesh. So it is very much important to increase the crop production per unit area to ensure the food security of the people from the limited land area. Maize which is the faster growing and high yielding cereal crop as well as the third most important cereal crop of Bangladesh can be cultivated to meet the demand of cereal crop in the country. In this research work a study was conducted during the dry season (Rabi) at Bangladesh Agricultural Research Institute (BARI), Joydebpur, Gazipur, to assess the irrigation water requirement of maize crop and to observe the impact of tillage methods on water use and maize yield. The outcome of the study suggested that two times (I_2) water application with zero (T_1) tillage, minimum (T_2) and traditional tillage (T_3) practices gives a substantial amount of yield with minimum water application which is a very resourceful finding for maize cultivation. The amount of yield changes from 7.133 ton/ha for I_1, 8.19 ton/ha for I_2 and 8.31 ton/ha for I_3 has been found from the field experiment.

Introduction

Bangladesh is predominately an agricultural country. Food security has been identified as a significant factor by the Government of Bangladesh contributing to its socio economic stabilization as well as development (Kashem et al. 2011). Food security of the huge population of Bangladesh is closely interrelated with agriculture. Moreover, agriculture is straightly interconnected to the concerns like poverty mitigation, raising of standard of living of people and increasing generation of employment. It is, therefore, important to have a profitable, sustainable and environment friendly agricultural system in order to ensure long term food security. Although Bangladesh is about to reach to a status of Middle Income Country by 2021, agriculture is still is the core employer in the country for its people. Around 47.5% of the population is directly involved in agricultural activities. Moreover, livelihood of around 70% of the population is dependent on agriculture in one form or another. The contribution to GDP from the agricultural sector was 12.65 percent in 2013-2014 Fiscal Year (FY). The percentage was 16.33 of GDP for the overall contribution of the broad agricultural sector (BER 2014).

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For achieving the Millennium Development Goals (MDGs) and post MDGs, and turning Bangladesh into a middle income country by 2021, the GDP has to grow at a minimum rate of 7% per year to achieve the goals of MDGs and post MDGs. Agricultural growth rate has to be kept at a constant rate of 4-4.5% per year to achieve this GDP growth rate by keeping pace with the growing population (Miah, 2015). Total factor productivity (TFP) indices capture the effect of improvements in technology as well as investments in rural infrastructures. It has been found from the empirical evidence that TFP of Bangladesh crop agriculture grew at an annual rate of 0.57% during the last few decades i.e. from 1948 to 2008 (Rahman and Salim 2013).

To ensure the food security of the population of this country from the limited land area, production per unit area must be increased with a faster rate of production. Since maize is the third most important cereal crop in Bangladesh as well as a faster growing and high yielding cereal crop can be cultivated to eliminate the shortage of cereal crop in this country. There was an insignificant amount of maize production in the early nineteenth decade in Bangladesh, but during 1997-98, about 2,834 hectares of land were under maize cultivation with a production of 3,000 metric tons (BBS 1999). The area was expanded to 1, 37,000 hectares and the corresponding production was 7, 83,640 metric ton up to 2007 (DAE 2007). Among different agricultural elements of Bangladesh, irrigation is the key input for achieving higher yield of maize. Although a large number of experimental works on irrigation had been carried out for the various crop production but a little amount of experiment on irrigation for maize cultivation with tillage treatments had been done in Bangladesh. Therefore, an attempt has been made to evaluate the effect of irrigation water with suitable tillage practices on the yield of maize in this research paper.

Previous studies

Hassan et al. (2003) monitored the maize and wheat cultivation on raised bed with irrigation. They showed that maize on raised beds consumed less irrigation water in comparison to basins. The water savings of raised beds over basins was ranged from 16% to 83%, with an average value of 32%. There were seasonal variations in irrigation depths because of different rainfall amounts and distributions in each season. The least irrigation water was applied in 2002 and555 mm rainfall was occurred. The number of irrigations applied was sometimes higher in raised beds but the amount of water applied in irrigation was always less than basins. The average amount of water per irrigation was 46 mm for beds and 78 mm for basins. The seasonal differences in total irrigation amount were varied because of the rainfall occurrences and its distribution over each period. Overall irrigation water applied to raise beds was probably the result of reduced evaporation, less wetted area and soil configuration in the raised beds, and over- irrigation in the basins. From the above experiment, it was found that in the raised bed irrigation for maize cultivation saved substantial amount of water in comparison to basin bed irrigation system for maize cultivation.

Islam et al. (2006) conducted an experiment at Regional Agricultural Research Station (RARS), Barisal, during Rabi season for the period of 2006-07 and 2007-08 to study the effect of different moisture regime and tillage on soil physical properties and its impact on the yield of wheat. Twenty treatments combinations comprising four tillage practices namely zero tillage (tillage by country plough having 5 to 6 cm depth), tillage by power tiller (10 to 12 cm depth), tillage by chisel (20 to 25 cm depth) and five levels of irrigation on the basis of IW/CPE ratio of 0.4, 0.6, 0.8, 1.0 and rain fed condition were tested in a split plot design with three replications. Irrigation and tillage had significantly influenced the yield and yield contributing characters of wheat. The highest yields were recorded as 4.5 t ha^{-1} and 4.6 t ha^{-1} with tillage (T_3) and irrigation (I_3), respectively. In case of the interaction effects of tillage and irrigation, the highest yields were recorded as 4.8 t ha^{-1} and 4.85 t ha^{-1} during 2006-07 and 2007-08, respectively from $T_3 I_3$. It was observed that the influence of tillage and/or irrigation had no significant effect on soil physical parameters during two years of conducting the experiment.

Methodology

A field experiment was conducted in the central research station of Bangladesh Agricultural Research Institute, Joydebpur, Gazipur during 2010-2011 to assess the water requirement for maize crop with various tillage practices mainly concerning to conservation of water for agricultural use. The experiment was set up in a Split Block Design (SBD) for tillage with a split plots arrangement of nine treatment combinations comprising of three replications. The unit plot size was 3m x 4m. Using the principle of ‘randomization’, each experimental plot was allocated a treatment such that a particular treatment did appear not more than once in a particular block considered in any direction. The layout plan of the experimental plots is shown in Figure1. Tillage practices have been assigned in the main plot and irrigations have been applied in the sub plots intensively since water saving is the main concern.

Treatment combination comprises of three methods of tillage T_1 (zero tillage or no tillage), T_2 (minimum tillage) and T_3 (traditional tillage practices). Also thee irrigation treatments which have been applied based on growth stages. BARI butta -6 seed were planted on 08 November, 2010. The maize was harvested on 06 April, 2011. The yield contributing characters were analyzed after harvesting. The yield contributing characters are plant height, grain per cob, 100 grains weight, numbers of plant per plot, line of grain per cob and number of cob per plant. The yield of maize per hectare was determined after threshing the maize.

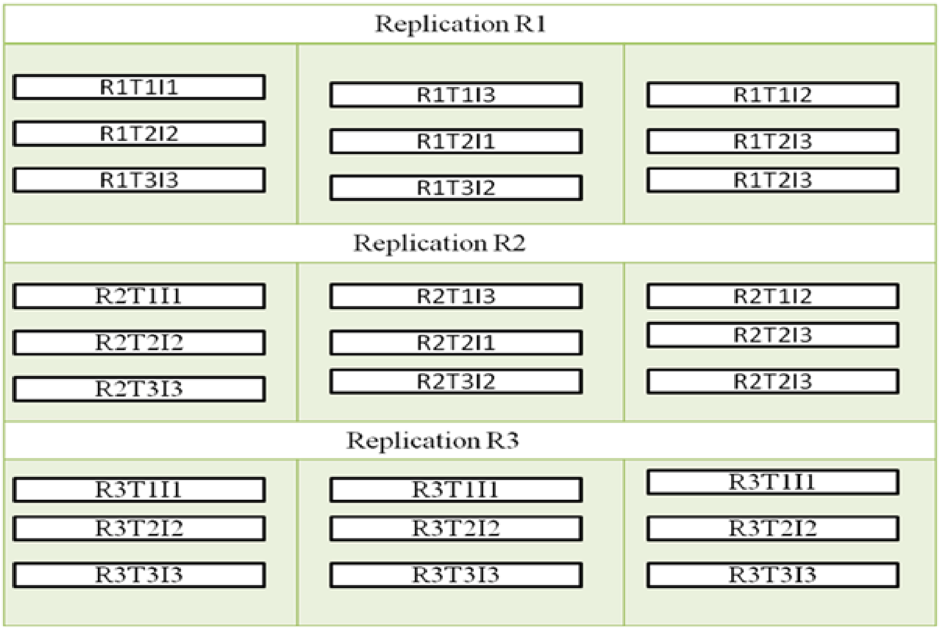


Figure 1. Layout of experiment

Design: SBD — No. of Treatments: 09
No. of Replication: 03 — No. of Plots: 27
Size of Each Plot: 3m × 4m
[N.B: Drawings are not done in scale]

Land preparation and application of fertilizer

The land preparation was started one week prior to maize seeds sowing. At first, the selected land was flooded (02 cm) by applying water in sufficient amount to soften the soil and treated by using power tiller to facilitate tilling. When the field was tilled and maize seeds were sown; fertilizer was mixed properly. The dosages of fertilizer were applied according to FRG (Fertilizer Recommendation Guide) - 2005, Bangladesh Agricultural Research Council (BARC) recommendations. A sufficient amount of water (02 cm) was applied uniformly over the whole experimental field for the survival of maize seedling after seven days of sowing.

Irrigation water application

Volumetric method was used to determine the depth of irrigation water. Three types of irrigation treatments were applied based on growth stages depending on the root zone depth at different stages.

Three level of irrigation treatments were:
I_1 = only one time irrigation was applied at 25 days after sowing.

I_2 = Two times irrigation were applied at 25days after sowing and 50 days after sowing.
I_3 = Three times irrigation were applied at 25 days after sowing, 50 days after sowing and 85 days after sowing.

Irrigation water was applied according to BARI (2005) rules.
Depth of irrigation water required for soil to reach the field capacity was determined by the equation (1).

$$d = \frac{FC\% - MCi\%}{100} \times A_s \times D \qquad \text{eq. (1)}$$

Where,
d = Depth of water to be applied, cm; FC = Field capacity of the soil in%; MC_i = Moisture content of the soil at the time of irrigation in %; A_s = Apparent specific gravity; *D* = Root zone depth of maize crop, cm.

The application of irrigation water was carried out with bucket by measuring the water in volume (liter). The depth of water calculated by equation (1) was converted into liter (volume) by unit conversion.

No runoff and deep percolation were allowed since irrigation water was applied to reach the soil moisture up to field capacity. Weed was controlled manually with BARI invented weeder from each plot for two times during experiment. Soil moisture was measured before and after application of irrigation water by gravimetric method in each replication.

Tillage practices

Soil tillage is the manipulation of soil which is generally considered as necessary to obtain optimum growth conditions for all crops including maize. The growth of crops concerns about agricultural sustainability, environmental pollution, soil erosion and also tillage practice with proper management strategies and proper selection which can ensure optimum water use and reduction of runoff losses.

Three methods of tillage practices were used in the experiment. They were:

T_1 = Zero tillage/ no tillage (strip tillage) i.e. no disturbances of soil, only sowing operation which was carried out by power tiller operated inclined plate planter.

T_2 = Minimum tillage i.e. minimum tillage and sowing operation. Both were carried out simultaneously by power tiller operated inclined plate planter.

T_3 = Traditional tillage operation (farmers practices) i.e. three to four times tillage with power tiller and sowing operation. Sowing operation was carried out manually by hands.

Effects of water application on maize yield contributing characters

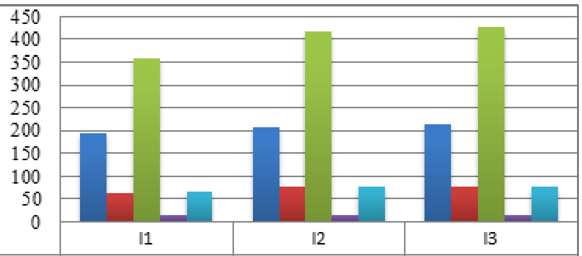
The effects of water application on the yield and yield contributing characters of maize have been shown in the Table 1 and Figure 2. The yield contributing characters of maize were significantly affected with the variations of water application. The lowest yield (7.133 ton/ha) was found in lowest amount of water application treatment (I_1). The maximum yield of maize was found 8.310 ton/ha in maximum amount of water application treatment (I_3). In I_2 treatment, the maize yield was recorded as 8.190 ton/ha that was the nearest value of I_3 irrigation treatment. The yield was increased rapidly with the increase of water application up to a certain level and then the yield was not sufficiently increased with the increased application of water.

Maximum plant height (214.7 cm) was found for I_3 irrigation treatment and minimum plant height (193.7 cm) was found in I_1 irrigation treatment. Number of plants per plot varies with the variation of water application up to a certain level and afterwards it does not depend on water application. Plants per plot were minimum (66) in one water application treatment (I_1) and the numbers were 75.33 and 75.00 in two (I_2) and three times (I_3) of water application treatment respectively. Maximum number of plants depends only on the availability of water, not on maximum number of water application. The maximum number of cob per plot was found in I_2 and I_3 treatments as 75.33 pieces and 77.67 pieces respectively and the lowest number of cobs 62.33 pieces was found in I_1 irrigation treatment shown in Table 1. Number of cobs was increased with the increase of water application. Maximum number of cobs does not indicate the higher yield because more than one cob in a single plant is unhealthy and it decreases yield. So, maximum water application does not give the maximum yield. Therefore, the optimum water application is required for maximum yield.

Maximum numbers of grains/cob were found in maximum water application that indicates little size of grains. So for greater yield and maximum benefit the optimum water application is essential. Another yield contributing character (100-grain weight) was found highest (15.73 gm) in I_3 irrigation practices which was very much similar with I_2 irrigation practices (15.23 gm) and lowest (14.93 gm) in I_1 irrigation practices.

Table1. Effect of water application on the yield and yield contributing characters of maize

Treatment	**Plant height (cm)**	**Plants/ plot**	**Cobs/ plot**	**Grain/cob**	**100-grain weight (gm)**	**Yield (ton ha^{-1})**
I_1	193.7	66.33	62.33	358.0	14.93	7.133
I_2	207.3	75.33	75.33	416.7	15.23	8.190
I_3	214.7	75.00	77.67	426.0	15.73	8.310



	I1	I2	I3
Plant height (cm)	193.7	207.3	214.7
No. of cobs per plot	62.33	75.33	77.67
grains per cob	358	416.7	426
100-grains weight (gm)	14.93	15.23	15.73
Plant population per plot	66.33	75.33	75

Figure 2. Effect of different amount of irrigation water on maize yields parameters

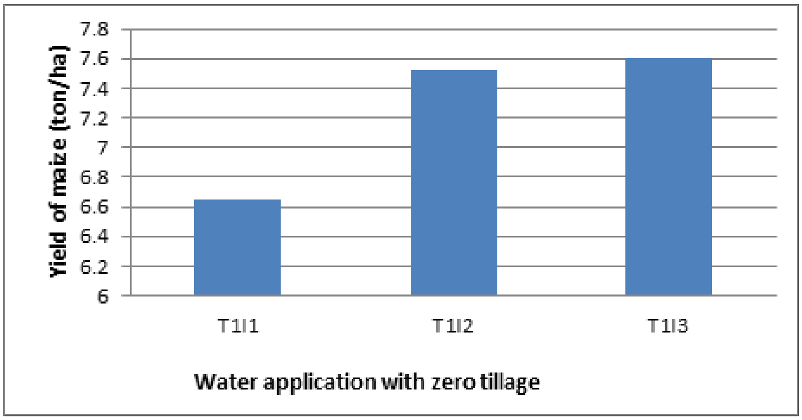


Figure 3. Effect of water with zero tillage on maize yield in the experiment

In zero tillage practice, three types of water applications were done. The three types of water applications were i) I_1 = one irrigation, ii) I_2 = two irrigation and iii) I_3 = three irrigation. The effects of irrigation on maize yields with zero tillage are presented in Figure 3. The maize yields were 6.65 t ha^{-1}, 7.52 t ha^{-1} and 7.6 t ha^{-1} for I_1, I_2 and I_3 irrigation treatments respectively. Maize yield of I_2 irrigation was statistically similar with I_3 irrigation except I_1 irrigation. Yield was increased with the increase of water application. One irrigation treatment practice with zero tillage is not suitable because the yield is lowest in this treatment combination. Two and three- times water application with zero tillage has nearly the similar effects on the maize yield.

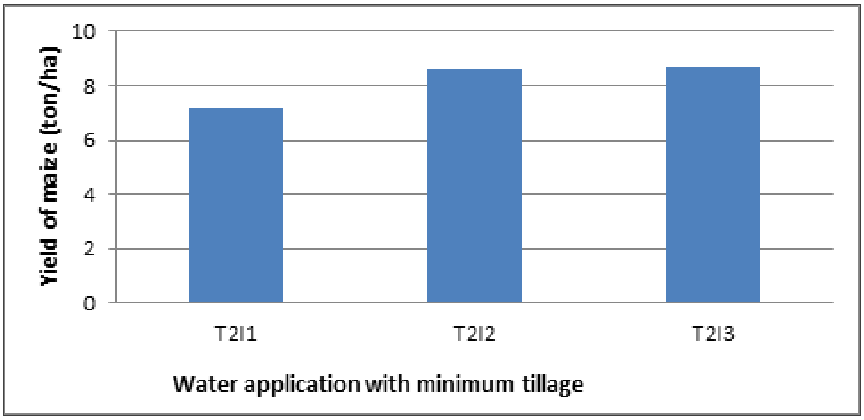


Figure 4. Effect of water with minimum tillage on maize yield in the experiment

In minimum tillage practice, the three level of irrigation water applied. The maize yield were 7.20 t ha^{-1}, 8.60 t ha^{-1} and 8.65 t ha^{-1} for I_1, I_2 and I_3 irrigation treatments respectively (Figure 4). Maize yield of I_2 irrigation was statistically very similar with I_3 irrigation except I_1 irrigation. A minor yield variation was found in two and three times water application. Minimum tillage with two times and three times water application gave the same results in yields and yield contributing characters of maize. The lowest amount of maize yield was recorded in minimum tillage with one time water application.

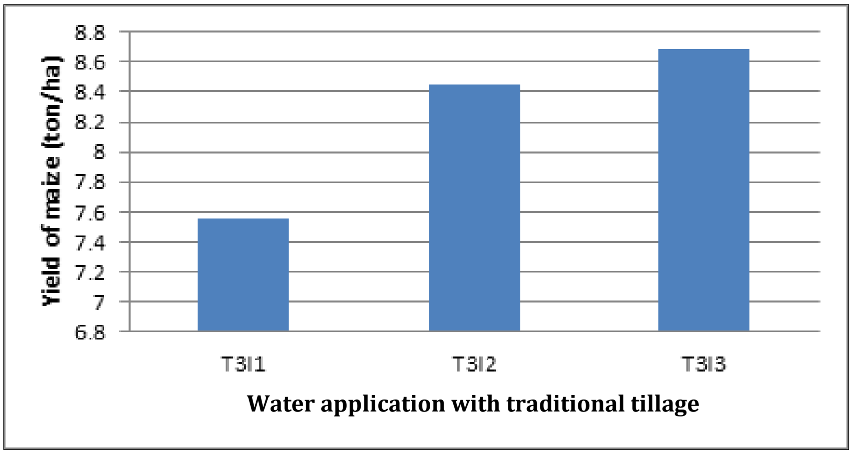


Figure 5. Effect of water with traditional tillage on maize yield in the experiment

Traditional tillage means five times tillage of soil and then sowing by hand which is a general practice of the farmers in our country. The effects of traditional tillage with one, two and three times water application for maize cultivation was observed. Traditional tillage with one time water application gave the lowest yield (7.55 t ha^{-1}) and the yields were 8.45 t ha^{-1} and 8.68 t ha^{-1} in two (I_2) and three times (I_3) water application with traditional tillage (Figure 5). Two (I_2) and three (I_3) times water application with traditional tillage have given the very similar results in yield and yield contributing characters. The yield was increased rapidly from one time water application to two times water application. The yield did not increase substantially from two times (I_2) water application to three times (I_3) water application. This result indicates the more water application is not beneficial for maize cultivation. One time (I_1) water application is also not sufficient water application for maize production because the yield is not substantial in one time water application.

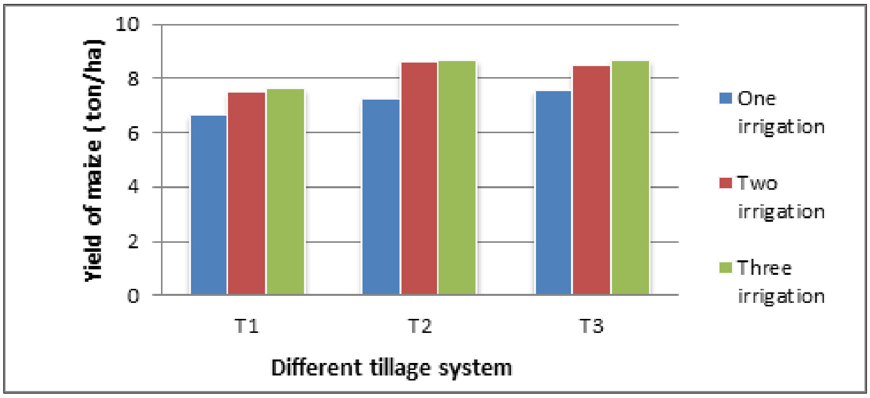


Figure 6. Effect of irrigation on maize yields with different tillage practices

The effects of tillage system with three levels of water application on maize yield were observed. The three types of tillage system were zero tillage (T_1), minimum tillage (T_2) and Traditional tillage (T_3) and three levels of water application were one time (I_1) water application, two times (I_2) water application and three times (I_3) water application. In every tillage system, the lowest yield was found in one time water application and the highest yield was found in three times water application. The yield for two times water application was very much similar with three times water application in each type of tillage system. The yield of maize was varied with the different amount of water application (Figure 6).

Conclusions

The rationale use of water for agriculture is a dire need at present days because the useable water resource is diminishing very rapidly. Therefore, the scientific application of water in agriculture sector is important for saving the water resources and for minimizing of cost of agricultural production. The cultivation of high yielding grain crops like maize is very important to meet the need of cereal grains for the people. From this research paper, it can be revealed that irrigation water has a significant effect on maize yield and yield contributing characters. Plant height, plant population, cobs per plot, grains per cob, 100-grain weight and yield has been affected significantly by the variation of water application. Unsatisfactory and poor results of maize yield contributing characters were found for one time (I_1) water application. The higher values of maize yield contributing characters were found for three times (I_3) water application. Satisfactory results of maize yield contributing characters were also found for two times (I_2) water application. I_3 (three times water application) irrigation was sufficient for higher yield maize cultivation but a huge amount of water was required in this practices. Statistically same significance for maize yield contributing characters was found in two times (I_2) and three times (I_3) water application. Among the three irrigation practices (I_1, I_2 and I_3) I_2 irrigation practice (irrigation applied two times, 25 days after sowing and 50 days after sowing) was found as preferable water application practice for maize cultivation in dry season of Bangladesh. Tillage has significant effects on maize yield and maize yield contributing characters. T_1 (zero) tillage is not preferable because lowest amount of yield has been found in zero tillage treatment. Statistically same effect was found on maize yield for minimum tillage (T_2) practice and traditional tillage (T_3) practice. Two times (I_2) water application with zero (T_1), minimum (T_2) and traditional tillage (T_3) practices gave a substantial amount of yield with minimum water application.

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PHYSICAL MODELLING FOR THE IMPROVEMENT OF BARISAL HARBOR AREA

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Abstract

A physical fixed bed model for the improvement of Barisal harbor area was carried out jointly WRE, BUET with RRI at RRI to investigate the flow pattern and velocity distribution at different depths and locations at the study reach of Kirtonkhola river under low, medium and high flow conditions. The objective of the study was to find out probable solution of erosion and siltation problem in the study area. Five test runs were conducted under these three flow conditions. The model was distorted and scaled on the basis of Froude's model law with horizontal scale ratio L_r = 1:200, vertical scale ratio H_r = 1:50, time scale ratio T_r = 1:28.28, velocity scale ratio V_r = 1:7.07 and discharge scale ratio Q_r = 1:70711.The model investigation showed that the velocities at medium flow condition were higher than low and high flow condition. Therefore the severity is governed mainly by the medium flow condition. The bank erosion limit was assumed to be 15 m and the velocity was at 0.8d depth for vulnerable to erosion. The bank materials were silt (compact), the critical velocity (non-scouring) at bed is 1.0 m/s for straight channel. For bank slope and flow curvature 20% reduction of critical velocity had been assumed. Siltation study was carried out indirectly from the velocity records and sediment size. Permissible velocity was determined from particle size, cohesiveness and depth of flow. The average velocities from model tests were much higher than the permissible velocity. It showed that there is least probability of siltation at Central Storage Depots (CSD) food jetty and Bangladesh Inland Water Transport Authority (BIWTA) workshop jetty location under the existing situation.

Introduction

Barisal is located along the right bank of Kirtankhola river at downstream of the junction with Bukainagar Nala. At Bhasan Char of Barisal district, the Kritankholariver branches off from the right bank of Arial Khan river. Further downstream, it leaves a branch from its left bank called Khairabad River. The Kirtonkhola river is tidal but its water is sweet round the year. Tides of the Kirtankhola river is semi-diurnal. Its period is 12 hours 25 minutes. At upstream of this junction, the river is strongly meandering, while from Barisal towards downstream the river is noticeably straight. A natural loop cut near the confluence of the Barisal river and the Arial khan during the sixties and subsequent river adjustment is an indication of the fact that the river was still in active process of development. The width of the river between the banks varies from a maximum of 800 m (approx.) near Char Upen to a minimum of 300 m (approx.) near the Launch Ghat location (shown in the Figure 1). Depth variation in the reach was also substantial being nearly 26 m to 9 m in the thalweg. The tidal variation was from about 1.5 m in the dry season to 0.5 m in

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the wet season. The maximum velocity was approximately 2 m/s. The discharge variation was from 3000 m^3/s during low water flow to about 4000 m^3/s for high water flow. Heavy erosion on the right bank of the Barisal river has occurred for a number of years in the area of the BIWTA workshop jetty.

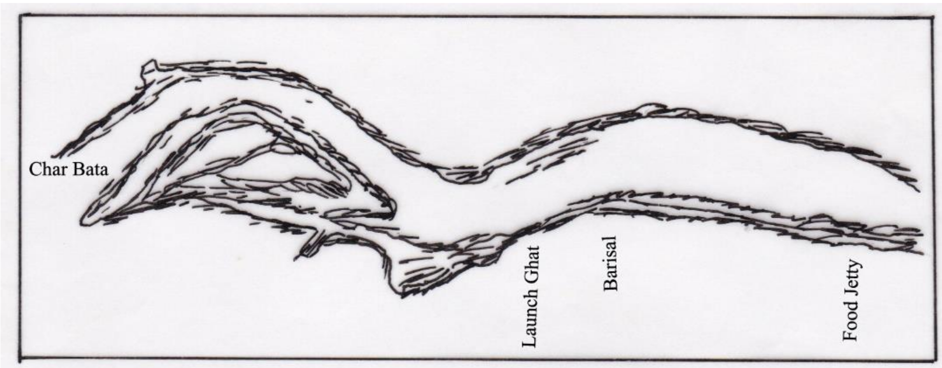

Figure 1. Location sketch of study area

For designing of maritime structures and dredged levels, it is necessary to know the highest and lowest astronomical tides. In terms of amplitudes of tidal constituents, are given by:

Mean sea level ± 1.2 (M_2+ S_2+K_1+O_1)
Where M_2 and $S_2$are semi-diurnal components, $K_{1,}$ O_1 are diurnal components and syndical or semi-diurnal tides prevail.

Mean sea level ± 1.2 (M_2+ S_2+ K_2)
Where M_2, S_2, K_2 are semi-diurnal components and diurnal tide is small.
Sir George Darwin suggested Indian Spring Low Water as chart datum for Indian Waters. It is given by the formula.

Indian Spring Low Water = Mean Sea level- (M_2+ S_2+K_1+O_1)
(Training Programme 1989)

The controlling parameter is a Froude number based upon the speed at which the projection of the moon on the earth moves over the earth's surface and $\sqrt{\square d}$, where d is the water depth. If the Froude number were less than unity, a dispersive wave would result; If the Froude number were unity, a build–up of the wave to a large amplitude would result; If the Froude number were greater than unity, a much smaller wave would result. Inui (1936) has shown for Froude number greater than unity that not only does a phase shift occur, but that negative force (or negative pressure area, which is large with respect to the water depth) generates a decrease in the water level first, followed by an increase in the water level, rather than just an increase in the water level. Tides have recorded in many areas for a great number of years and they have been analyzed to obtain the amplitude and phase of the tidal components. The ocean's tides have been classified as semidiurnal, mixed, and diurnal. The type that occurs at a particular place depends upon the ratio diurnal/semidiurnal $(K_1+O_1)/\ (M_2+\ S_2)$. When the ratio is of the order of 0.1 the tides are semi-diurnal, when it is about unity they are mixed, but predominantly semidiurnal; when it is about 2, they are mixed but predominantly diurnal; when it is 15 or so, they are diurnal (Defant 1958).

Literature review

Taming tidal rivers is always a strenuous work particularly to engineers. Tidal modeling plays a vital role in the field of river training and related works. Lakhya/Dhaleswari Confluence model study was an example of tidal model study. It was a complicated one because it involved two rivers that interact. The purpose of the model was to give the most favorable route for a channel. The complicated tidal river modeling can be performed by two methods, one control method and more than one control method. To avoid modeling of the whole river with all branches, equivalent basin can be made at the upstream end of the reach of the model. One control means control by downstream water level calibration by velocities at upstream and downstream end of modal area. Tidal models by more than one control are more accurate method. The principle of this model is calibration by water levels adjustment/correction by velocities. In 1972, The Tromsoe model study was performed and it was the first model with more than one tidal control (Kamphuis 1975). This method was used in Calabar River Model in Nigeria, Tromsoe Model in San Francisco, Lakhya/Dhaleswari Confluence model in Bangladesh. In Calabar River Model, the model was calibrated on tidal levels velocities, tidal phase difference was known for longer reach than covered by the model, model phase difference was computed & used, and only small corrections were needed to obtain correct velocities. The Lakhya/Dhaleswari Confluence model was very challenging study because it had three tidal controls, very small slopes, interference between rivers and both reversing flow conditions (dry season) and one–directional flow (floods). It soon became apparent that the correct slope in the model was too small to give stable run conditions. Instability was caused by hydraulic coupling between the control system. To avoid the instability two methods were taken into account, (1) Increase slope/ velocity to alter system response (2) Introduce singular head loss between water level followers and main model. The regulation system consisted of two water level controlling systems. One system controlled the downstream water level and other controlled the upstream water level. The two systems were independent of each other except that Time reference is common and there is a hydraulic connection between the two regulation systems (Einar1986). River Research Institute (RRI) of Bangladesh studied some models by more than one control systems, among them The Doarika tidal model study (Einar1986), Bhola Town Protection model study and 3rdKarnafuly bridge models are significant. In 1986, The Doarika tidal model had been studied by more than one control system with the help of tide generator, Water level followers and water supply control systems (Einar 1986). Another model named as Bhola Town Protection work was studied by RRI in 1995 with more than one control system and in 2005, 3rdKarnafuly bridge model was studied with the help of more than one control systems with manually tide generating system.

Methodology

Study Area

The study area for model study was selected based on heavy erosion on the right bank of the Barisal River. This heavy erosion was occurred for number of years in the area of BIWTA workshop jetty. As a result, one half of the jetty was collapsed and also rendered the other half structurally unsound for useful purposes. A physical (tidal) model of the Barisal River was constructed to include the area of existing erosion and siltation and also to include the probable erosion and siltation area near the food jetty.

Data collection

Water levels and cross-sections were collected by depth soundings. Stage hydrograph was made to get a detailed picture of the tidal influence in the river. These measurements were taken by self-recording instruments (Aanderaa WLR5) at char Bata and food Jetty location but only during certain periods to cover low, medium and high stages. The Prototype discharge information was quite useful for model study. In order to obtain a correct picture of the flow pattern, velocity measurements were recorded throughout the study area. Comparing the trend of river shifting from the available hydrographic charts of 1968, 1970, 1975 and 1982, the 1992 situation had been extrapolated.

Data preparation

Topographical information of river bottom elevations at 19 numbers of sections distributed nearly uniformly throughout the length of the reach had been considered to reproduce the topographical features of the river bottom. Subsequently few more sections at some critical positions were used to reproduce the bed profile more accurately. In this case, the scale of sounding information was 1:10,000. In order to get a detailed picture of the tidal influence in the river (e.g. celerity of the tidal wave, time lag between high and low water etc.) high precision, time synchronized water level measurements were performed by Norwegian Hydrodynamic laboratories (NHL). Analysis of these short-term stage recordings enabled the selection of the pilot signals to be used in model simulation. Pilot signals are representative tidal waves of the prototype used for model simulation. The pilot signals for low water and medium and high water conditions are shown in Figure 2 and Figure 3.

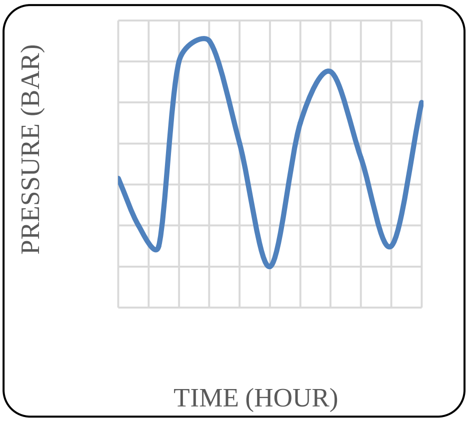

Figure 2. Pilot signal for low water condition

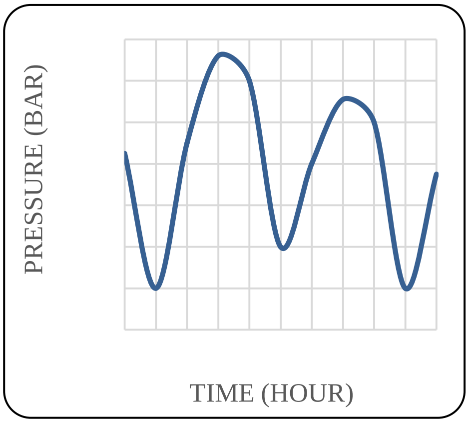

Figure 3. Pilot signal for medium and high water condition

The computation of discharge was made by BIWTA at char Bata and Food Jetty location both in the high and low water seasons. After analysis of the discharge data the following flow conditions had been adopted for the purpose of model run: Low water flow equals to 3000 m^3/s and High water equals to 4000 m^3/s. Velocity measurements were taken at Floating Dock, Char Bata and Food Jetty during the high, medium and low water seasons. Comparing the trend of river shifting from the available hydrographic charts of 1968, 1970, 1975 and 1982, the 1992 situation had been extrapolated. The model study was made to forecast the behavior of the River for this extrapolated situation if no protection works were taken.

Model setup and instrumentation

The model had been constructed to a horizontal scale of 1: 200 and a vertical scale of 1:50 in a 9m X 36m space to accommodate the main river reach including the sumps. Water level followers, that constantly monitor the model water level, were installed at each end of the model.

Layout and instrumentation setup of the model are shown in Figure 4 and Figure 5.

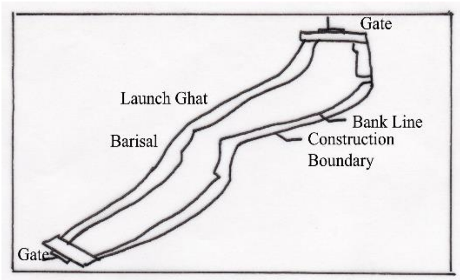

Figure 4. Layout plan of model

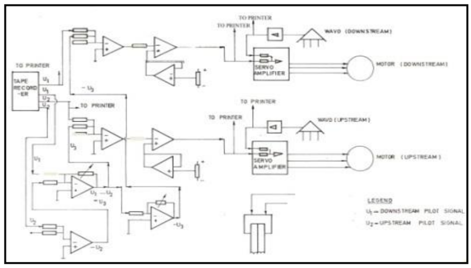

Figure 5. Schematic diagram for operation of instruments

Each end of the model was fitted with adjustment gate connected electromechanically for automatic operation. Hysteresis for the gate operation is shown in Figure 6.

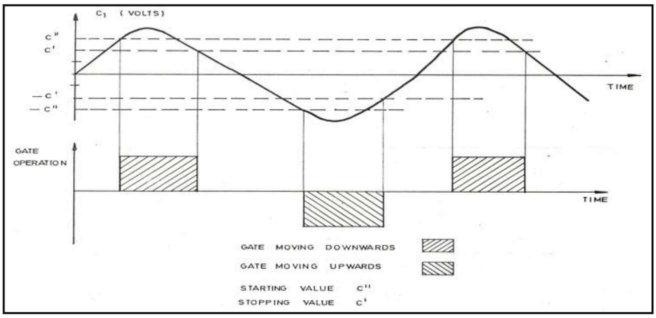

Figure 6. Hysteresis for the gate operation

To reproduce the time dependent water surface slope, it was necessary to control the water levels at each end of the model. For this reason the regulation system consisted of two mutually independent sub systems, each controlling upstream and downstream water levels.

Model design

In the design of the model scale conditions related to the governing process have to be fulfilled in order to obtain complete similitude between model and prototype. The process is flow. For scaling and design of the model Froude's model law $F_r = \frac{v}{\sqrt{gd}}$ and $F_r = 1$ were considered. The design of the model had been made after analyzing the field data. Low, medium and high discharge had been considered for the design, as these discharges were representative of the morphological development.

Model calibration

The calibration of the model was based on the field measurement of water level and current velocities. These current velocities were achieved in the model by introducing a certain mean slope between the upstream and downstream ends of the model. The mean slope was composed of the variable slope from tidal waves superimposed on the constant longitudinal slope due to upland discharge. Calibration of the model had been conducted in existing condition of the river (without any proposed intervention in low, medium and high water). The main focus of the model calibration had been concentrated on the process of flow and sediment transport. The measurement during the calibration includes water levels, bed levels, point velocities, float tracks and discharges. The model was calibrated for three flow conditions viz. low water, medium water and high water. Two pilot signals, one representative of low water flow and while the other for medium and high water flow were used. Since the transformation due to tide propagation from downstream to upstream is negligibly small, the same pilot signal was used for both upstream and downstream with appropriate time lag.

For stabilizing the regulation system and to prevent unwanted disturbance to enter into the model, filters had to be placed at both ends of the model. Pilot signals had to be transformed accordingly to compensate for the head loss through the filters.

Test procedures

Five application tests had been conducted in the model that included present situation (T_1) for low, medium and high discharges. These tests aimed at assessment and recording of velocities and flow pattern. Test (T_2) was conducted present situation with erosion protection works. Test (T_3) had been introduced present situation with dredge cut at Launch Ghat including erosion protection works. Test (T_4) had been conducted present situation with adjusted channel in front of groins. Test (T_5) had been conducted with future situation without protection water levels had been measured. The measurement made in the model included water level, velocities and flow fields, reduction of the critical velocity (erosive) as well as changes of the flow pattern at the erosive zone by some protective measures and simultaneous increase of the critical velocity (silting) at accretion zone. An A-OTT current meter was used to measure the flow velocity. The instruments were used in the model for tide generating system given below

- Tape recorder
- Weir motor (Gate controlling)
- Servo Amplifier

- Water level follower
- Printer

The model tests were carried out for three flow conditions, low, medium and high. The model was run with rather large amplitudes (especially for low and medium flow conditions) in order to generate high velocities, as one of the main purposes of the investigation was to study the erosion. A summary of the prototype measurements for the model calibration and characterizing the three flow conditions are given below:

<u>Low flow condition</u>

Mean water level	: +0.30 mPWD (+1 ftPWD)
Max. tidal amplitude	: ± 0.63 m
Max. velocities	: 1.40 m/s downstream and 1.60 m/s upstream
Max. discharge	: Approx. 2800 m^3/s (downstream) and approx. 3000 m^3/s (upstream)

This corresponds to the flow conditions on January 26-27, 1983.

<u>Medium flow condition</u>

Mean water level	: +1.35 m PWD (+4.50 ftPWD)
Max. tidal amplitude	: ±0.50 m
Max. velocities	: 2.10 m/s downstream and 1.40 m/s upstream
Max. discharge	: Approx. 4000 m^3/s (downstream) and approx. 3000 m^3/s (upstream)

This corresponds to the flow conditions on June, 14-15, 1983.

<u>Highflow condition</u>

Mean water level	: +1.80 mPWD (+4.50 ftPWD)
Max. tidal amplitude	: ±0.32 m
Max. velocities	: 2.10 m/s downstream and 1.40 m/s upstream
Max. drop in water surface elevation	: 0.05 m downstream and 0.00 m slack water

This corresponds to the flow conditions on September 23-24, 1983

Results and discussion

Study of present situation (T_1)

Study of erosion

Evaluation of present erosion is based on the results obtained from the model reproduction of the present situation, study of erosion consisted of two items:

- Identifying the zones of erosion
- Classifying these zones according to severity

Both velocity and flow pattern were used for the above purpose. Identification of the zones of erosion has been primarily based on the study of flow patterns and severity of the erosion has been determined on the basis of velocity records and visual observations. Some of the areas were not apparently recognizable as having severe erosion problem from the surface flow pattern records. However, visual observation indicated the presence of strong transverse and spiral flow at those zones, where the occurrence of severe erosion is also supported by high velocity records.

Flow patterns for the low flow condition are shown by Figure 7a and Figure 7b.

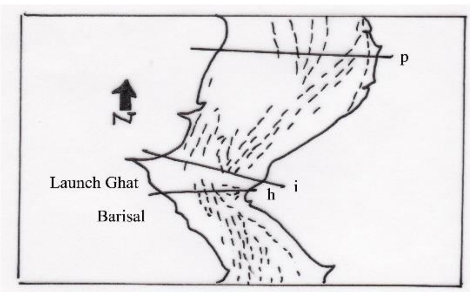

Figure7a. Flow pattern for low flow condition (flood tide)

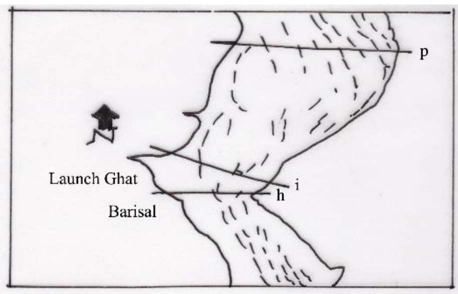

Figure7b. Flow pattern for low flow condition (ebb tide)

Examination of the figures indicates that the left bank of the river between sections '*p*' to '*i*' is under attack by the current during the ebb tide. The flow lines are almost parallel to the bank lines during the flood tide throughout the reach except between the sections '*i*' and '*h*' where vortices are developed close to the left bank during both tides. The vortex for the ebb tide condition is relatively stronger.

Figure 8a and Figure 8b shows the flow patterns for the medium flow condition.

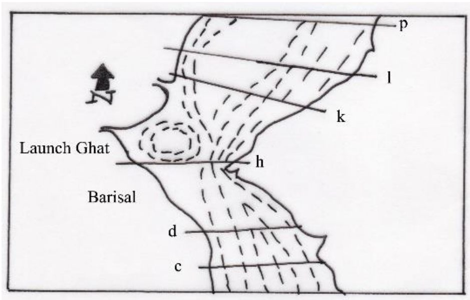

Figure 8a. Flow pattern for medium flow condition (flood tide)

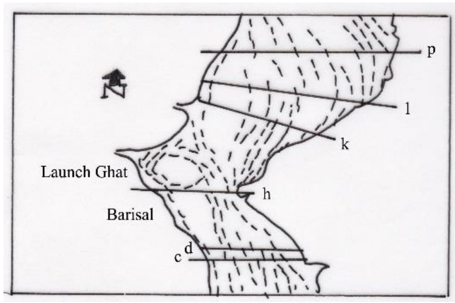

Figure 8b. Flow pattern for medium flow condition (ebb tide)

Here also the left bank is under attack during the ebb tide mild attack on the left bank is noticeable between sections *'k'* and *'c'*.

Strong flow concentration develops near the left bank around section '*h*' along with vortex formation close to the right bank between section '*i*' and '*h*' for both flood and ebb tides. Flow concentration is also noticeable at the right bank between sections '*k*' and '*l*' and a mild attack on the left bank between sections '*o*' and '*p*' during flood tide.

During high flow condition near stagnancy develops during flood tide and as such no photographs for flow pattern were taken. The attack by the flow in that case was shifted downwards and nearly same results were found as of medium flow condition.

From the study of the flow pattern as discussed above and also from visual observation the stretch of the riverbanks vulnerable to erosion is identified and shown in Figure 9.

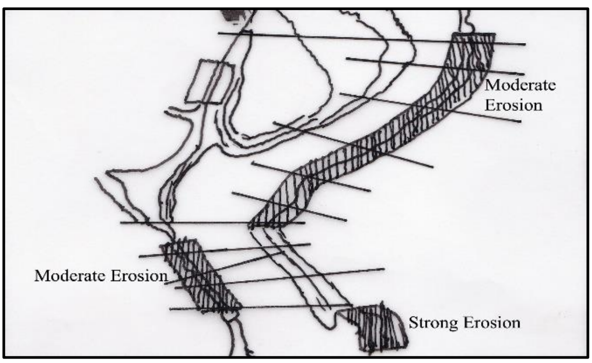

Figure 9. Bank erosion areas

Observation and comparison of velocity plotting for low, medium and high flows show that in general the velocity for the medium flow condition is higher than any of the other two situations. Therefore, the severity of erosion is governed mainly by the medium flow condition. To classify severity of bank erosion the bank limit was assumed 15 m and velocity was assumed at 0.8 depths.

Study of siltation

The problem of siltation had been studied near the BIWTA workshop jetty, CSD food jetty and Launch Ghat. Siltation study in this fixed bed model had been made indirectly from the velocity records and sediment size. Under the present data availability identification of siltation zones can only be made from consideration of permissible velocity. Since the reach is under tidal influence, siltation may occur during the slack current period at those sections where the permissible velocity is not exceeded at any time. Since the average velocities as obtained from model test were much higher than the permissible velocities, it is inferred that there is least probability of siltation at CSD Food jetty and BIWTA Workshop jetty location under the present situation.

Study of present situation with erosion protection works (T_2)

Four numbers of groins of appropriate size and shapes were placed at suitable locations after several trials, in the left bank of the river between sections '*q*' and '*k*', which is shown in Figure 10.

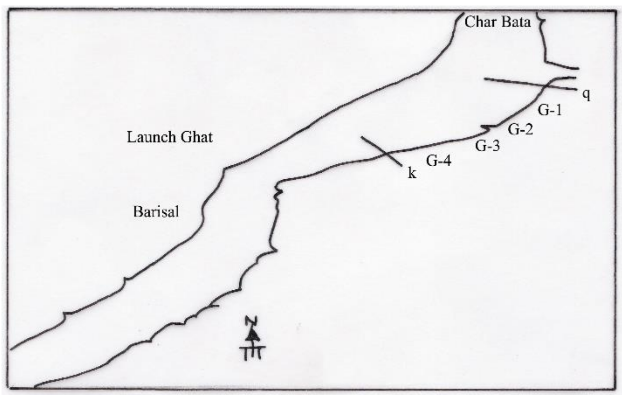

Figure 10. Location of groynes

From comparison of the flow patterns for situations before and after the placement of groins, it was seen that a favorable flow pattern had been achieved; the flow lines were more parallel to the banks.

Figure 11 shows the velocity distributions at 0.6 depths for situation before and after the placement of groins in the reach. From the figure it was observed the velocity had been considerably decreased near the bank reflecting a reduction of the erosive forces on the banks. Another important investigation area near BIWTA workshop jetty and CSD Food jetty are also influenced by the placement of groin. Here also the velocity and flow pattern are favorable.

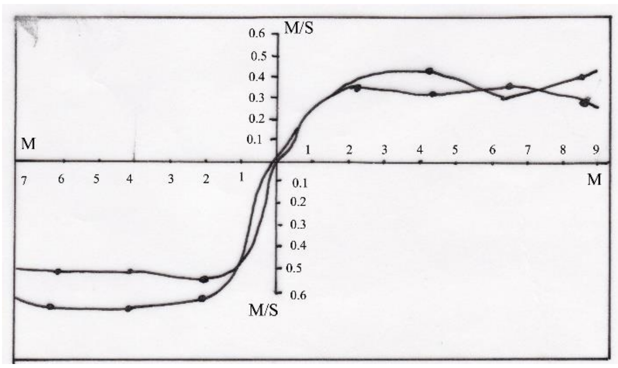

Figure 11. Record of velocity for test runs 1 and 2

Present status with dredging operation at Launch Ghat (T_3)

Under the present condition the dredged channel at Launch Ghat was an upstream blocked dead channel. During both flood and ebb tide the water within the channel remains almost stagnant, enhancing the siltation process compared to other adjacent areas. The test run had been designed to open up the upstream end of the channel by a dredge cut to give it a flushing effect during all flow conditions. The flushing effect is quite noticeable through the new dredge cut channel, with the flow going downstream through the channel during the flood tide and in a reverse direction during the ebb tide. In addition flow pattern also indicate a change in the characteristics of the vortex formed in that area.

Figure 12 shows the velocity distribution in the cut channel. The velocity records show clearly the generation of considerable velocities through the channel indicating a good flushing effect.

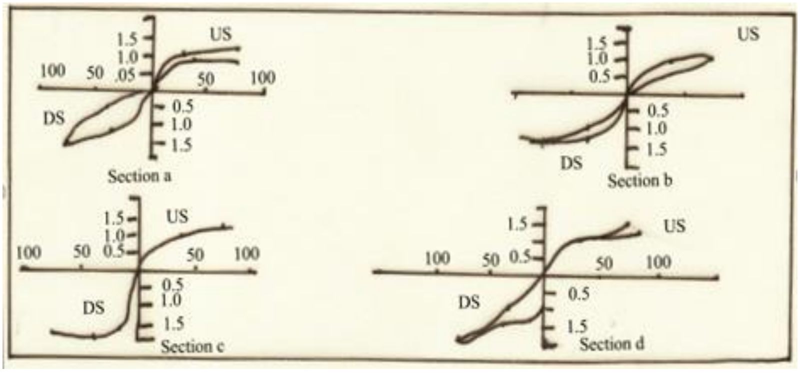

Figure 12. Velocity distribution in the cut channel

Present situation with adjusted channel in front of groynes (T_4)

Placing of groyne had restricted the effective width of the present channel. Consequently the char land in front of groins would be eroded to adjust the flow. The test run had been designed keeping in mind that the adjusted channel flow would have quite significant influence on the characteristic of the flow in general and in front of the dredge cut channel in particular. A comparison of the flood and ebb tide flow for this situation with corresponding flow before any channel adjustment in front of groynes had been made, it was clearly seen that an improvement on the flow pattern had occurred. It was therefore expected that in the actual situation placement of groins would have a better flushing effect through the dredge cut channel than as was indicated by the flow pattern shown in Test run No. 3.

Figure 13 shows the velocity distribution in the dredge cut channel for medium flow situation. This velocity distribution when compared with the corresponding velocity distribution for the Test run No. 3 situation indicates that there had been an overall increase in the velocity in the dredge cut channel showing a better flushing effect.

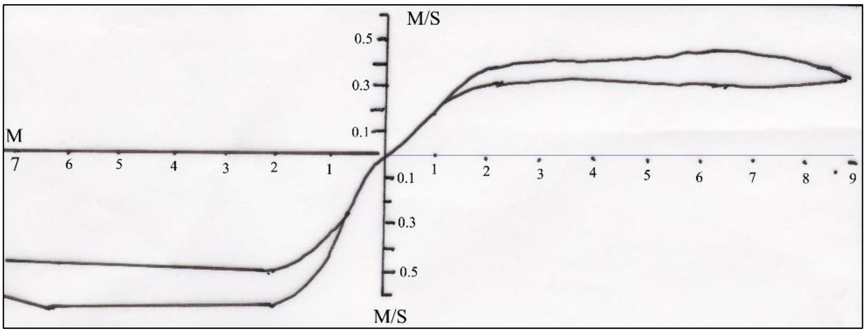

Figure 13. Velocity distribution in the cut channel in front of groynes

Future situation without protection works (T_5)

Study of future situation without protection works was needed to have a picture of erosion and siltation for the extrapolated condition in the year of 1992. Figure 14 indicates a considerable change of the flow pattern compared to the corresponding 1982 situation (Figure 8). The comparison of flow pattern shows that the existing (1982) siltation zone between sections '*q'* to *'j'* close to the right bank (near Char Upen) was likely to be extended downstream up to the section m in 1992 situation. This fact was also supported by the velocity records shown in Figure 15.

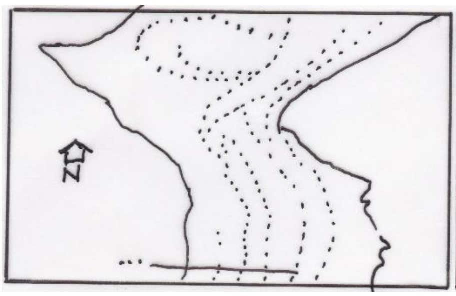

Figure 14. Flow pattern for test run 5 (ebb tide)

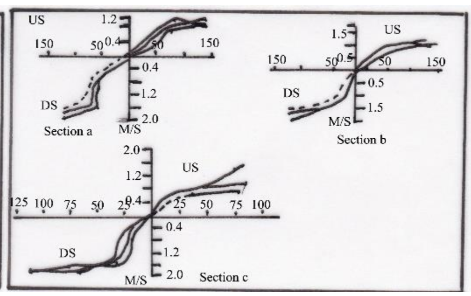

Figure 15. Velocity record for test run 5

Conclusions and recommendations

The model study carried out for Barisal Harbour area gave considerable support to the following recommendations:

a) Erosion protection works were necessary for zones subjected to moderate and strong erosion. Beneficial effects of groins were evident from the model results. Groins as protective measures were to be constructed on the left bank, the locations and sizes of which are shown in Figure 8. However, the model tests were conducted with only four numbers of groynes. Visual observation indicated formation of strong eddies in between groynes, which could have been eliminated with more number of groins.

b) Suitable bank protection works for the reach between BIWTA workshop Jetty and Cargo Ghat should be taken up.

c) The blocked upstream mouth of the existing dredged channel at Launch Ghat should be opened up for flushing effect.

d) There was no noticeable adverse effect near the CSD Food jetty under the extent of test runs conducted.

e) It is recommended to RRI authority to provide modern computerized tide and wave generator and necessary survey equipment and higher training/degree for tidal and wave model study in future.

Acknowledgement

Authors are grateful to water resources engineering department, BUET, BIWTA, BWDB officials and staffs who were engaged with the study of Barisal Harbor Area. The authors are also grateful to NHL (Norwegian Hydraulic Laboratory) officials/Engineers and inter-consultant A/S who was associated with the field survey, data preparation, model study works and instrumentation system as well as highly technical works for the Barisal Harbor Area study model. The authors are grateful to DHI (Danish Hydraulic Institute) Denmark for the technical support of Hydraulics and instrumentation especially for tide and wave generating system.

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PERFORMANCE EVALUATION OF BURAGHAT RUBBER DAM PROJECT IN IRRIGATION DEVELOPMENT AT HALUAGHAT IN MYMENSINGH

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Abstract

Rubber Dam is a latterly developed hydraulic structure which is usually built to impound surface water for a single purpose i.e. irrigation, flood control etc. In 1995, Rubber Dam projects were first commissioned in Bangladesh with the technical co-operation of the Institute of Water-resource and Hydropower (IWHR), China, as it is very convenient and cost-effective technology for irrigation. After implementing, it is very crucial and challenging task to study the suitability and effect of Rubber Dam in irrigation development. This study attempted to evaluate the performance of Buraghat Rubber Dam project (BRDP) and to identify the constraints embroiled to its efficient water management at Haluaghat Upazilla of Mymensingh District. The performance is evaluated by means of some technical parameters including Command area efficiency (CAE), Management performance ratio (MPR), Water-use efficiency (WUE) and Benefit-cost ratio (BCR). It is observed from the study that the CAE of scheme-1 and scheme-2 of the project is 30.71% and 27.14% respectively and MPR is 0.019 and 0.021 in that order indicating substandard. Furthermore, the study revealed that the WUE of both schemes is 32 kg m^{-3} and 37 kg m^{-3} respectively. However, BCR of the same is 1.50 and 1.52, which has found to be quite reasonable. Therefore, a considerable number of performance constraints of the Buraghat Rubber Dam irrigation project are identified to achieve the targeted fruitful outcomes.

Introduction

Bangladesh is a thickly populated agro-based country of South Asia. Irrigation plays a vital role in stimulating countries agricultural productivity as well as overall economic growth. However, continued water scarcities in the recent years have severely limiting irrigation facilities. Over withdrawal of groundwater through tube wells to meet the ever increasing irrigation demand and drinking water facilities has already created serious health hazards by means of arsenic contamination and other adverse environmental impacts. Therefore, surface water conservation and rainwater harvesting have become feasible options mitigating water crisis through minimizing excess withdrawal pressure on groundwater especially during dry season irrigation. Under these circumstances, Rubber Dam has evolved as a cost-effective hydraulic structure for surface water conservation of medium and small rivers. Rubber Dam is a new type of hydraulic structure as compared to the conventional gated structures like sluice gate, regulator, barrage etc. China is the pioneering country where Rubber Dams have been used successfully in small and medium rivers over the last 50 years (1960s) as the cheapest hydraulic structure.

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Rubber Dams have wide prospect in utilizing surface water for irrigation, hydro - power generation, flood control, pisciculture, water retention for aquifer recharge, reducing or preventing sea water intrusion into fresh water areas, protect low-lying coastal areas from tidal flooding, environmental improvement and recreation purposes.

The Dam structure consists of four parts: rubber bag, anchorage, foundation and pump house (filling and emptying system) as shown in Figure 2. Body of the Rubber Dam frequently termed as Dam bag is made of rubber reinforced by oven synthetic fabric. The reinforcement provides the tensile strength with rubber acting as the adhesive and water proofing element. The rubber fabric is a multi-layered fabric made of synthetic fiber (usually nylon), which is rubberized on one or both sides and coated, with plastic film. The thickness of the three-ply fiber reinforced rubber fabric is 8 mm. The fabric is quite flexible and exhibits good wear-resistance characteristics.

Rubber Dams are placed across channels and streams to raise the upstream water level when inflated and generally being bolted with a concrete foundation erected from the river or channel bed. A layer of stainless steel mesh or ceramic chips can be embedded in the surface layer to reduce or prevent vandal damage. Anchorage of rubber membrane can be made using concrete wedge blocks or steel clamps with anchor bolts. Rubber dams can be filled with water, air or both. Air filling is quicker but needs relatively sophisticated pumping and pipe-valve system. Water filling is relatively slow but ordinary pumps can be used with less complicated pie-valve system. Capacity of pump for filling or emptying depends on operational need of the dam. Air filled dams require low pressure typically 4 to 10 psi. The present trend suggests an increased use of air filled membranes because they can be inflated or deflated more rapidly, and they are less affected by freezing conditions. According to form of dam tube, they are classified into pillow like dam and inclined or slope dam.

Water is a vital and costly input for crop production. Since it is limited, every drop of water should be utilized properly for optimum yield especially for rice production. Therefore, in order to minimize irrigation cost and for the proper utilization of water, the Rubber-Dam irrigation project on Buraghat River under Haluaghat Upazila of Mymensingh district was planned earlier in 2004 but actually implemented in 2007. In the light of aforementioned issues, some specific objectives were considered to conduct this study and these are: (i) To determine the command area efficiency (CAE), management performance ratio (MPR), water-use efficiency (WUE), and benefit-cost ratio (BCR) of the project (ii) To identify constrains involved in the project and finally some recommendations were given for efficient water management of the BRDP.

Literature review

Salameh (2004) carried out a study on the Karama Dam with a capacity of 55 Mio m^3, which was constructed in 1995 on Wadi Mallaha in the Jordan Valley area with the aim of storing water for irrigational uses. The study findings expressed that the dam fails to collect water after nine years of its construction because there are no sources available to fill it. Dam reservoir was extremely saline (20,000 micro S/cm). Reservoir bottom collapses owing to dissolution of salts took place and large amounts of water were lost to the underground. Equipment of pump water for irrigational uses has been corroding, and the government is paying the depreciation, capital, and running cost of a fiasco project.

Lowitt (2002) discussed the construction and cost benefit analysis of the Optima Dam project located in Optima town in the central part of the Oklahoma Panhandle. This project was justified on the basis of its multiple purposes: flood control, probable irrigation, municipal water supply, silt control and recreation. The project was reconsidered in 1962, restructured in 1966 and completed in 1978. Unfortunately, flat lands, low runoff and high evaporation of the semiarid Panhandle made the area poorly suited to reservoir construction.

Ali and Wooldridge (1997) assessing the performance of G-K (Ganges-Kobadak) project, a large-scale irrigation scheme in Bangladesh with the command area of 1.25,000 ha. The study shows that the agricultural production in the command area has increased significantly under the project. However, project performance has deteriorated with time. The project has undergone major rehabilitation works aimed at overcoming physical constraints to the control of water distribution.

Dutta (1985) carried out a study on on-farm water management of 100 nos. of minor irrigation projects in Ghatail-Kalihati-Gazipur-Pubail area of Bangladesh. The study revealed that, the water loss in the deep tube-wells, shallow tube-wells and low-lift pump irrigation projects might be as high as 17%, 27% and, 21% of the pumped discharge per 100 meters length of the canal. The study also expressed that, the water loss relatively low in the canals, which were maintained well. Bhuiyan (1977) conducted a research work on "Evaluation of constrains to efficient water utilization in small scale irrigation schemes in Bangladesh." The study identified that, failure of pumps, poor maintenance of water distribution systems, pool, location of pumps and inefficient layout of canals, management performance of co-operatives are the key constraints against efficient water utilization. Lacks of motivation, inadequate credit facility, poverty, influence of pressure group etc. were some of the problems relating to irrigation water use.

Sarkar et al. (2001) evaluates the performance of Rubber Dam project in irrigation development at Nalitabari, Bangladesh on the basis of some performance indicator. The study disclosed that, the performance indicators like command area efficiency ranged from 58.45 to 85.34% with an average value of 63.91% management performance ratio varies from 0.012 to 0.114 with an average value of 0.028. Yield efficiency swayed from 33 kg m^{-3} to 54 kg m^{-3} with an average value of 41.8 kg m^{-3}. Overall agricultural benefit was augmented under that project where benefit-cost ratio ranged from 1.26 to 1.39 with an average value of 1.34. Miranda (1988) published that, the acceptable range of management performance ratio was 0.75 to 1.50 in irrigation practices in crop diversification project in Indonesia & Philippine. Haq et. al. (1985) reported that, the command area efficiency was 60% and 90% percent in Kharip-i and Kharip–ii season respectively, in the selected territories in G-K (Ganges-Kobadak) project.

Materials and methods

Study area

The location of the project area lies approximately between $25^0$7'N and $25^0$9'N latitude and between $90^0$22'E and $90^0$25'E longitude and positioned in south side of Buraghat River. The Buraghat River originated from Garo-hill (Meghalaya pradesh) of India and streams to the south side of the Haluaghat Upazilla. The total irrigable land under the studied project is about 700 ha (1729 acres). But actual area covered by the project is about 200 ha (494 acres). The topography of the study area is mostly flat which consists of 30 percent highland, 40 percent medium high land and 30 percent medium low land. The western and eastern part of the study area comprises of sandy sediments, elsewhere the deposits are predominantly clay, loamy, clay and sandy loam with pH ranges from 5 to 6.5. The reaction of the soil is highly acidic to low acidic. Climate in this area is cool, rainy and cool summer. Farmers under this project usually cultivated High Yield Varieties (HYV) of Boro rice. A few HYV of wheat and some vegetables are also cultured under the schemes of the project.

Salient features of the Buragh at Rubber Dam

The salient features and specifications of BRDP are given in Table 1.

Table1. Salient features and specification of BRDP

Dam features		**Specifications**
Length of Rubber Dam	:	30 m
Dam height	:	3.5 m
Maximum retention	:	3.5 m
Length of concrete floor	:	27 m
Dam body		
☐ Material	:	Reinforcement Rubber
☐ Shell thickness	:	8 mm
☐ Thickness of Cover	:	3 mm
Bridge	:	30 m
Guide bunds	:	8 km (Earthwork)
Approach road	:	2 km
Pump house	:	1 no. [for filling & emptying dam bag]
Bag filling time	:	12-15 hrs.
Pump Capacity	:	100 m^3/hr.
Scheme life	:	(15-20) yrs.

Figure 1. Typical view of Rubber Dam

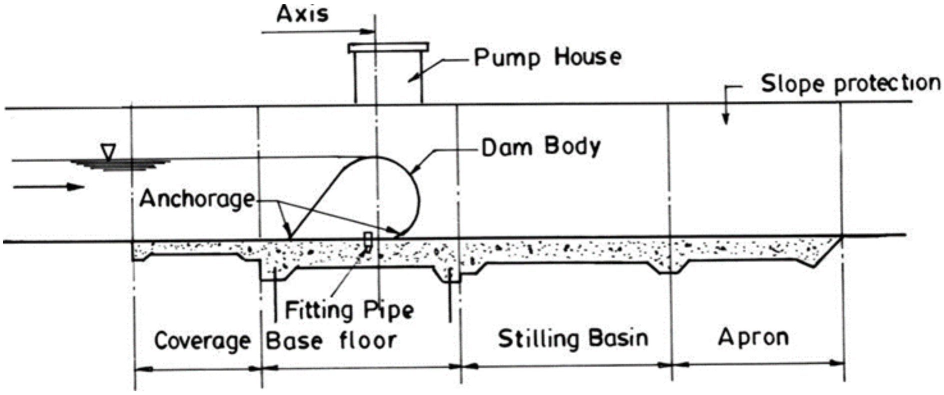

Figure 2. Schematic view (cross-section) of Rubber Dam

The irrigation schemes under BRDP are executed by Irrigation and Water Management Co-operatives (IWMC). It is formed with twelve executive members including chairman (1 no.), secretary (1 no.), joint secretary (1 no.) and general members (9 nos.) for various dealings. Water charge for BRDP was fixed at Tk. 2470.00 per hectare and usually, the rate has been settled by its executive committee on the basis of total investment cost. The general meeting normally held in the last week of every month in the association office.

Data collection

Data were gathered from Local Government Engineering Department (LGED), Mymensing has well as Irrigation and Water Management Co-operatives (IWMC) of BRDP situated in Gazir Bhita, Haluaghat, intended to the assessment of existing management practices of the schemes. Congregated information was then justified through non-formal field inspections and interviews with secretary and joint secretary of IWMC and farmers. The data are presented herein:

- Approximate discharge: 1.13268 m^3s^{-1}
- Type of irrigation system: Gravity Flow.
- Size of the Inlet structure (Main canal): Rectangular; (Width: 1.5 m, Height: 1.8 m)
- Length of main canal: 1700 m.
- Nos. of Secondary canal: 3
- Size of Secondary canal : Rectangular; (Width: 0.75 m, Height: 1.0 m)
- Area covered by Main canal: Surjopur village and Gazir Bhita village (Total = 86 ha).
- Similarly Three Secondary canal covers: Uttar Nolkora village, Katolmari village and Purbo Somalia Para village (Total = 114 ha).
- Capital of IWMC: Tk. 244560 (1 USD≈78 BDT).

Theoretical considerations

The theoretical considerations were reviewed by Molden and Gates (1990) and Molden et al. (1998).

Command Area Efficiency (CAE)

It is the ratio of actual command area to the potential command area and expressed in percentage.

$$\text{Command Area Efficiency} = \frac{\text{Actual Command Area}}{\text{Potential Command Area}} \times 100$$

Management Performance Ratio (MPR)

It is the ratio of total volume of water supply to total volume of water demand.

$$\text{Management Performance Ratio} = \frac{\text{Total Volume of Water Supply, m}^3}{\text{Total volume of Water Demand, m}^3}$$

$$\text{Total volume of Water Supply} = \text{Actual discharge capacity} \times \text{Total operating time}$$

$$\text{Total volume of Water Demand}$$

$$= \text{Irrigation water requirement for crops} \times \text{Actual command area}$$

Water-use Efficiency (WUE)

It is expressed as the rate of biomass produced (or harvested yield) per cubic meter of water used.

Benefit-Cost Ratio (BCR)

It is the ratio of gross return to total cost.

$$\text{Benefit Cost Ratio} = \frac{\text{Gross Return}}{\text{Total Cost}}$$

Total Cost includes cost of seed or seeding, fertilizer, plough, labor, insecticides, tax, and organization maintenance in Tk. per hectare.

Gross Return: It includes the value of crops and Straws in Tk. per hectare.

$$\text{Net Return} = (\text{Gross Return} - \text{Total Cost})$$

Results and discussion

Command Area Efficiency (CAE)

Command area efficiency of scheme-1 and scheme-2 of the BRDP are tabulated in Table 2. Command area efficiency depends largely upon irrigated area and potential command area. Actual command area depends on wide variety of factors like farmer's involvement in irrigation, interest for cultivation, regular maintenance of water conveyance system, favorable soil conditions, efficient water management practices etc. The lower command area efficiency had occurred due to lower discharge from rubber dam reservoir, considerable amount of water losses from the conveyance system, which is at about 40% and poor water management practices in the irrigation system.

Table 2. Command area efficiency of the BRDP

Sl. No.	Name of irrigation schemes	Irrigated area, (ha)	Potential Command area, (ha)	Command Area Efficiency, (%)
1	Scheme-1	86	280	30.71
2	Scheme-2	114	420	27.14

Management Performance Ratio (MPR)

The management performance ratio (MPR) indicated (Table 3) that both schemes of BRDP were poorly performed due to higher farm canal density; lower operating times and delayed starting of irrigation activity. The discrepancy of MPR denoted that there was great scope to get better water management practices in these existing schemes of the project.

Table 3. Irrigation water management performance ratio of schemes of the BRDP

Sl. No.	Name of irrigation schemes	Name of the village under the scheme	Distribution pattern	Total volume of water supplied (including conveyance loss) (m^3)	Total volume of water demand (m^3)	MPR
1	Scheme-1	Surjopur , GazirBhita	Main canal	4377694	220160000	0.019
2	Scheme-2	Uttar Nolkora, Katolmari and Purbo Somalia Para.	Secondary canal	5802990	273600000	0.021

Water-use Efficiency (WUE)

Water use efficiency of both schemes under the rubber dam project ranges from 32 to 37 kg m^{-3} with an average value of 34.5 kg m^{-3} (Table 4). Higher yield efficiency could be attained reasonably by providing training on effective irrigation planning and better water management practices to the farmers.

Table 4. Performance of Water-use efficiency of schemes of the BRDP

Sl. No.	Name of irrigation schemes	Irrigated area (ha)	Average yield (kg/ha)	Total volume of water supplied (including conveyance loss) (m^3)	Water-use efficiency (kg/m^3)	Average Water-use efficiency (kg/m^3)
1	Scheme-1	86	6131	4377694	32	34.5
2	Scheme-2	114	6263	5802990	37	

Benefit-Cost Ratio (BCR)

Benefit-Cost ratios of both schemes of the Buraghat Rubber Dam Project (Table 5) were reduced in consequence of mounting total cost and decreased gross return. Effective marketing system should be developed with the intention that, farmers can sell their product at more competitive price.

Table 5. Performance of benefit-cost ratio of schemes of the BRDP

Sl. No.	Name of irrigation schemes	Total cost (Tk. /ha)	Gross return (Tk. /ha)	Net return (Tk. /ha)	Benefit-cost ratio
1	Scheme-1	53549	77805	26920	1.502
2	Scheme-2	52610	77237	25027	1.519

Performance constraints of irrigation schemes of BRDP

This investigation identified some performance constraints of the Buraghat Rubber Dam irrigation schemes. The constraints involved were identified through inspecting physical condition of the sluice gate, river as well as canal situation of the project site and assessment of information collected from the farmers. Various constraints includes: there are no drainage facilities to remove excess water, insufficient storage of water, leakage problem, siltation problem, high conveyance loss in the canal, conflicting interest of water suppliers and water users, factional conflict among IWMC members and beneficiaries, unfamiliarity of Rubber Dam technology, lack of consciousness regarding irrigated agriculture and defective water charge collection system.

Conclusions and recommendations

The study was directed with the aim of appraising performance of the Buraghat Rubber Dam Project (BRDP) on the basis of some performance indicators. It was found from the study that the Command area efficiency of the BRDP is not adequate. The smaller value of Command area efficiency (CAE) shows that considerable amount of land are yet to be brought under irrigation by optimizing management practices. The Management performance ratios (MPR) were lower than the acceptable range of MPR and the Water-use efficiency (WUE) of the schemes were better. However, Benefit-Cost ratio (BCR) of the same was quite reasonable. Therefore the following recommendations are drawn to improve the performance of the irrigation schemes under this project

- Training facilities focused on modern agricultural technology should be increased for concerned personnel.
- Crop diversification could be added into crop-based irrigation as this cropping system might assure better project performance through extending command area.

- Irrigation canals should be lined with stable materials to reduce leakage and conveyance losses. Farmers should take care of wastage of water by filling ditches, ponds and burrows in the canal bunds, run-off in the drain and overtopping the canal bunds.

- Suitable crop calendars should be developed and appropriate irrigation schedules should be prepared in relation to the varying water needs of crop at different stages of growth.

- The main and distribution canal systems should be controlled by yearly re-excavation to get proper function.

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OPERATIONAL BEHAVIOR AND PERFORMANCE OF PUMPS IN PHYSICAL MODELING AT RIVER RESEARCH INSTITUTE, FARIDPUR.

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Abstract

This paper deals with the operational behavior of pump and pump performance characteristics, installation, maintenance and major trouble and counter measures of pumps mainly used in modern water works specially for River Research Institute (RRI). The aim of this paper is to provide an engineer with basic information on pump characteristics that will enable him to make a first estimate of the number, dimensions and types of the pumps he will need and will provide him with a sound basis for choosing the right type of pump for the job. Maximum efficiency of pump was found 92% at 350 m^3/min specific speed. Pump performance was expressed by characteristic curves for total head, shaft power and efficiency. For satisfactory pump operation it is necessary to examine the suction condition prevailing at the pump inlet. It was observed that available net positive suction head (H_{SV}) was always larger than the required net positive suction head (NPSH). It is recommended to provide the available NPSH to be more than 1.3 times the value of H_{sv}. When a demand pattern against the time was plotted it was observed that the demand must be met with the selected number of unit of operation. It is recommended that the number of pumping units and their rated capacities must be determined so that the cost involved in investment and operation is economically viable. The investment cost will be increased with the increase of the number of units of pumps. From the study it was observed that a larger unit is preferable in view of efficiency providing less operating cost per unit capacity pumped. Periodic maintenance is needed for long survival of the pump.

Introduction

Pump is the heart of any hydraulic system as it determines both the efficiency and reliability of the system. River Research Institute (RRI), Faridpur conducts many physical modeling to investigate river morphology. To effectuate these studies pumps are to create water flow. The pumps used at RRI are centrifugal pumps. A centrifugal pump is a rotodynamic pump that uses a rotating impeller to increase the pressure of a fluid. The pumps are designed on the principle that mechanical energy is converted into fluid energy by rotational motion of an impeller and the liquid leaving by the impeller is collected by a casing so that the liquid is lifted up or forced into a pressure vessel. There are many types of

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pumps, such as reciprocating, rotary, regenerative and jet types. The majority of heavy duty pumps used in modern water works facilities and liquid handling industries are of turbo type such as centrifugal, mixed flow and axial flow pumps (Ritzema 1994).

The hydraulic efficiency of a centrifugal pump depends significantly on the impeller and casing geometries and small changes in geometrical details can lead to large changes in performance (Moussa 2014). Important characteristics of centrifugal pump (1) Pressure head: Pressure is the driving force which is responsible to movement of liquid fluid (2) Friction head: Friction is the resisting force that slows down fluid particles. (3) Flow head: Flow rate is the amount of volume that is displaced per unit time. The objective of the paper is to provide the fundamental knowledge and information regarding determination of pump specification, selection of types, operating conditions and routine maintenance for application of pumps in water works for Physical modeling at RRI

Literature review

Modeling of pump application and efficiency has been addressed in the literature by several authors. Wilson (1946) published the basic models for the torque and volumetric efficiencies of positive displacement pumps. Using Wilson's basic model, Shlosser (1961) introduced nonlinear terms to account for the leakage flow that occurred at high Reynolds number flow and torque losses due to fluid acceleration. Zarotti and Narvegna (1981) demonstrated that this assumption was inadequate as the models deviated substantially from experimental data across a wide operating range of the pump. Consequently, these authors proposed more sophisticated models to account for the variation in loss coefficients. Pourmaovahed (1992) published his work which identified the large amount of scatter that exists in experimental efficiency data. This work has since been used to reconsider not only the reliability of the test data itself, but also the reliability of the previous modeling that has been done based upon these data. Bacharoudis et al. (2008) stated that various parameter such as performance of impeller, blade angles affect the pump performance and energy consumption. Navier-Stokes equations and Computational Fluid Dynamics (CFD) finite volume was used to calculate the performance of each impeller, the flow pattern and the pressure distribution in the blade passages. The results showed that increase in outlet blade angle causes a significant improvement of the hydraulic efficiency of pump. Taber (2011) investigated the influence of impeller diameter and speed on performance of centrifugal pump. The results showed that change in impeller speed and diameter has significant impact on performance of centrifugal pumps. Affinity laws were used to predict the performance of centrifugal pump at reduced speed or smaller diameter impeller. It was found that decrease in impeller diameter result in the reduction of the head, flow and also drops the efficiency.

Patel and Doshi (2013) examined the influence of blade angle on performance of a centrifugal pump. The investigation of blade exit angle was expensive and lengthy process thus a mathematical model was developed to investigate the effect of blade angle on performance of centrifugal efficiency. It was found that, both head and efficiency of centrifugal pump increase with the increase in blade exit angle. Kumar et al. (2014) examined the effect on impeller parameters such as number of blade, inlet blade angle etc. on the performance of centrifugal pump. The Computational Fluid Dynamics (CFD) analysis was implemented and velocity and pressure distribution were predicated. The optimum values of impeller were discussed and guidelines were presented in order to improve the efficiency of centrifugal pumps.

Methodology

A number of journal, books, reports, lecture notes, pump catalogues were collected and studied thoroughly on application of pumps and pumps characteristics for water works. Theoretical knowledge was gathered and necessary reviews were drawn on the basis of easy operation and maintenance of the pumps.

Specific speed

Hydraulic performance of a turbo pump is determined by three important factors. These are capacity, speed and total head. Using these three factors, we can determine the specific speed by the following equation

$$N_{\alpha} = \frac{N\,Q^{1/2}}{H^{3/4}} \qquad \text{eq. (1)}$$

Where, N = speed (rpm), Q = capacity (cum/min), H = total head (m)

The specific speed in the above equation is an essential criterion to determine the impeller shape as well as hydraulic characteristics of the pump. The shape of pump impeller varies continuously according to the values of specific speed. In general small values of the specific speed are suitable for high heads with small capacities while large values are for low heads with large capacities. (Manual for short course on application of pumps for irrigation and drainage)

Efficiency and required Power

The efficiency of a pump is defined as a ratio of the pump output to the shaft power input. The pump output refers to the theoretical hydraulic power imparted to the liquid leaving the pump and is obtained by multiple of liquid density, capacity and total head as given by

$$L_W = d.g.(Q/60).(H/1000)kW \qquad \text{eq. (2)}$$

Where, L_W = pump output, d = density of liquid (kg/m^3), g = acceleration of gravity (m/s^2), Q = capacity (m^3/min), H = total head (m)

The above equation could be reduced to

$$L_W = 0.163 \times Y \times Q.H \qquad \text{eq. (3)}$$

Where, Y = Specific weight of liquid (kgf /litre)
Then the efficiency of pump is given by

$$e_P = 0.163 \times Y \times Q.H / L_P \qquad \text{eq. (4)}$$

Where L_P = Shaft power input (kW).

Pump performance was expressed by characteristic curves for total head, shaft power and efficiency plotted on the ordinates against capacity on the abscissa. (Manual for short course on application of pumps for irrigation and drainage)

Suction performance

For satisfactory pump operation it is necessary to examine the suction condition prevailing at the pump inlet. The suction condition is best expressed in terms of "available" NPSH (Net positive Suction Head). This is calculated by

$$H_{SV} = H_A + h_S - h_i - h_V \qquad \text{eq. (5)}$$

Where, H_{sv} = NPSH available (m), H_A = absolute pressure head acting on suction liquid lever (m), h_S = actual suction head (m), (for suction lift it becomes a minus value), h_i = friction loss head in suction pipe (m), h_V = saturated vapor pressure head (m).

The value of required NPSH can be obtained from "Suction specific speed" S defined by the following equation.

$$S = \frac{N Q^{1/2}}{h_{sv}^{3/4}} \qquad \text{eq. (6)}$$

Where, S = suction specific speed, N = speed of pump (rpm), Q = capacity (cum/min), h_{sv} = NPSH required (m)
(Manual for short course on application of pumps for irrigation and drainage)

Determination of total heads

Total head of a pump is obtained by the sum of actual head and hydraulic losses in the connected pipes, fittings and valves and is given by

$$H = H_a + H_r \qquad \text{eq. (7)}$$

Where H = total head (m), H_a = actual head (m), H_r = sum of total losses (m)

Actual head refers to the static difference between suction and discharge liquid levels prevailing during pump operation.
(Manual for short course on application of pumps for irrigation and drainage)

Selection of pump type

As a preliminary consideration, the hydraulic type of pump was selected according to the total head required for the pump to be used. A general guide for type selection in terms of the total head is given in the following table for conventional pumps, in which applicable bore sizes are also shown.

Table 1. Selection of pump types

Pump Type	Specific Speed	Total Head		Bore
		Horizontal	**Vertical**	
Centrifugal (Radial Flow)	100-600	Single Stage 10-150m	Single Stage 10-200m	>40mm
		Multistage>50m	Multistage>10m	
Mixed flow	400-1400	4-15m	Single Stage 4-60m	>200mm
			Multistage>10m	
Axial flow	1300-2000	<6m	<8m	>300mm

For the required capacity and the total head, the pump speed can be obtained by

$$N = \frac{\left(N_{\alpha} H^{\left(3/4\right)}\right)}{Q^{\left(1/2\right)}} \qquad \text{eq. (8)}$$

Selection of higher specific speed may result in smaller pump size. When the pump is directly coupled with an electric motor or and engine, the speed must be adjusted to suit the rated speed of the selected prime mover. The synchronous speed of a three phase AC motor is given by

$$N_{mn} = 120\, Hz / N_{P} \qquad \text{eq. (9)}$$

Where, N_{mn} = motor Synchronous speed. Hz = frequency of AC Electric Source. N_P = number of poles.

Results and discussion

The value of pump efficiency depends mainly on the specific speed and capacity of the pump is shown in Figure 1.

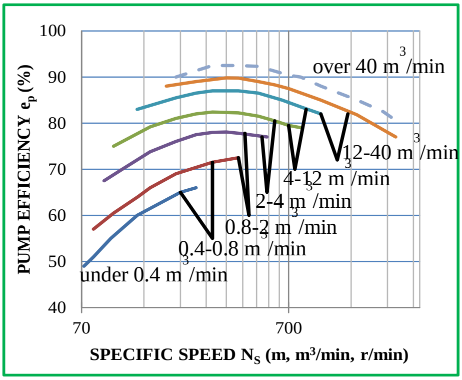


Figure 1. Pump efficiency versus specific speed curves

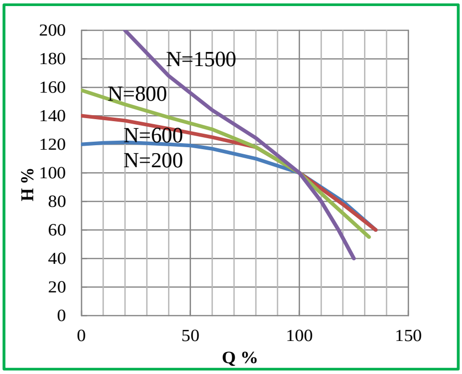


Figure 2. Pump characteristics for different specific speed

Pump characteristics

The forms of characteristic curves were closely related to the values of specific speed as shown in Figure 2. In which each characteristic was given in percentage of the value at the best efficiency point. Each characteristic has the following tendency:

a) Total head capacity

For small specific speed values, the curve is flat with a small shut off to rated head ratio. Whereas the curves becomes steep as the specific speed increases.

b) Shaft power

The shaft power attains the minimum value at shut off for small specific speed values, rising as capacity increases. For mixed flow range, it remains about the same at all capacities. Whereas for an axial flow pump it increases remarkably towards the shut off.

c) Efficiency

The variation in efficiency against capacity is more favorable for smaller values of specific speed.

The value of available NPSH was solely determined by installation conditions of a pump are shown in Figure 3.

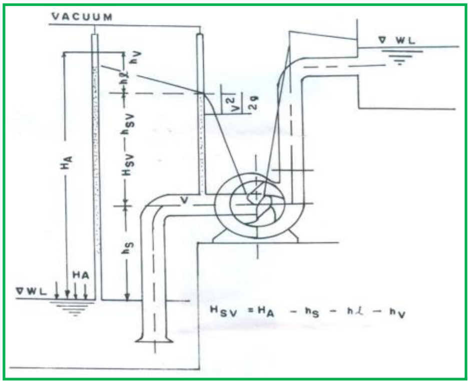


Figure 3. Net positive suction head

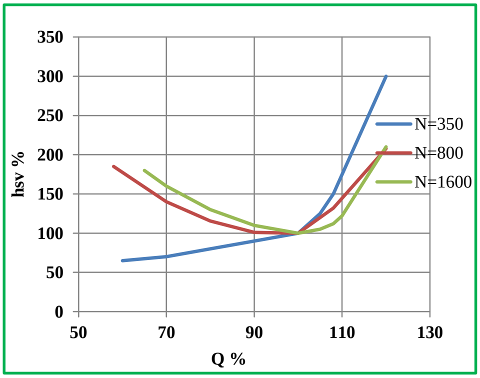


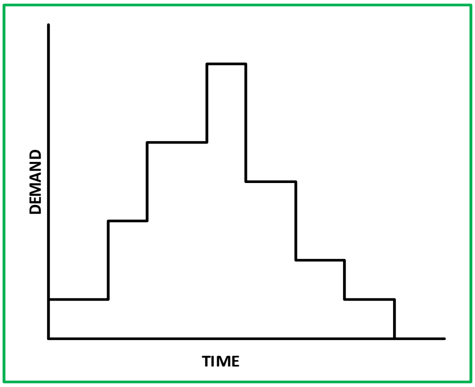
Figure 4. Variation of required NPSH against capacity

Figure 4 shows the tendency of required NPSH variations against capacity for representative values of specific speed. The available NPSH, h_{SV} must always be larger than the required NPSH. It is recommended to provide the available NPSH to be more than 1.3 times the value of h_{sv} as given by equation

$$h_{sv} = \left[\frac{N Q^{1/2}}{S} \right]^{4/3} \qquad \text{eq. (10)}$$

Adaptability to variable demand

In pumping services, the demands often fluctuate considerably according to time periods and seasons of the year and purposes. When an appreciable variation in demand exists, pumping must be divided into several units and different sizes. It must be noted that the most economical operations could be attained by operating the pump in its optimum efficiency point without throttling. A demand pattern against the time is shown in Figure 5 it was observed that the demand must be met with the selected number of unit of operation. The demand can be expressed in the order of required capacity interims of operating time duration as shown in the Figure 6. When two identical large pumps and one small pump are considered V_{1p}, V_{2p}, V_{3p} represent the pump capacities with one, two, three units respectively.



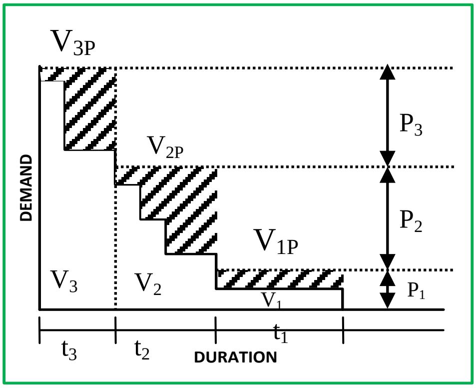


Figure 5. Demand vs. Time curve

Figure 6. Demand vs. Duration curve

As the demands for respective periods are given by V_1, V_2, V_3 the excess capacities are expressed by the hatched area which should be made minimum to attain a higher degree of adaptability.

For centrifugal pumps the pump speed can be checked as shown in Figure 7, in which H_S represents the suction actual head minus the head losses at the suction side. The operating capacity range was assumed to be up to 120% of the BEP (best efficiency point).

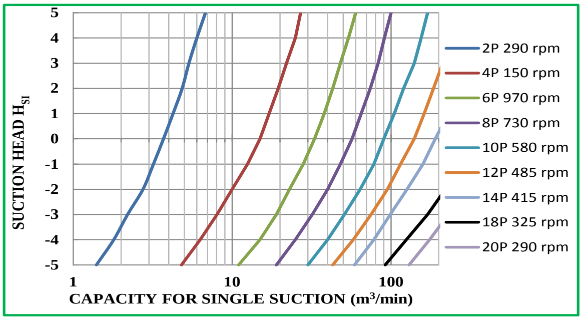


Figure 7. Suction head and speed for centrifugal pumps

Basic features of pump operation

Installation of pumps

To provide proper functioning of pumping equipment, it is necessary to follow established installation procedures as outlined below.

a) Foundation

In most cases, the pump was installed on a concrete foundation. The foundation is required to have sufficient strength to withstand the weight of pumping unit and dynamic load exerted by its operation. The base plate of pump is usually fixed by means of anchor bolts, for ample sized holes must be provided for grouting in the foundation or the base plate itself.

b) Placement

For positioning the pump unit steel liners are placed between the foundation and the base plate. The liners should be placed at both side of the anchor bolt. Use of a pair of tempered liners is advantageous for fine adjustment. After primary checks on concrete positioning, the anchor bolts can be grouted with mortar.

c) Alignment

For providing vibration free operation of pump exact alignment at the coupling is indispensible. Factory adjusted adjustment can be resolved by adjusting respective liners under the base plate. Alignment can be checked with a straight edge and a filler gauge. Installation of connecting pipes and valves must avoid undue forces exerted on the pump exit. After rechecking the alignment, the liners are preferably spot welded and the base plate can be completely grouted after which the coupling bolts are finally inserted.

Operation procedures

Preparation: Prior to initial startup of a pump the following preparation in general to be performed and confirmed, which also applies when a pump is to be started after a long idle time.

a) Confirmation on liquid passages

Suction sump or chamber must be cleaned of filled with liquid to be pumped, piping joints must be tightly connected to ensure leakage free operation.

b) Check on lubricants

Amounts of lubricating oils must be checked by oil level gauges etc. The same shall apply for the driving equipment.

c) Confirmation on power source

For motor driven pumps due preparation of cabling and electric power should be made. Protective device should be checked for functioning.

d) Auxiliary equipment and instruments

All necessary related equipment must be confirmed of their proper functioning.

Starts and stops

Established order should be followed in performing normal starts and stops. In normal cases the following order should be observed on pump types.

a) Starting order

- Confirm that discharge valve is fully closed.
- Confirm that the prime mover is ready to start.
- Confirm that suction liquid level or pressure is normal.
- Start and confirm necessary sealing/lubricating/cooling.
- Priming of pump by vacuum pump if applicable.
- Confirmed that discharge pressure reaches prescribed value after full speed is attained.
- Open discharge valve and check discharge pressure which must correspond to the value within operational range.

b) Stopping order

- Fully close the discharge valve. Switch off prime mover.
- Rest prime mover to its starting position.
- After complete stop of pump, stop sealing/lubrication/cooling.

Maintenance

Periodic maintenance

To maintain prescribed performance of a pumping unit for its service life, it is indispensible to provide appropriate maintenance services, it is always recommended "preventive maintenance" which is termed as scheduled maintenance be performed to prevent occurrence of troubles and to maintain proper functioning of the equipment. For a motor driven pumping unit to be operated continuously, list of periodic inspection on items is given below:

Table 2. Inspection items for pumps

Interval	Inspection items	Remarks
Daily	Appearance as to leaks etc Noise & Vibration, Bearing Temperature Gland packing	To be within ambient temperature; +40º c Leakage 1 drop/sec
Monthly	Bearing lubricants Gland packing	Check quantity check for gear
Every 6 Months	Replacement of bearing Lubricants. Replacement of fixing bolts, Check on Protective devices.	
Every 1-4 year	Overhaul inspection rubbing parts and corrosion/erosion of parts	Checks for wear on rubbing parts and corrosion/erosion of wet parts

Major troubles and counter measures

During service life of a pump some troubles may be encountered depending on operating conditions. Vibration of a pump is most effectively detected at its bearing and at the top of the unit, and its amplitude and frequency are measured

by vibration meter. Figure 8 shows permissible range of amplitude against pump speed.

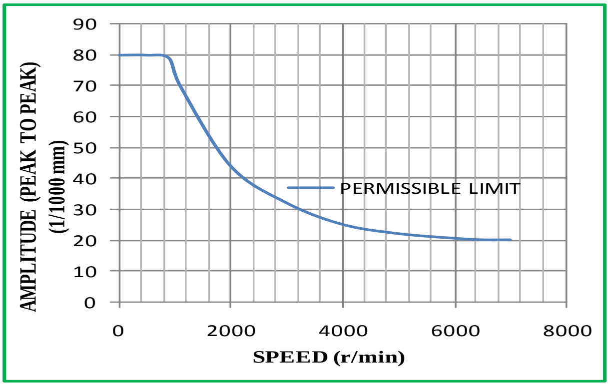


Figure 8. Permissible variation amplitude

Vibration modes are classified into: (1) vibration with the same frequency as the pump speed, (2) vibration with frequency several times the pump speed and (3) vibration with irregular frequencies.

Major causes and counter measures are given in the following Tables 3, 4 and 5 respectively.

Table 3. Vibration with the same frequency as the pump speed

Items	Cause & checks	Counter measures
Misalignment	Check coupling alignment	Readjustment of alignment
Distortion of centering during operation	# Hydraulic thrust acting from discharge pipe # High temperature liquid Causing displacement of center height # Non uniform diameter of coupling bolt bushings	Apply stay tools on flexible joints Pump type to be changed Use uniform Bushings
Inferior Foundation	# Vibration observed on weak foundation # Anchor bolts loosened	Reinforce foundation Tighten anchor bolts
Unbalance of Rotating component	# Bending of shaft # Uneven impeller wear # foreign matter inside Impeller # Bearing wear	Straighten or replace Balance impeller Remove foreign matter Replace bearing

Table 4. Vibration with frequency several times pump speed

Items	Cause and checks	Counter measures
Pressure pulsation	Piping and/or casing vibrate with frequency ZN, where Z = no, of impeller blades, N = pump speed support Vibration with frequency ZN where Z = least common multiple of Z and number of guide vanes.	Enclose clearance between impeller and volute tongue; Reinforce piping or casing Enlarge clearance between impeller and guide vane inlet
Antifriction bearing failure	Pump vibrates with frequency PN where P = Number of bolts/rollers, N = pump speed (Bearing temperature often high)	Replace bearing avoid bust/water intrusion into bearing

Table 5. Vibration with irregular frequencies

Items	Cause and checks	Counter measures
Partial capacity	Check operating capacity by reading discharge and suction pressures (Vibration is enhanced at capacities less than the rated)	Keep discharge valve full open. Increase capacity by providing bypass piping.
Cavitations	Check available NPSH against required NPSH at operating capacity. (Noise is accompanied and pressure gauge fluctuates)	Reduce suction lift. Enlarge suction pipe diameter. Remove clogging.

Conclusion

Attempts had been made to find out the classification of pumps, pump performance characteristics, selection of pump types, pump specifications, installation, operation, repair and maintenance, major troubles and counter measures etc. Maximum efficiency of pump was found 92% at 350 m^3/min specific speed. Pump performance was expressed by characteristic curves for total head, shaft power and efficiency. For satisfactory pump operation it is necessary to examine the suction condition prevailing at the pump inlet. It was observed that available net positive suction head, (H_{SV}) was always larger than the required net positive suction head (NPSH). It is recommended to provide the available NPSH to be more than 1.3 times the value of H_{sv} It is recommended that the number of pumping units and their rated capacities must be determined so that the cost involved in investment and operation is economically viable. The investment cost will be increased with the increase of the number of units of pumps. A larger unit is preferable in view of efficiency providing less operating cost per unit capacity pumped. Investment and operating cost should be kept minimum over and anticipated designed life of pumping equipment. In order to ensure uninterrupted operation it is recommended to install at least two units in most cases. A standby unit is often necessary to replace against breakdown of one unit and for and important application requiring uninterrupted operation over a long period. For proper functioning of pumping units it is necessary to follow established installation procedure such as foundation, placement, alignment etc. For initial startup of the pump care should be taken such as confirmation on power source etc. It is always recommended "preventive maintenance" that is scheduled maintenance should be performed to prevent occurrence of troubles and to maintain proper functioning of the equipment. Periodic inspection is needed to operate a motor driven pumping unit. Periodic inspections are: (1) daily inspection for leakage, noise and vibration, bearing temperature, gland packing etc. (2) Monthly Inspection for bearing lubricants, gland packing etc. (3) For every six months replacement of bearing lubricants and gland packing, tightening of fixing both etc. (4) For each year overhaul inspection is needed. For inspection of motors it is recommended to check noise and vibration, bearing temperature, slip rings brush movement, bearing lubricants, insulation resistance, check on protection devices, replacement of bearing lubricants etc. This review paper hopefully will be useful in application of pumps in physical modeling at RRI.

Acknowledgement

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EFFECT OF PARTICLE SHAPE ON ITS OTHER GRANULAR SOIL PROPERTIES

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Abstract

Particle shape is usually determined for the coarse grained soil. It is one of the most important soil-grain properties. The other important soil grain properties are angle of shearing resistance of sands, gradation, denseness i.e. density index, uniformity of gradation etc. The mentioned factors have some inter related impacts. The strength of soil is an utmost important factor among all soil engineering parameters and is determined finding out the angle of shearing resistance. In this paper, particle shapes and particle sizes are determined minutely in the laboratory and its in-situ test such as SPT values and corresponding density indices; others are collected from the report of soil mechanics division of geotechnical research directorate of River Research Institute (RRI). In order to find out the other granular properties by determining particle shapes an attempt has been made to predict the strength of grain soils through conventional means. The study finds that SPT values as well as strength of granular soils increases with depth. It is also found that particle sizes increase with depth and their SPT also increases with depth. The shapes are found sub-rounded with medium spherecity while the depths are increased. The study result will help find out the denseness and uniformity of gradation of relevant soils in any location of Bangladesh. So, it will be an opportunity for a design engineer who may expedite the construction works by determining the particle shape of granular soil. Moreover, a design engineer may predict the granular soil strength and other grain properties through determining the particle shape of coarse grained soils. The findings of the present study are expected to help predict granular soil strength and other grain properties of any location by determining particle shape.

Introduction

Bangladesh is a developing country where lots of structures are being built on the soil. In general, Bangladesh soils contain sand, silt and clay. We know the compositions of all the ingredients make soil, but all ingredients are not in the same ratio. As a result, their classification has become different. Soils are classified according to the quantity of ingredients present in the composition and dominate the character of this particle on the soil. Indeed, soils behave in terms of loading, sewerage according to their sizes and shapes. On the other hand, particle shapes of non-cohesive soils have good linkage to its strength. Strength is an utmost important property of construction soil. Sometimes it is found that strength is difficult to determine for non-cohesive soils. As non-cohesive soils usually are collected in disturbed condition and natural density is not found out generally. Under such circumstances, this attempt has been made as such if the

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particle shapes and sizes are determined then soil strength is easily found out by knowing its SPT value.

Literature review

Garg (2008) observed that as the 'shape of the grains' is concerned; there exists no universally accepted measure of the grain shape. However, there are three general classes of grain shapes; i) Bulky grains, ii) Flaky grains & iii) Needle-shaped grains.
Bulky grains: If all the dimensions of a soil particle are about the same, as in sand and gravel, the soil grain is said to have a bulky-shape. Such soil grains are formed by the mechanical breaking of the rock masses.

i) Bulky grains: If all the dimensions of a soil particle are about the same, as in sand and gravel, the soil grain is said to have a bulky-shape. Such soil grains are formed by the mechanical breaking of the rock masses.

ii) Flaky grains: Flaky and plate like grains resemble a piece of paper, and are very thin at edges compared to their length and breadth. They have net electrical charges, on them. Sub-microscopic crystals of clay minerals usually exhibit this type of grain shape.

iii) Needle-shaped grains: Needle-like grain shape is exhibited by a special group of clay minerals, kaolinites.

When the soil formation takes place initially, these bulky grains have sharp edges and are termed angular. After a large interval of time, these edges get worn out. The effect may be considerable during transport by wind or water. Such highly smoothened particles are called well-rounded. In between these two extreme shapes of 'angular' and 'well rounded', there exist other shapes, such as sub-angular, sub-rounded, and rounded;

Beach sands and alluvial sands which are subjected to considerable activity of water, ranges from sub-angular to sub-rounded in shape; while beach gravels are rounded in shape. Wind blown sands are usually well rounded. The grain shape of coarse-grained soils can be observed under a microscope (Garg 2008).

Arora found that the angularity of particles has great influence on the behavior of coarse grained soils. The particles with a high value of angularity tend to resist the displacement, but have more tendencies for fracturing. On the other hand, the particles with low angularity (more denseness) do not crush easily under loads, but have low resistance to displacements as they have a tendency to roll. In general, the angular particles have good engineering properties, such as shear strength.

Arora also found that the angle of shearing resistance of sands in the field can be determined indirectly by conducting in-situ tests, such as the standard penetration test (SPT). The factors that affect the shear strength of cohesionless soils are; shape of particles, gradation, denseness etc.

The shearing strength of sands with angular particles having sharp edges is greater than that with rounded particles. A well graded sand exhibits greater shear strength than a uniform sand. The degree of interlocking increases with a increase in density. Consequently, the greater the denseness, the greater the strength.

Table 1. Representative values of shearing angle of sands

Soil	Shearing Angle in degree
Sand, round grains, uniform	27^0 to 34^0
Sand, angular, well-graded	33^0 to 45^0
Sandy gravels	35^0 to 50^0
Silty sand	27^0 to 34^0

It is noticeable that smaller values are for loose conditions and larger values for dense conditions (Arora 2010).

Using natural sands Dodds (2003) found that strength increases with particle size and may decrease with increasing sphericity.

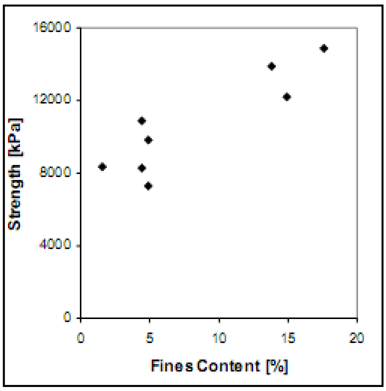


Figure 1. Strength vs. Fines content

The shape of Georgia crushed granites is size dependent. Larger grains exhibit uneven surface texture and comparatively rounded corners. Smaller grains have planar sides and angular corners.

The shape of natural sands varies markedly, from very round and spherical to angular and non-spherical. The maximum void ratio depends on the coefficient of uniformity and particle shape. Lower roundness values in crushed sands lead to higher maximum void ratios, higher critical state friction angles, lower small strain stiffness values, and increased mortar strength. Increased fines content leads to increased workability and strength in the tested sands and conditions. The value of flow is correlated with mortar strength for the sands and conditions tested. The strength of a sand exhibiting a given flow value is higher for crushed sands than for natural sands (Dodds 2003).

Rodriguez and Edeskär (2013) found that the particle shape is affected by the size of the aggregates. Aggregates in small fractions are more elongated and less rounded i. e. more angular, compared to larger fractions. Furthermore, the Aspect Ratio and Circularity seems to be the most suitable quantities to describe the tailings behavior in relation with the empirical model. The accuracy in predicting the friction angle of the tailings by previously published relations based on uniformly graded sand material is low. But the systematic underestimation of the friction angle indicates that it would be possible to establish such empirical relations based on tailings material (Rodriguez and Edeskär 2013).

Methodology

Administratically, Payra is a river of Galachipa Upazilla of Patuakhali district in South-Western Bangladesh. This is the main entrance for the beach of Kuakata. It is watered by the Bay of Bengal (Wikipedia 2015).

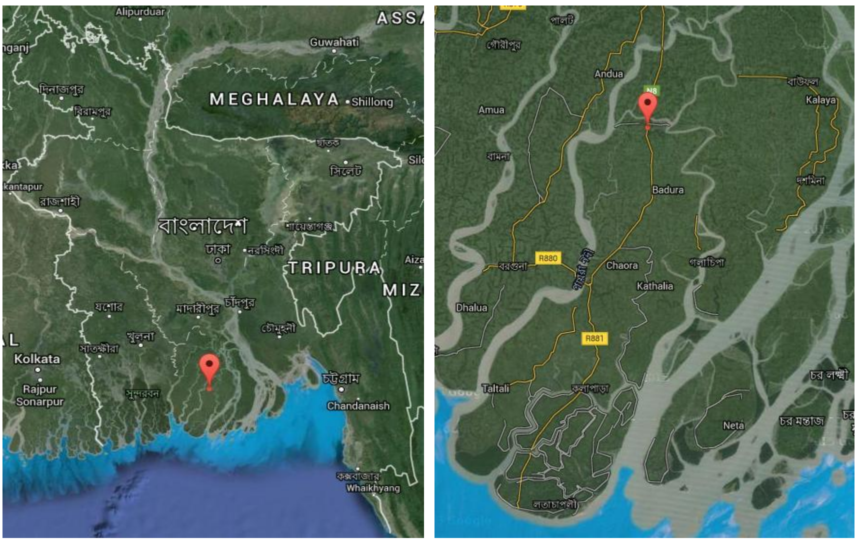


Figure 2. Map of the study area

We know that the soil is called coarse grain whose sizes are 0.02 mm to 0.42 mm. In this context, some coarse grained soil's particle shape and their simultaneous Standard Penetration Resistance for Test (SPT-values i.e. N values) are collected and grain sizes are collected from the Payra Port Project's report of soil mechanics division of Geotechnical Research Directorate of River Research Institute (RRI). Accordingly, other relevant data such as granular soil properties, SPT values, grain sizes, the image of grain shape, relative density have been collected from the same report (Soil Report 2015).

Mentioned tests are conducted through laboratory conventional method and the particle shapes are analyzed as per delivered instructions of the Payra Port Project authority, such as examining the grains under a microscope or magnifying lens and comparing them to a set standard BS14688-1which are shown below the table and figure.

Table 2. Terms for the designation of particle shape

Parameter	**Particle shape**
Angularity/roundness	Very angular Angular Sub angular Sub-rounded Rounded Well rounded
Form	Cubic Flat Elongate
Surface texture	Rough Smooth

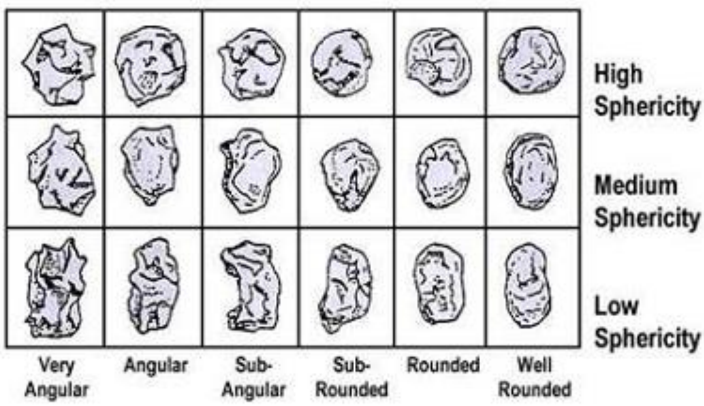
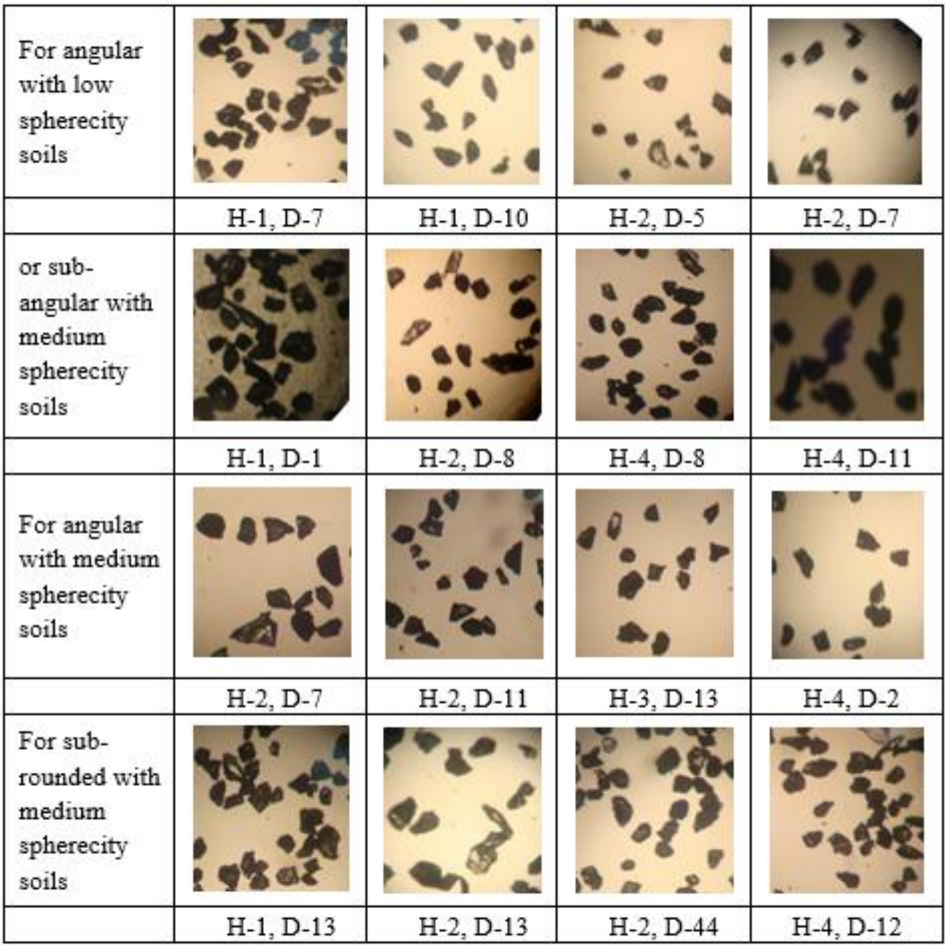


Figure 3. Particle shapes of grannular soils

The study area has been located in the map in Figure 2. For ease and notable classification particle shapes are identified according to the Table 2 and Figure 3 respectively

Results and discussion

Analysis of particle shapes of granular soils of Payra Port area have been shown in Figure 4 and a table of depth, SPT values, angle of shearing resistance ¢, density index, grain size and relative density have been presented the tabular form in Table 3.



[N.B. H-1, H-2 etc. means-Hole number and D-1, D-2 means-Sample number in disturbed condition]

Figure 4. Particle shapes of granular soils of Payra River

Table 3. Testing parameters of granular soil of Payra port area

Location	Hole No.	Sample No.	Depth in m	SPT values	Angle of shearing resistance, φ in degree	Density Index	Grain shape	Grain size in mm	Relative Density
Payra River, Galachipa, Patuakhali	1	D-1	2.1336	4	30	very loose	Sub-angular, Medium Spherecity	0.08-0.25	2.654
	1	D-7	11.2776	8	33	loose	Angular, Low Spherecity	0.074-0.25	2.657
	1	D-10	16.4592	12	35	medium dense	Angular, Low Spherecity	0.074-0.25	2.657
	1	D-13	20.4216	27	33	medium dense	Sub-Rounded, Medium Sphericity	0.08-0.25	2.656
	2	D-5	8.2296	10	34	loose	Angular, Low Spherecity	0.08-0.42	2.657
	2	D-7	11.2776	5	33	loose	Angular, Medium Spherecity	0.08-0.42	2.657
	2	D-8	12.8016	12	32	medium dense	Sub-Angular, Medium Spherecity	0.08-0.42	2.659
	2	D-11	17.3736	10	34	loose	Angular, Medium Spherecity	0.08-0.25	2.657
	2	D-13	20.4216	25	32	medium dense	Sub-rounded, Medium Spherecity	0.08-0.42	2.655
	3	D-7	11.2776	8	34	loose	Angular, Low Spherecity	0.074-0.25	2.666
	3	D-9	14.3256	13	36	medium dense	Angular, Low Spherecity	0.074-0.25	2.661
	3	D-13	20.4216	23	38	medium dense	Angular, Low Spherecity	0.074-0.25	2.658
	4	D-2	2.1336	3	33	very loose	Angular, Medium Spherecity	0.074-0.25	2.664
	4	D-3	3.6576	4	27	very loose	Sub-rounded, low Spherecity	0.08-0.25	2.656
	4	D-8	12.8016	10	30	loose	Sub-Angular, Medium Spherecity	0.074-0.25	2.661
	4	D-11	17.3736	18	35	medium dense	Sub-Angular, Medium Spherecity	0.074-0.25	2.661
	4	D-12	18.8976	22	30	medium dense	Sub-Rounded, Medium Spherecity	0.074-0.25	2.665
	5	D-2	3.6576	3	30	very loose	Sub-Angular, Low Spherecity	0.074-0.25	2.661
	5	D-5	8.2296	9	34	loose	Angular, Medium Spherecity	0.074-0.25	2.657
	6	D-11	17.3736	18	36	medium dense	Angular, Medium Spherecity	0.074-0.25	2.659

A number of graphs have been presented below such as Depth vs. SPT graph, particle shape vs. SPT graph, particle shape vs. relative density graph and SPT vs. angle of internal friction, ¢ graph etc. have been plotted in Figure 5, Figure 6, Figure 7 and Figure 8 respectively.

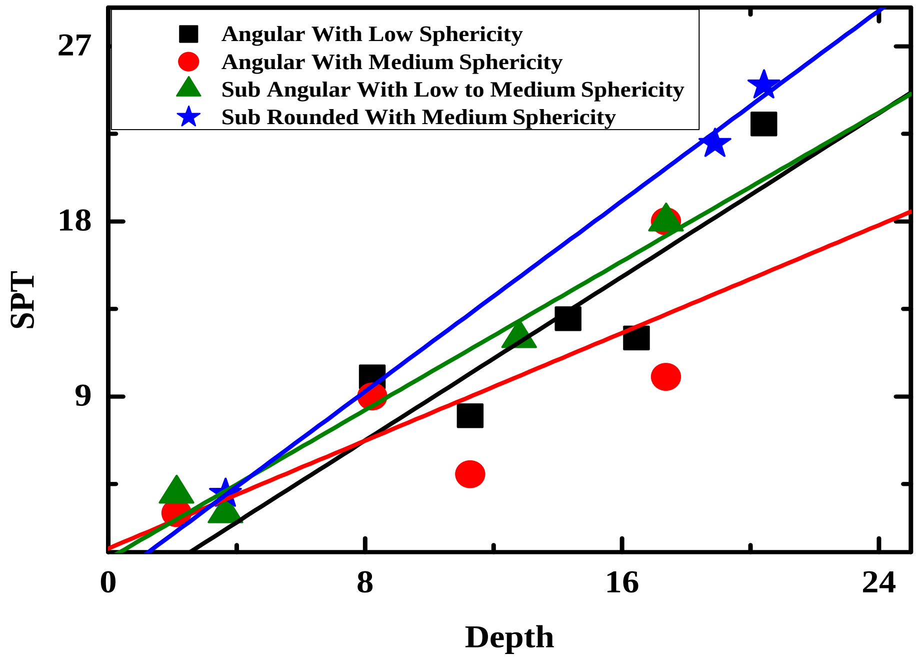


Figure 5. Depth vs SPT value curve for different shapes of granular soil

From the depth vs. SPT graph in Figure 5 it has been found that SPT values have been increases with the increases of depth. Though, the increasing trends have been observed as seen as all types of granular soils even it is found especially in case of sub-rounded with medium spherecity soil. That means the strength is more than that of other granular soils with shapes.

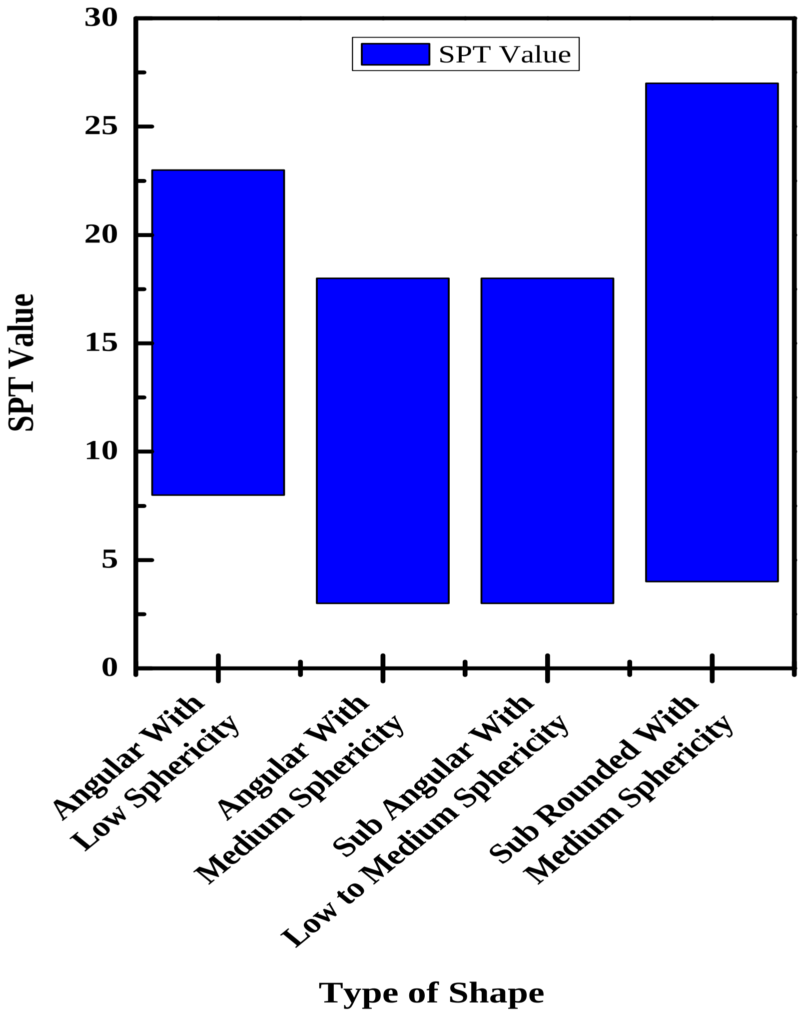


Figure 6. Range of SPT value for different shapes of granular soil.

Figure 6 is a comparison graph of SPT values of different types of granular soil. Here it is found that SPT values have been varied from 8 to 23 for angular with low spherecity soil, from 3 to 18 for angular with medium spherecity soil, from 3 to 18 for sub-angular with low to medium sphericity soil and from 4 to 27 for sub-rounded with medium spherecity soil. From the comparison graph, it is observed that SPT values of sub-rounded with medium spherecity soils are more than that of other granular soils. The increasing value of SPT indicates that its strength increases i,e. the strength of this soils is higher than that of other granular soil.

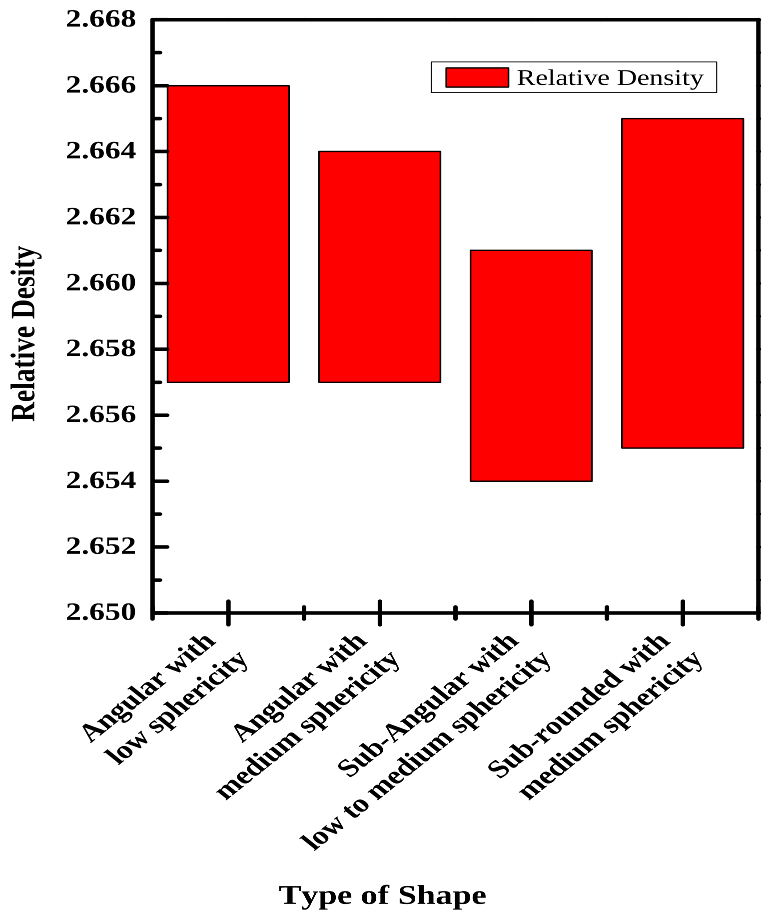


Figure 7. Range of Relative density for different shapes of granular soil

Figure 7 is a comparison graph of relative density of different type's granular soil. Here it is found that relative densities have been varied from 2.657 to 2.666 for angular with low spherecity soil, from 2.657 to 2.664 for angular with medium spherecity soil, from 2.654 to 2.661 for sub-angular with low to medium sphericity soil and from 2.655 to 2.665 for sub-rounded with medium spherecity soil. From the comparison graph, it is observed that relative densities of sub-rounded with medium spherecity soils are more than that of other granular soils which indicates their sizes are large than that of other soil. As a result, the predicted strength of these soils is higher than that of other granular soil.

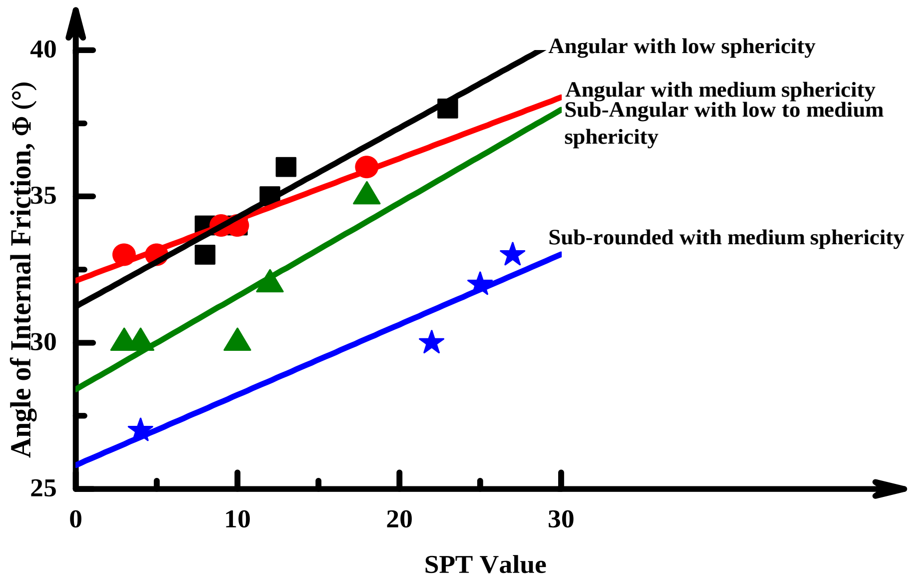


Figure 8. SPT value vs. ¢ graph for different shapes of granular soils

From the SPT vs. ¢ graph in Figure 5, it is found that angle of internal friction ¢ increases with the increase of SPT values in all types of granular soils. The increasing value of ¢ signify the increases of strength.

Conclusions

It is concluded from the present study that standard penetration resistance increases with the increase of depth in all type of granular soils. The maximum SPT value is observed 27 which indicate that the soil is in medium dense condition as well as its corresponding strength. This SPT value is noticed especially in case of sub-rounded with medium spherecity soil at depth 20.4216 m. The comparison graph also points towards the more strength for sub-rounded with medium spherecity soil as SPT values of these soils are more than that of other granular soils. The particle sizes of this soil vary from 0.08 mm to 0.25 mm and relative density varies from 2.655 to 2.665. They are the maximum values obtaining from observation in comparison with other granular soils. In the same order, the angle of shearing resistance increases for granular soils. This study may help the design engineers to know the sizes and corresponding strength according to the shape and their corresponding SPT values. In this study, a few number of granular soils have been considered for grain shape analysis and for this reason, some difficulties have been arisen to analyse the results. So, authors recommend that a further study may be needed on good number of soil samples to get good results with more accuracy.

Acknowledgement

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HYDROLOGICAL ANALYSIS AND HYDRAULIC ASSESSMENT OF THE EXISTING PAGLA-JAGANNATHPUR-AUSHKANDI ROAD AND ROAD STRUCTURES USING MIKE 21C MODEL

Pintu Kanungoe[1], Md. Johurul Islam[2], Mohammad Dulal Bawali[3], Md. Azizul Haque Podder[4], Md. Tofiquzzaman[3] and A. K. M. Ashrafuzzaman[4]

Abstract

River Research Institute (RRI) has carried out a mathematical model study to assess the adequacy of the existing Pagla-Jagannathpur-Aushkandi road and associated road structures (bridges and culverts) and to devise appropriate hydrological and hydraulic design parameters of the same together with river training works where necessary. The study is based on extensive field survey data that include cross-sections of rivers and drainage routes, topographic data, road alignment data, soil characteristics, road structure data etc. The secondary data used in the study include historical hydrological data of rivers, time series satellite images etc. A two-dimensional mathematical model has been developed covering the entire stretch of the road (29 km). The initial bathymetry of the model is formed with topographic data and cross-section data of the rivers and drainage routes that cross the road. The model boundary conditions corresponding to different return period discharges have been determined by flood frequency analysis. At some upstream boundaries where measured discharge data are not available, slope area method is used to calculate discharges corresponding to different return period floods (20 year, 50 year and 100 year). The study results show that the existing Pagla-Jagannathpur-Raniganj-Aushkandi Road alignment except at some bridge approaches is found to be suitable route under likely hydrological and hydraulic conditions. The existing top level of the road is below the design formation level in most parts of the road. The design formation level of the road is 9.75 and 9.70 mPWD at Debor Point and Raniganj respectively. At some locations the existing road runs almost parallel to and very close to the rivers. The road structures at these locations may draw substantial flow during flood condition and some of the same have already damaged due to not taking this fact into account during design. Also there is potential for occurrence of parallel flow along the approach embankments of these structures. Protective measures should be undertaken along the East side slope of the approach embankments on both sides of the bridge over the Naljur river (from chainage 23.135 km to chainage 23.264 km) and the needed extent of protective measures is 90m and 50m towards Jaganathpur and Raniganj respectively extending from the bridge abutments. New PC girder bridges may be considered at the structure locations where approach is damaged fully or partially due to flood.

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Introduction

Sunumganj district is located in the Northeast region of Bangladesh. Dhakshin Sunamganj and Jagannathpur are two upazillas under Sunamganj district. These upazillas are naturally resourceful with rice and fish cultivation. At present, there is a road communication between these upazillas and rest of the country. But the existing road faces some problems from Debor point to Ranigonj. The study area is shown in Figure 1.

There are some instances of road pavement settlement, partial damage or complete washing out of approach road, collapse of bridges due to undermining of foundation, damage of road embankment side slope, damage of approach road embankment slope protection works and damage of culverts due to scour. Also it is needed to connect these Upazillas with national road network. Under such circumstances, it is essential to improve the Pagla-Jagannathpur-Raniganj-Aushkandi road. If this road is improved to the status of regional highway, it will be easier to transport agricultural products from Dhakshin Sunamganj and Jagannathpur to other parts of the country, and people of this region will get transport facilities throughout the year.

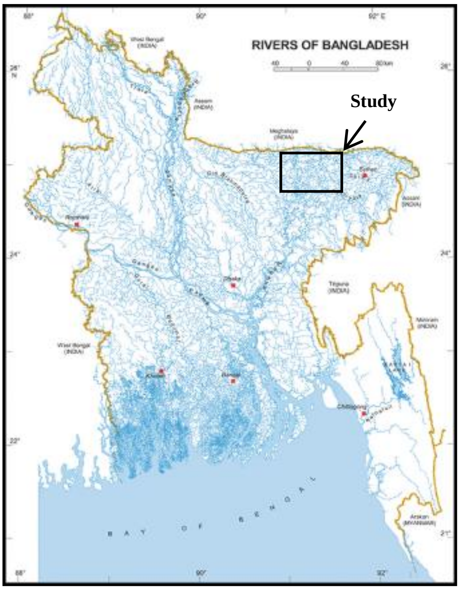

Figure 1. Location of study area

It will connect Jagannathpur Upazilla headquarter to the district town and rest of the country. It will also shorten the distance between capital city of Dhaka and Sunamganj district by 51km. Schools, madrasas and small cottage industries will be benefited. As a result, socio-economic condition of the people will improve. The main purpose of this project is to establish a direct and shorter roadway connection for Sunamganj with Dhaka, port city Chittagong and Eastern part of Bangladesh. This road will reduce traffic congestion in Sylhet city corporation area and improve overall land transport facilities of Sunamganj district along with traffic safety (RHD, Request for Proposal 2014). Road embankment along the haors creates obstruction to natural flow of water and is subjected to wave actions (BUET 2008). Also bridges often constrict the flow area under the bridge and the bridge piers enhance this constriction resulting in increase of speed, acceleration of scouring process, backwatering of stage etc. Mathematical modelling is state-of-the-art technology applicable in planning, design and improvement stages of the road embankment and road structures to ensure safe and economic design of the same. The technology can also be applied for devising suitable mitigation measures to counteract any negative impact of the road project.

The MIKE 21C tool is suited for river and floodplain hydro-morphological studies and includes modules to describe flow hydrodynamics, sediment transport, alluvial resistance, scour and deposition, bank erosion and planform changes. The modules can run interactively, incorporating feedback from variations in the alluvial resistance, bed topography and bankline geometry to the flow hydro-dynamics and sediment transport (DHI 2006).

Methodology

A field survey campaign has been carried out to collect field data necessary for model development and hydro-morphological study. Field survey includes road alignment survey from Debor Point to Raniganj, road structure survey, bathymetric survey, velocity measurement and soil sample collection. Data collected through field survey are road levels, road cross-sections, location and dimension of existing road embankment slope protection works, location and dimension of existing road structures, pier diameter, bed level near the structure, river and khal cross-sections, velocity through the road structure, soil sample from the road etc.

The pre-monsoon 2014 bathymetric survey data and DEM data have been used to form the initial bathymetry of the model. A two-dimensional model has been developed which covers around 29 km stretch of the road from Debor Point to Raniganj. A curvilinear computational grid has been generated to study different aspects of the road project within the stipulated time. The grid for hydrodynamic simulation of the model has a dimension of 100×1600. It means the length and width of the study reach are represented in the model with 100 and 1600 grid points respectively. The grid is generated incorporating an expanse of floodplain on both sides of the road.

The initial bathymetry is prepared based on the pre-monsoon 2014 bathymetric survey data collected under this study and DEM data. After completion of the bathymetric survey the data are processed. The initial bathymetry is then prepared using standard MIKE 21C bathymetry preparation module. The initial bathymetry corresponding to the generated grid is shown in Figure 2.

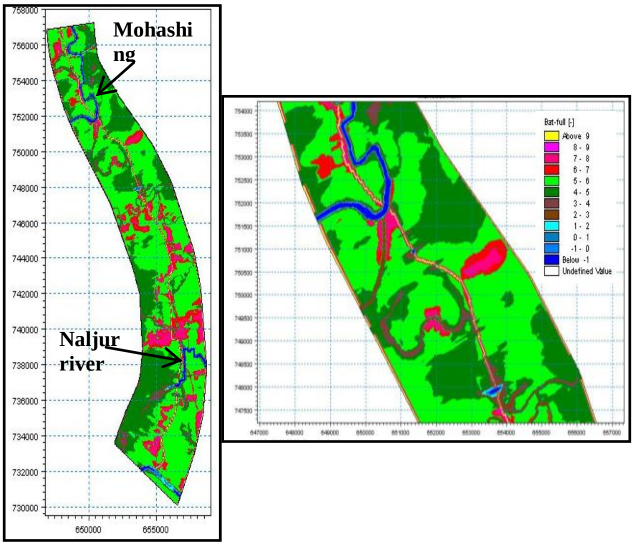

Figure 2. Initial bathymetry of the model (MIKE 21C)

There is no water level and discharge gauge station within the model domain to compare the model results with measured ones. However, it appears that the model has reproduced flow pattern in the study area for different return period discharges with reasonable accuracy.

Results and discussion

Hydraulic analysis of the road and road structures has been conducted by use of the developed two-dimensional mathematical model. Since the road and the road structures are already in place the model runs have been conducted with three different return period discharges (20 year, 50 year and 100 year) to assess the hydrodynamic response of the structures in terms of discharge and velocity through the structures, water level at and along the structures, flood depths around the structures, afflux etc. The analysis is made to assess the performance of the structures under design and extreme discharge conditions. The results of the analysis are described below:

Velocity fields

The velocity fields at and around the road and road structures for different return period discharges have been furnished in Figure 3. It is noticeable that for all return period discharges, the velocity over the floodplain is very low (< 0.2 m/s). Relatively high velocity is observed along the course of the Mohashing and the Naljur rivers. The simulated maximum velocity through the existing road structures is determined for 50 year and 100 year return period discharges (RRI 2014).

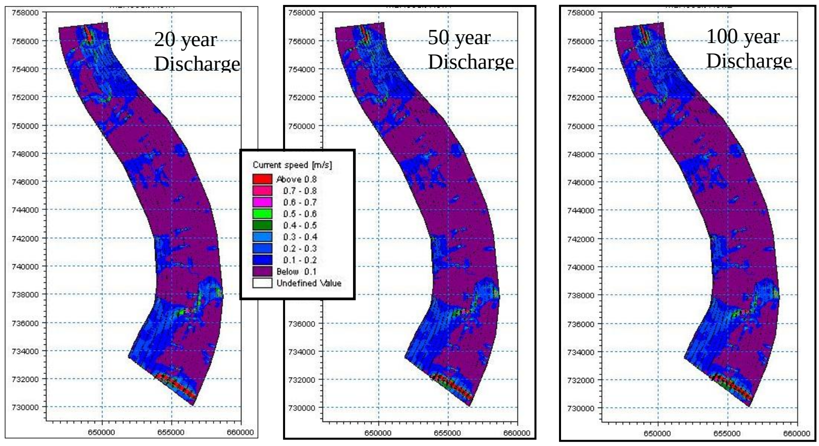

Figure 3. Velocity fields for different return period discharges at and around the Existing Pagla-Jagannathpur-Raniganj road (MIKE 21C)

The road structures should be designed for 50-year discharge. Therefore, magnitude of flow through each structure corresponding to this (50 year) discharge has been extracted from the model simulation results. It is observed that very high velocity occurs through the structures that have already experienced full or partial damage.

Water levels along the road

The two-dimensional plots of the water levels at and around the road for different return period discharges appear in Figure 4. It is evident from Figure 4 that the afflux caused by the road and road structures is not high and varies from 3cm to 7cm for different return period discharges. It is noticeable from Figure 4 that there is not much variation in the water levels along the road from Debor Point to Raniganj. There is a mild water level slope from Debor Point to Raniganj with relatively higher water level at Debor Point. In the floodplain slight local variations in the water level are noticeable. The overall water level slope in the floodplain is From North-East to South-West (RRI 2014).

Flood depths

The road runs through the low-lying and haor areas. These areas go under water during flood season. There are scattered villages and bazaars connected by unmetalled roads with the road under study. The source of inundation water is principally the overbank spillage of Kalni-Kushiyara, Surma, Mohashing and Naljur rivers. Flood depths at and around the road for different return period discharges are shown in Figure 5.

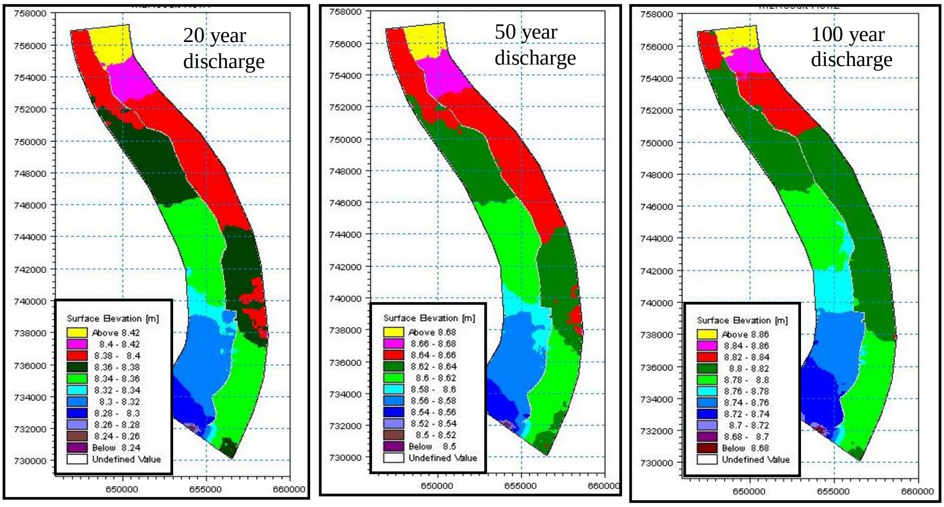

Figure 4. Simulated two-dimensional plots of water level at and around the Pagla Jagannathpur-Raniganj road for different return period discharges

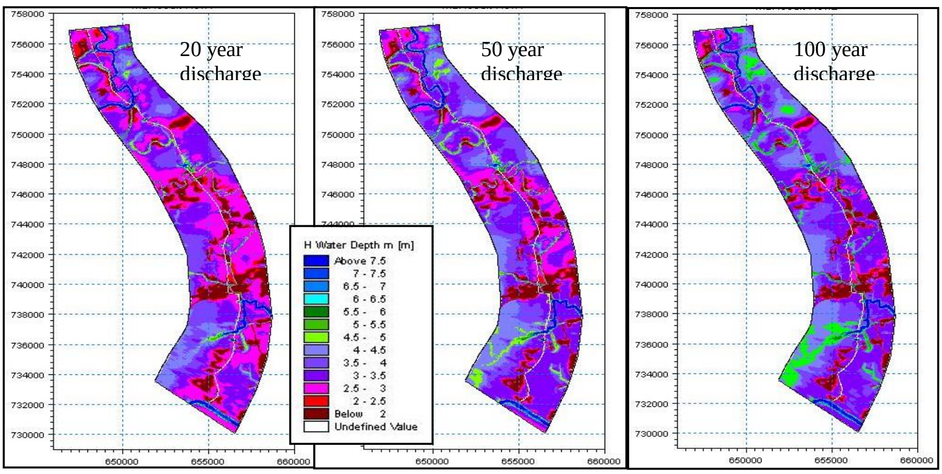

Figure 5. Simulated two-dimensional plots of water depth at and around the Pagla-Jagannathpur-Raniganj road for different return period discharges

Slope protection works

During extreme flood the low areas around the road experiences average flood depth more than 3 m with low flow velocity. Along the existing road there are a number of road structures (culverts and bridges) to allow for safe passage of floodwater. The structures have been constructed over the drainage routes that cross the road. No hydro-morphological study has been conducted to decide about hydrologic and hydraulic design parameters of these structures. As a result there is occurrence of parallel flow along the approach embankment at some structure locations. The slope of the road embankment should be protected against likely damage by parallel flow current at these locations. On the other hand, there is potential for road embankment slope damage due to wave actions at some locations. Therefore, appropriate measures should be taken against such slope damage. The vulnerable locations of the road embankment slope damage have been identified under the framework of this study. The hydrologic and hydraulic design parameters of the slope protection works have been furnished (RRI 2014). The identified locations where slope protection works will be needed are shown in Figure 6.

Figure 6. Identified locations for road embankment slope protection works (Source: Google earth 2014)

Hydrological and hydraulic design of road and road structures

For the design of road and road structures the following design data are used:

Formation level of the road	: 9.70 to 9.75 mPWD
Standard High Water Level (Markuli)	: 9.75 mPWD
Standard Low Water Level (Markuli)	: 1.60 mPWD
Velocity	: 0.5-2.50 m/s
D_{50} of silt	: 0.08 mm

For the design of slope protection works the following design data are used:

Design flood level	: 8.61to 8.66 mPWD
Velocity	: 0.8 m/s
D_{50} of silt	: 0.08 mm
Depth of flow	: 3 to 4 m
Wind speed	: 30 m/s
Fetch length	: 4 km
Wind duration	: 2 hours
Wave height	: 1.35 m
Wave period	: 3.5 sec
Wave runup	: 1.08 m

Navigational clearances

The rivers and khals in the study area do not fall under any classified navigational route by BIWTA. No minimum vertical and horizontal clearance is specified either by BIWTA or RHD. In determining appropriate navigational clearance local requirements for the passage of fishing boats, market boats, coal or stone barges etc. should be taken into account. Navigational status of different rivers and khals that crosses the road is taken by discussing the local people.

Conclusion and recommendations

The following conclusions and recommendations have been drawn based on study results.

- The existing Pagla-Jagannathpur-Raniganj-Aushkandi Road alignment except at some bridge approaches is found to be suitable route under likely hydrological and hydraulic conditions. The afflux caused by the road and road structures is not high and varies from 3 cm to 7 cm for different return period discharges.

- At some locations the existing road runs almost parallel to and very close to the rivers. The road structures at these locations may draw substantial flow during flood condition. Also there is potential for occurrence of parallel flow along the approach embankments of these structures. Approach road slope protection measures should be undertaken there.

- The road embankment may come under wave action at some locations. The wave runup is 1.08 m. The model simulated flow velocity through the different existing structures varies from 0.5 m/s to 2.5 m/s. The highest velocity is observed at Jagannathpur regulator. The actual velocity through this regulator could be well above 3.0 m/s for flood discharges because the regulator could not be reproduced in the model to its actual dimension.

- Road embankment slope protection works should be undertaken against occurrence of parallel flow along the road embankment and wave action.

- The existing navigation clearance for the P.C girder bridge over the Mohashing river is found to be 2.45 mPWD. The observed minimum bed level under the bridge is found to be about -10.42 mPWD. In this case, CC blocks/geo-bags should be kept ready for emergency dumping, if necessary.

- The approach road slope of the bridge over the Mohashing river at Jagannathpur end should be protected against parallel current and wave action. The required length of the protective measure is 200m on both sides of the road.

- A 52m long bridge with two spans may be considered at Chainage 18.176km in place of the existing culvert. The design flood level and height of the bridge are 8.63 mPWD and 9.87 mPWD respectively. The deck level of the bridge is 11.97 bmPWD. The Design scour level around the pier is suggested to be -1.0 mPWD. The bottom level of the pile foundation should be set well below this level.

- The extended part (Bailey bridges at both ends) of the P.C Girder Bridge at Vomvomi (Chainage at 7.064 km) will have to be closed by compacted earth filling. In order to safe passage of flow through the existing bridge, both sides protective works in the form of retaining wall will have to be constructed. The gaps between the existing road and abutment wall along with proposed retaining wall will have to be developed as a road embankment at both sides by compacted earth filling. Since the velocity has been increased 1.3 times with permanent bridge opening that may develop more scour at and around the bridge sub-structure. To overcome this situation the existing bed level (-7.04

mPWD) will have to be filled by dredged soil up to G.L. Then the protective works should be placed according to the design furnished.

- The existing length of the bridge over the Naljur river (from chainage 23.135 km to chainage 23.264 km) is shorter than the effective width of waterway according to the Lacey's formula (177 m). 20 year and 50 year return period flood level at the bridge location is 8.35 mPWD and 8.61 mPWD respectively. The observed minimum bed level along the centre line of the bridge is found to be -7.7 mPWD. The required height of the bridge is 10.35 mPWD. The deck level at the centre of the bridge is 12.45 mPWD.

- Protective measures should be undertaken along the east side slope of the approach embankments on both sides of the bridge over the Naljur river (from chainage 23.135 km to chainage 23.264 km). The needed extent of protective measures is 90m and 50m towards Jaganathpur and Raniganj respectively extending from the bridge abutments.

- The likely magnitude of discharge through the bridges and culverts in the south of the bridge over the Naljur river under design flood condition is substantially high compared to the existing opening of these structures. The bridges are also narrow in width. These structures should be replaced by PC girder bridges with adequate opening to accommodate for the likely design flood discharge.

- The existing damaged approach culvert at Chainage 18+176 km may be replaced by PC girder bridges. New PC girder bridges may be considered at the structure locations where approach is damaged fully or partially due to flood. The suggested hydrologic and hydraulic design parameters may be considered for construction of these bridges.

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NUMERICAL MODELLING USING MIKE21C FOR THE PROPOSED BRIDGE ON KALNI RIVER UNDER HABIGANJ ROAD DIVISION

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Abstract

River Research Institute (RRI) has conducted a mathematical model study to determine the suitable location of proposed bridge along with alignment of approach road and to provide the hydraulic and hydrological design parameters of the same including river training works. The study is based on the historical hydrological data, time series satellite images, recent bathymetry, bankline and sediment data. The collected data have been analysed using appropriate methods to derive information about present physical conditions of the river, past and future trends in morphological development and necessary inputs for development of MIKE 21C model. The model is developed covering a river stretch of 15km including the likely bridge location. The study results show that the proposed bridge over the Kalni river should be located in the straight reach at Sullah Ghat. The length of the bridge is 340 m. The left end and right end co-ordinate of the bridge is 627988E, 718823.5N and 627827E, 719123N respectively. Six spans of the bridge may be considered with two spans of equal length (60 m) in the middle. The rest four spans (two in the left side and two in the right side) are of 55m each. The design discharge for the bridge and bridge substructure is 3468 m^3/s and 3667 m^3/s respectively. The design water level for the bridge and bridge substructure is 8.89 mPWD and 9.08 mPWD respectively. The wave runup is 1.41 m considering road embankment slope of 1:2. The bridge deck level at centreline of the bridge is 23.75 mPWD. The design scour level at the abutment is -4.13 mPWD. The bottom level of the bridge girder should be kept at 20.75 mPWD. The bottom level of pile foundation for the abutment should be placed well below this level. The design scour level for the bridge pier is suggested to be -14.35 mPWD. The bottom level of the pile foundation should be set well below this level. The thalweg profile in the vicinity of the proposed bridge shows the potential for large bed degradation at the meander bends in the upstream of the bridge location. The minimum bed level at the bridge location is -3.24 mPWD whereas the minimum bed level at these bend locations is as low as -24.57 mPWD. The approach road formation level at access road is 10.3 mPWD.

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Introduction

Sunamganj and Habiganj districts are located in the North-East region of Bangladesh. Ajmiriganj and Sulla are two remote upazillas under Habiganj and Sunamganj districts respectively that are situated in haor area. Sunamganj-Madanpur-Derai and Habiganj-Baniachong are already constructed part of Roads and Highways Department (RHD) road network. Madanpur-Derai-Sullah (Derai-Sullah part) road and Baniachong-Ajmiriganj road are two ongoing Annual Development Programme (ADP) projects of Government of Bangladesh (GoB).

Now, the complementary road communication between Sullah and Ajmiriganj is imperative in order to have a fast and easy road communication with the capital city Dhaka and other parts of the country via Habiganj for the people of Sullah as well as Sunamganj district. In order to connect Sunamganj and Habiganj districts and to enhance the communication facilities, Road Division, RHD, Habiganj has planned and taken initiatives to construct a roadway bridge over the Kalni river.

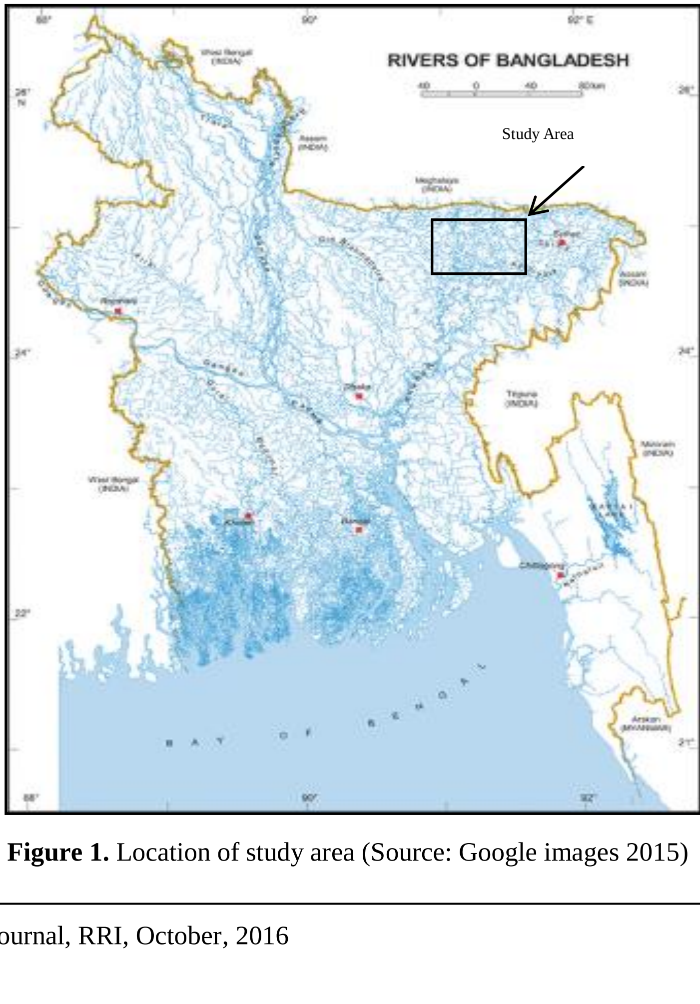

Figure 1. Location of study area (Source: Google images 2015)

The project area (Figure 1) is hydrologically affected by the influence of the Meghalaya Foothills and Shillong Plateau on the north of India. The proposed bridge site is located in the Kalni river catchments having overland flow, interflow and base flow of varying degrees. As such, there are several reasons why it is important to consider likely hydrological, hydraulic and morphological impacts when planning to construct a bridge or any water resources development intervention in this area or over a river.

Rivers of this region are very dynamic in nature and are changing and evolving. Periodic human interference from past to present can initiate major adjustments to the river system. Failure to recognize these ongoing changes may lead to implementation of inappropriate projects that operate poorly in future.

Moreover, river dynamics and channel changes or shifting can impose a degree of uncertainty in planning future water resources developments. In view of the above mentioned facts a number of hydrological and morphological aspects related to the proposed bridge seem to be crucial for investigation. In this connection, a comprehensive study is needed using mathematical modelling technology to address all the relevant aspects of the bridge project and suggest appropriate measures to be taken to ensure its overall sustainability. On the other hand, since the proposed road and bridge will be constructed in a low lying area that goes regularly under water the project may change the existing hydrologic regime substantially if not properly planned and designed (RHD, Request for Proposal 2012).

Methodology

In order to conduct the study necessary hydrological data of the Kalni river, satellite image of the study area and other relevant information have been collected. A field survey campaign has been conducted to collect the recent bathymetric data of the river, nearby road alignment data etc. Sub-soil investigations have also been conducted at four locations at and around the proposed bridge. In order to conduct EIA and EMP study environmental data from the project area and relevant agencies have been collected.

The collected data have been processed and analysed to the extent of gaining understanding of the present physical conditions of the river at the bridge location and environmental conditions in the project area and also deriving information to use as model inputs. A two-dimensional model covering an extent of about 15 km of the Kalni river has been developed using modelling software MIKE 21C. The initial bathymetry of the model is formed by use of the recently surveyed bathymetric data. The initial bathymetry of the model is shown in Figure 2.

Necessary hydrological, hydrographic and sediment data have been collected through a field survey campaign. Historical hydrological data of the rivers concerned and satellite images of the study area have been collected from Water Resources Planning Organization (WARPO), Dhaka and Centre for Environmental and Geographic Information Services (CEGIS), Dhaka respectively. The collected data have been processed and analysed to the extent of deriving necessary inputs for the MIKE 21C model that has been developed for hydraulic analysis of bridge and approach road and also for obtaining other information relevant to the bridge project. The model has been calibrated for monsoon 2005 hydrological data. The calibrated model is then applied for different scenarios simulations in base and with bridge conditions.

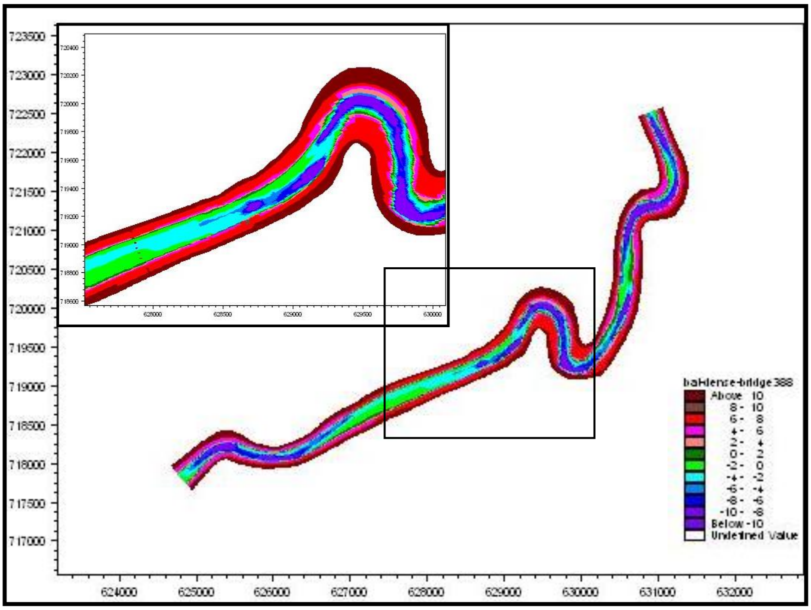

Figure 2. Initial bathymetry of the model (MIKE 21C)

Results and discussion

Suitable reach for bridge location

It appears from the analysis of collected hydrological and hydrometric data of the river and also satellite image of the study area that the straight reach near the Pirijpur Ghat and Sullah Ghat is almost stable in terms of lateral migration and bed degradation. The favourable river stretch for siting of the bridge is shown in Figure 3. Four alternative bridge locations have been selected in this reach for hydrodynamic and morphological assessments to decide about suitable bridge location. The bend in the downstream of the straight reach is still mild and has potential for future development. On the other hand, two consecutive meander bends in the immediate upstream of the Pirijpur Ghat are sharp and increasing in meander ratio slowly due to resistance offered by the cohesive bank materials. The river at these bend locations is unstable and yet to achieve its dynamic equilibrium. Since the disturbances caused by the river instabilities at the upstream bend locations may travel downstream, the bridge location should be safely away from these bends.

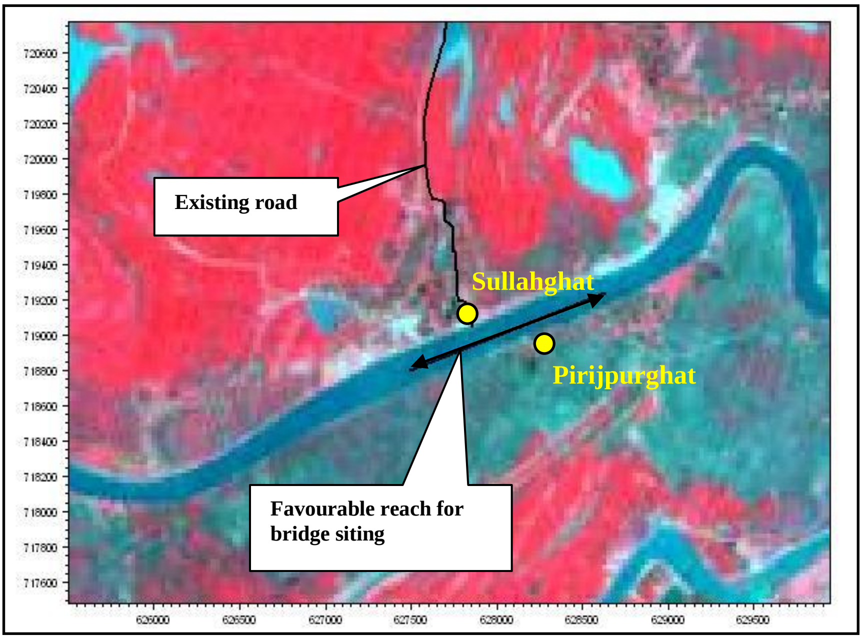

Figure 3. Favourable river stretch for siting of the bridge (Source: Satellite image 2014, CEGIS)

Proposed bridge alignment and velocity field

The hydrodynamic simulations of different return period discharges show similar velocity distribution pattern along the cross-sections with maximum velocity being almost in the middle of the river. After taking into account different relevant issues, the bridge location is selected at Sullah Ghat. The alignment of the bridge is shown in Figure 4. The design discharge for the bridge has been estimated as 3468 m^3/s from flood frequency analysis and based on the design discharge and other relevant issues in view; appropriate length (340 m) for the bridge has been determined. The model simulations with different return period discharges have been conducted with bridge in place to see the effects of bridge constriction on existing hydraulics at and around the bridge. It is found from the simulation results that the bridge causes local increase in velocity around the bridge piers but has negligible effects on the water level upstream compared to the base condition. It means that with the selected bridge opening the free passage of flood flow will not be hampered. The velocity field in the vicinity of the bridge for 100-year discharge and with bridge in place is shown in Figure 5. The bridge is introduced in the model as accurately as possible as per decision as to number of spans, pier width etc.

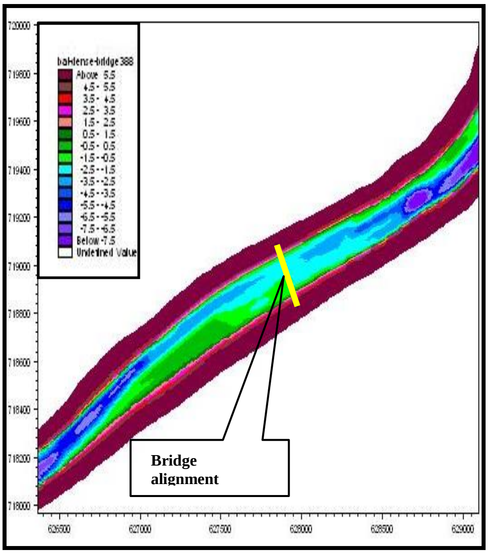

Figure 4. Proposed alignment of the bridge over the Kalni river at SullahGhat (MIKE 21C)

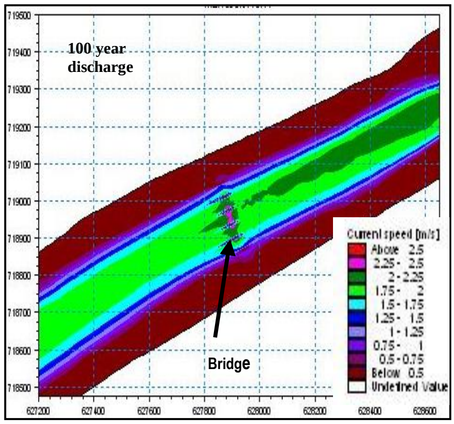

Figure 5. Velocity field at and around the bridge for 100 year discharge (MIKE 21C)

Hydrological and hydraulic design of bridge and approach road

The hydrological and hydraulic design parameters of the bridge and approach road obtained from the study are given below:

Design discharge for bridge substructure (100 yr. return period)	: 3667 m^3/s
Discharge for bridge and approach road (50 yr. return period)	: 3468 m^3/s
Design flood level for bridge substructure (100 yr. return period)	: 9.08 mPWD
Design flood level for bridge (50 yr. return period)	: 8.89 mPWD
Standard Low Water Level	: 1.11 mPWD
Standard High Water Level	: 8.55 mPWD
D_{50} of silt	: 0.08 mm
Significant wave height	: 1.25 m
Wave period	: 4s
Wave runup	: 1.41 m
Formation level of approach road (Sub grade level)	: 10.3 mPWD
Bottom level of the bridge girder	: 20.75 mPWD
Deck level at centreline of the bridge	: 23.75 mPWD
Design scour level for abutment	: -4.13 mPWD
Design scour level for pier	: -14.35 mPWD

Velocity information at and in the vicinity of the Bridge

Velocity information at and in the vicinity of the bridge location in base and with bridge conditions is shown respectively in Table 1 and Table 2 below.

Table 1. Velocity information at bridge location in base condition

Return Period (year)	Discharge (m^3/s)	Maximum velocity (m/s)	Cross-sectional mean velocity (m/s)
25	3259	1.87	1.17
50	3468	1.95	1.22
100	3667	2.05	1.23

Table 2. Velocity information with bridge in place and for 100-year discharge

Location	Maximum velocity (m/s)	Near bank velocity (m/s)
Along left bank in the immediate upstream and downstream of the bridge	-	1.2 to 1.5
Near left abutment	0.80	-
Near right abutment	0.50	-
Along right bank in the immediate upstream and downstream of the bridge	-	1.1 to 1.5
At the second pier from the left	2.20	-
At the third pier from the left	2.40	-
At the fourth pier from the left	2.44	-

Bridge height and span arrangements

According to Bangladesh Inland Water Transport Authority (BIWTA) navigational route classification the proposed bridge area falls under Class II navigational routes (Zakiganj-Fenchuganj-Ajmiriganj-Dilalpur transit route). It means minimum vertical clearance should be 12.20 m with reference to Standard High Water Level (SHWL). The SHWL is found to be 8.55 mPWD. The bottom level of the bridge girder in this case is the summation of Standard High Water Level and minimum vertical clearance as specified by BIWTA. The bottom level of girder is thus 20.75 mPWD. The minimum horizontal clearance for Class II navigation route is 76.22 m. The selected bridge length is 340 m. It can be seen that the total bridge length is divided into 6 (six) spans with two spans of equal length (60 m) in the middle. The rest four spans (two in the left side and two in the right side) are of 55 m each.

Need for river training works

At the proposed bridge location and immediately upstream and downstream of it, the river is flowing almost in straight alignment. However, upstream of this straight reach there are two consecutive sharp bends. These bends do migrate slowly due to resistance offered by the cohesive bank materials against bank erosion. From local people's experience and interview and mathematical model run it seems that the bank erosion in this reach is very negligible. The minimum bed level at the proposed bridge location is -3.24 mPWD. However, large bed degradation is observed at the upstream bend locations. The minimum recorded bed level there is as low as -24.57 mPWD. As per subsoil investigation report the composition of left bank soil is silty clay and clayey silt and the bank is very less erodible. So, it may be opined that since the banks of the Kalni river at the bridge site is stable, there may not be any bank protection works and guide bund required at present to guide the flow.

Alignment and road structures along the link road

The appropriate alignment of the link road connecting the proposed bridge over the Kalni river with existing Baniachong-Ajmiriganj road has been determined considering relevant issues.

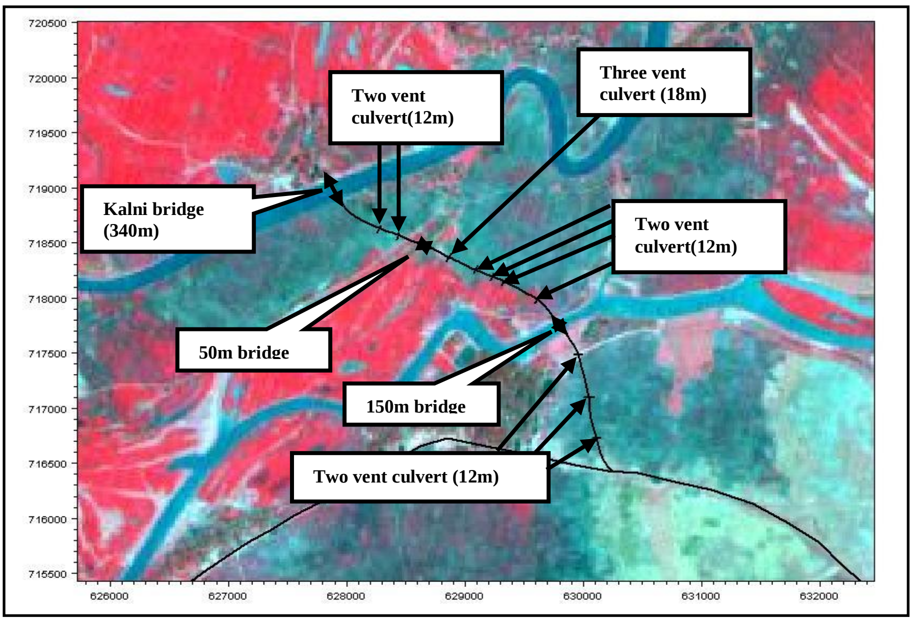

Figure 6. Positions of bridges and culverts along the link road (Source: Satellite image, 2014 CEGIS)

The link road lies on an active floodplain of the Kalni river. In order to ensure safe passage of floodwater during an extreme event, road structures at different positions along this road have been proposed. The type, position and length of the road structures are shown in Figure 6. It can be seen from Figure 6 that one 150 m long bridge, one 50 m long bridge, nine two vent (12 m) culverts and one three vent (18 m) culvert will be needed at different locations along the link road (access road).

Approach road slope protection works

The abutments and slopes of the approach embankments should be protected from erosion caused by parallel current and wave action. The approach road is 320 m long.

Conclusions

The following conclusions have been drawn based on study results:

- The proposed bridge over the Kalni river should be located in the straight reach at Sullahghat.

- In the study reach the river flows through Surma-Kushiyara floodplain and Meghna estuarine floodplain physiographic units. The soils of these regions consist mainly of loam and clay and thereby, are resistant to bank erosion. Bank erosion does occur there but at a slow pace.

- Analysis of satellite images and recent cross-section data and model results show that there is no lateral stability problem at the proposed bridge location. There are defined banks, well grown trees and human settlements on both sides of the river at the proposed bridge location. It is unlikely that the bridge could be outflanked during its design life.

- The outer bank of the mild bend downstream of the proposed bridge location may develop further slowly.

- The thalweg profile in the vicinity of the proposed bridge shows the potential for large bed degradation at the meander bends in the upstream of the bridge location. The minimum bed level at the bridge location is -3.24 mPWD whereas the minimum bed levels at these bend locations is as low as -24.57 mPWD.

- Due to unique planform of the meander bends in the immediate upstream of bridge, complex flow occurs there and the disturbance travels downstream of the bend. The river is yet to reach its dynamic equilibrium at these bends locations.

- The length of the bridge is 340m. Left end co-ordinate of the bridge is 627988E, 718823.5N and right end co-ordinates of the same is 627827E, 719123N.

- The design discharge for the bridge and bridge substructure is 3468 m^3/s and 3667 m^3/s respectively. The design water level for the bridge and bridge substructure is 8.89 mPWD and 9.08 mPWD respectively.

- The standard high water level is 8.55 mPWD. The wave runup is 1.41m considering road embankment slope of 1:2. The approach road formation level at access road is 10.3 mPWD. The bottom level of the bridge girder should be kept at 20.75 mPWD. The bridge deck level at centreline of the bridge is 23.75 mPWD.

- Six spans of the bridge may be considered with two spans of equal length (60 m) in the middle. The rest four spans (two in the left side and two in the right side) are of 55 m each;

- The design scour level at the abutment is -4.13 mPWD. The bottom level of pile foundation for the abutment should be placed well below this level. The design scour level for the bridge pier is suggested to be -14.35 mPWD. The bottom level of the pile foundation should be set well below this level.

- The meander bend upstream of the proposed bridge may undergo both lateral and down valley migration at a slow pace and it may influence the flow pattern in the downstream straight reach. The evolution of the outer banks at the consecutive sharp bends in the immediate upstream of the straight reach should be monitored closely.

- There is no immediate threat of lateral bank migration at the proposed bridge location. The abutments and slopes of the approach embankment should be protected against current and wave action. The length of the left approach embankment is 320 m.

- The suitable alignment of the link road connecting the bridge and the existing Baniachong-Ajmiriganj road has been furnished. The length of this link road is 3650 m. Total volume of earthwork for approach road on both sides of the bridge is 106700 m^3.

- There should be road structures along the link road to allow for smooth drainage during flood season. In order to allow for safe passage of flood water during an extreme event (1 in 100 year) one 150 m long bridge, one 50m long bridge, nine two vent (12 m) culverts and one three vent (18 m) culvert will be needed at suggested positions.

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ESTIMATING URBAN FLOOD HAZARD ZONES USING SWMM IN CHITTAGONG CITY

Aysha Akter[1] and Ahad Hasan Tanim[2]

Abstract

Identifying urban flood-prone areas with their relative hazard scale is the fundamental work of urban flood management. This may introduce using a probabilistic methodology by flood hazard zoning of potential urban flood hazard area. Based on statistical analysis of US EPA Storm Water Management Model (SWMM) with a threshold value of maximum depth of each node of link network number of overflow events in a specified time period can be obtained. Further using Kernel hazard density the spatial analysis in ArcGIS can be used to obtain a GIS compatible maps for the hazard zoning of the potentially flood prone areas. In this study Chittagong city, the second largest city of Bangladesh has been taken as a case study and the wettest year 2014 was selected for model simulation. The validated model outcome reasonably identified the flood prone vulnerable zones which is comparable to the outcome of recent field studies. Thus, it is expected that the acquired flood hazard mapping will play major role once this is observed using details field data. Finally, this would provide flood risk information to the decision makers and flood protection works to prioritize the relatively more flood hazard zone for management purpose.

Introduction

Chittagong city area has experienced the highest number of flood incidence in last decades. Social environment, local economy and ecology have been hampered and degraded due to prolonged urban flooding. During the flood about 7 million city dwellers face severe disruption in their daily life (CWASA 2015). The overflow in the drainage system frequently occurred due to combined effect of heavy rainfall as well as tidal effects. Attempts have been made by frequent dredging of the drainage systems without any comprehensive study of flood. As a result overall flood protection works fail to offer the expected supports. More than 2.73 Billion Bangladeshi Taka has already been employed by the Chittagong Development Authority (CDA) during last decades but the overall flood protection work faces difficulties to provide permanent protection to the adjacent city dwellers. Including Chittagong Water Supply and Sewerage Authority (CWASA) project based study, there were few research studies identified the key issues, i.e., heavy rainfall, tidal effect of adjacent Karnafuli River, intervention in sewerage system due to solid waste blockage as well as climate change (CWASA2015; Table 1). Thus, while analyzing the flood risk hazard those factors must be considered in addition the idiosyncratic topography of the area.

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Table 1. Research study summary on urban flooding in Chittagong City area

References	Causes	Identification method	Limitations
Mohit and Akter (2013)	Climate change, solid waste blockage	Field survey	Proper identification of water logged hotspot, extent and depth
Mohit et al. (2014)	Heavy rainfall	Hydrological modeling	Water logging extent and surcharged location in drainage system
Ahmed et al. (2014)	Heavy rainfall, tidal effect, urbanization	Field survey	Tidal effect and Hydraulic data scarcity of modeling study
Tanim and Akter (2015)	Heavy rainfall	Numerical modeling	Tidal effect
Tanim et al. (2015)	Heavy rainfall	Hydrological modeling	HEC-HMS inability to simulate tidal effect
Akter and Tanim (2015)	Tidal effect	Numerical modeling	Hotspot of urban flooding

With the advent of computer based numerical modeling, flood hazard mapping can be achieved with reasonable accuracy. So far a number of factors have to be considered while studying flood hazard based on topography. Elkhrachy (2015) considered some causative factors while generated flood hazard mapping those are runoff, soil type, surface slope, surface roughness, drainage density, distance to main channel and land use. Mastin (2009) proposed a watershed model for flood management that can simulate several factors such as influence of frozen ground on peak discharge, evaporation and groundwater flow, channel losses. Špitalar et al. (2014) carried out a probabilistic analysis based on the human impacts on the flash flood that relies on the computation of probability distributions by occurrence (PDF) and the PDF of occurrences weighted by the number of injuries and fatalities for different spatial, temporal, and hydrologic parameters. Bhatt et al.(2010) applied satellite based observations for alluvial plains of North Bihar (India) that are most vulnerable to flood hazards. Recently, GIS and other modern spatial techniques such as HEC-GeoRAS offer capabilities to extract drainage networks and basins that have potential to influence accumulation of runoff. To identify the nodes that are most vulnerable can be integrated using GIS (Youssef and Pradhan 2011).

Cherqui et al. (2015) first analyzed the combined sewerage system at a strategic scale (based on the intervention in combined sewerage system) of the whole territory, then analyzed specific areas which potential flooding is identified using Kernel density function. Understanding of past long-term hydrologic variability required to anticipate the effects of potential climate change impact (natural or anthropogenic) on hydrology. Such probabilistic estimation is a critical need for extreme floods for better understanding of flood processes, flood-hazard mitigation and flood risk assessments. This paleohydrologic information provides valuable information based on which flood hazard analysis can be performed. Probabilistic estimation of flood frequency can be obtained from the statistical analysis of overflow events. On the other hand US EPA Storm Water Management Model (SWMM) is Federal Emergency Management Agency (FEMA) approved computer program that computes dynamic rainfall-runoff for flood analysis single event and long-term (continuous or period-of-record) runoff quantity and quality from developed urban and undeveloped or rural areas. FEMA specified following steps must be followed while modeling with SWMM (e.g. this procedure assumes that the floodplain areas beyond the encroachment stations will be completely filled).

- SWMM can consider the loss of floodplain storage and the loss of conveyance, while steady flow models can consider only the loss of conveyance in computing floodways. However, unsteady flow programs do not have an option to determine floodways automatically to account for the loss of floodplain storage and conveyance. Therefore, the encroachment stations for the floodway must be determined first from a steady flow model, such as HEC-2, using the equal conveyance reduction method;
- The HEC-2 format is recommended when specifying data for the natural cross sections in the SWMM model; and
- The maximum computed flow at the cross sections (conduits) and the corresponding time of occurrence is obtained from the Conduit Summary Statistics table for the 1% annual chance flood.

For this reported study following the above mentioned issues, the statistical analysis of SWMM on a particular node was carried out and supposed to provide such sorts of information which can be incorporated with Kernel flood hazard density to identify potential flooding zone.

Methodology

Study area

The study area (Figure 1) was considered to be representative of urban flood prone area, which is located adjacent to the bank of Karnafuli River. These small catchments (44.19 km^2) originate in Chittagong district in the southern range of country having tertiary maritime deposit and reach the sea forming a coastal river estuary transition zone. This means that the alluvial fan of flood plains have combination of free flash flooding and coastal flooding of adjacent river. Historically human occupation of the city area has changed the land use pattern that reveals increased urbanization is one of the impact factors of flood occurrence. The physical properties of study area are shown in Table 2. The soil formation in the study area varies from sandy to silty type. The studied basin lied in tropical climate with mean annual rainfall 3000mm. There are 3 major primary drains viz. Mohesh Khal, Khal no 18 and a major parts of Chaktai Khal (CDA 1995) those are connected to a numbers of secondary drainage systems. Finally, there are 8 outfalls those are disposing in the Karnafuli River. These outfalls are located in the estuary.

Table 2. Physical properties of sub-catchments

Sub-catchment ID	Area(Hectors)	Width(m)	% slope	Imperviousness (%)
S1	363.09	1780	4.14	62.99
S2	176.51	1154	11.59	59.64
S3	409.9	1602	5.52	4.52
S4	153.47	775	7.12	84.39
S5	331.24	1409	4.14	64.21
S6	320.05	3170	7.25	83.4
S7	363.24	1628	3.39	32.05
S8	158.94	1431	8.17	41.02
S9	233.49	2501	13.55	17.97
S10	177.69	1112	16.44	0.46
S11	355.25	1822	3.69	48.93
S12	146.24	995	3.44	60.34
S13	191.8	1222	6.45	48.91
S14	158.14	1839	2.56	38.32
S15	314.63	1701	3.98	13.05
S16	274.63	1789	8.82	71.71
S17	190.16	1584	4.61	63.26
S18	100.1	475	5.1	74.93

Data preparation

The topography information was acquired from the Shuttle Radar Topography Mission (SRTM) of 30 m Digital Elevation Model (DEM). The link network (drainage network) was developed using Chittagong Development Authority (CDA) vector data. The SCS-CN method was used to determine the infiltration amount for the given precipitation data. Meteorological data, i.e. precipitation, evapotranspiration, wind speed, radiation and temperature, for the catchment area were obtained from Bangladesh Meteorological Department (BMD) for the year 2014. Thus, the obtained meteorological data were further processed to obtain evapotranspiration. The land slopes of the sub-catchments were obtained using spatial analysis of Arc-GISv9.3 (Figure 2).

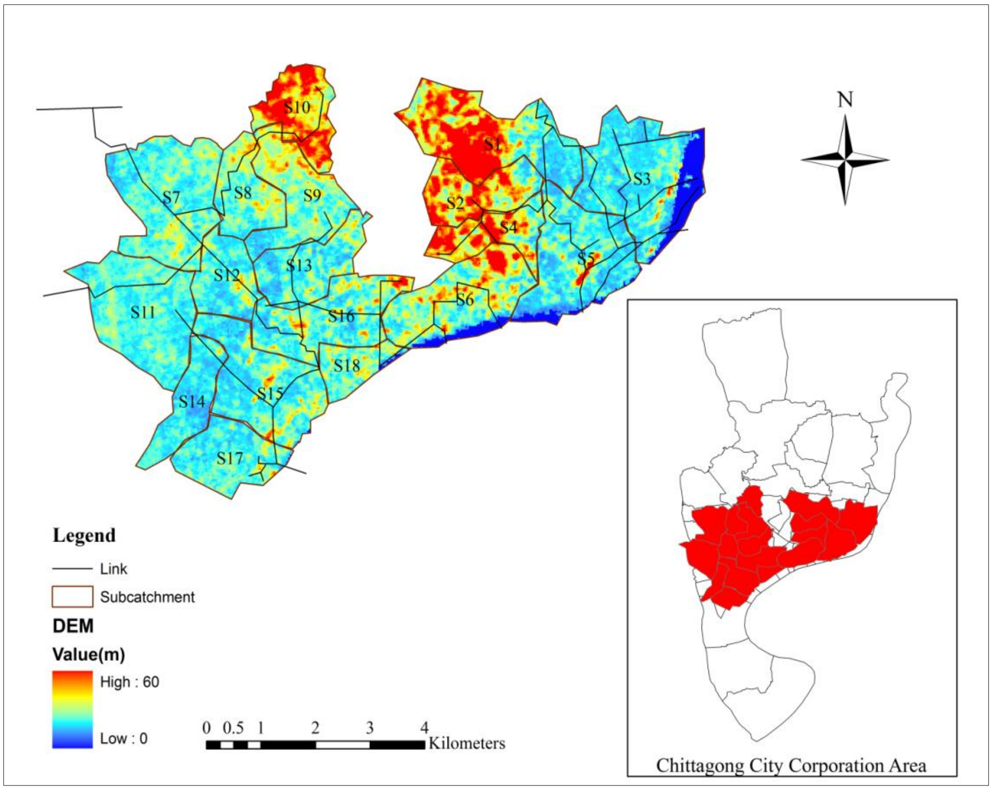

Figure 1. Study area

As the basis of hydrologic impact evaluation, urban land use study was carried out using the images acquired from U.S geological survey Landsat_8 (http://earthexplorer.usgs.gov) for 1st December 2013 considering cloud cover of1. 11.

This image was further processed using ERDAS IMAGINE. The satellite images were generated by applying coefficients for radiometric calibration, geometric rectification and projected to the Universal Transverse Mercator (UTM) ground coordinates with a spatial re-sampling of 30 m. Land use study was carried considering 4 criteria viz. vegetation, bare soil, water body and built-up area. The built-up area was accounted as impervious area (Table 2).

The overall accuracy ranges are from 0 to 1, and kappa value exists between -1 and 1. If the test samples were in perfect agreement (all the same between classification results and predicted results), values for the overall accuracy and Kappais equal to 1. The overall classification accuracy of each image was over 85.25% with kappa values over 0.816, meeting the accuracy requirements.

Delineation of sub catchment

Initially raw data from Shuttle Radar Topography Mission (SRTM) with 30m resolution Digital Elevation Model (DEM) was analyzed with Arc-GISv9.3 and HEC-GeoHMSv5.0 terrain processing tool with input stream network. The output 18 sub-catchments were obtained with the input stream network which is existing link network of study area. The delineation was followed several steps such as DEM reconditioning, filling sink in DEM, assigning flow direction and flow accumulation. Obtaining stream segmentation finally sub-catchment grid cell was processed which was used with terrain processing of HEC-GeoRASv4.3 preprocessing with stream network. The area of each sub catchment can be calculated in SWMM interface which imported as metadata from ArcView.

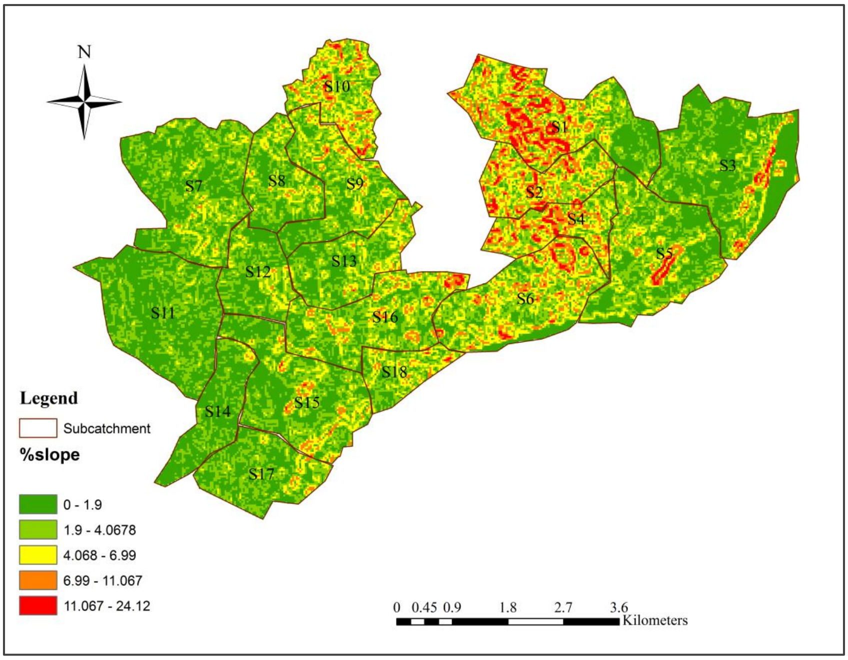

Figure 2. Slope analysis of study area

Importing stream-network in SWMM from HEC-GeoRAS

HEC-GeoRAS v4.3 preprocessing was conducted using the raw SRTM 30 m as grid layer. Then channel topography like stream centerline, flow path, bank line and cross section with floodplain are processed from exporting HEC-GeoRAS. Total 152 conduit links as transects (irregular channel cross section) and 144 junction nodes are imported in SWMM. Total 8 numbers of outfalls are located in adjacent Karnafuli River. Each conduit is subjected to an upstream boundary condition of runoff hydrograph and downstream boundary condition of tide curve at outfall.

Tidal harmonics of Karnafuli River

Observing water level records in coastal waterways in Karnafuli river has two tides a day each tidal cycle lasting about 12 hours 24 min. Both of the tidal range and the main water level vary seasonally and from place to place along the river (Akter and Tanim 2015). The variation in a month shows a neap tide and a spring tide. The tidal fluctuations can be classified as a diurnal fluctuation because tide usually describes in this case one high and one low water occur in the period of the rise, and also of the fall, of tide is approximately 12 hours (NOAA, 2006). Figure 3 shows the pattern of tidal fluctuations during June 2014 in Khal 10.

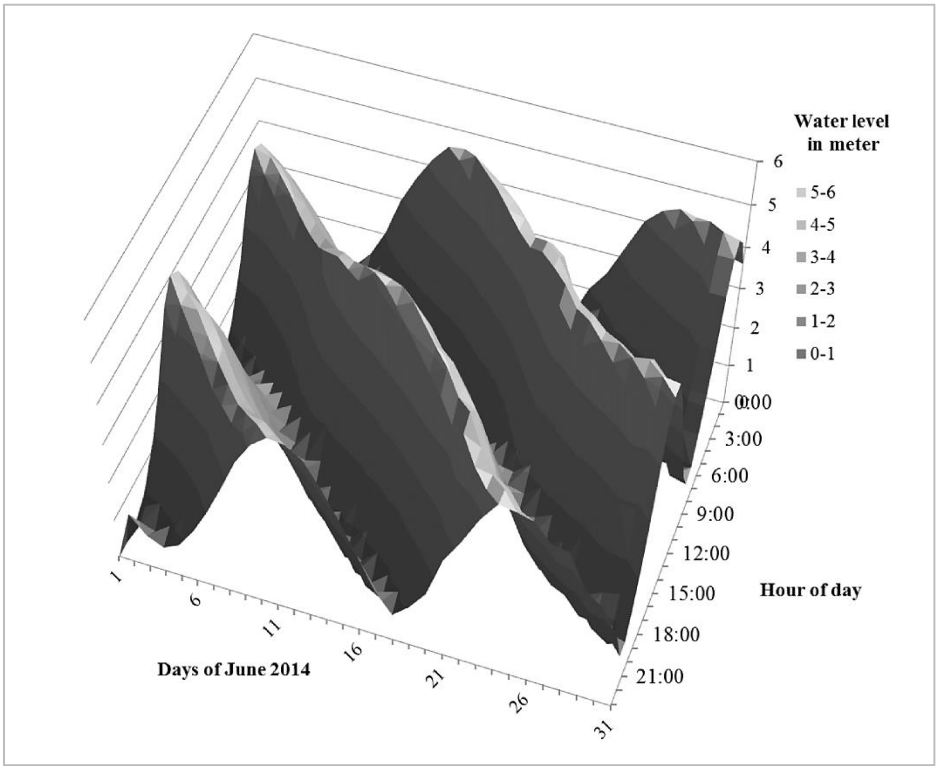

Figure 3. Tidal pattern of Karnafuli River at the Mohesh Khal outfall

Routing method and time steps

Dynamic wave routing method, based on 1D Saint-Venant flow equations, was used for simulating this study. In this method, flood occurs when the water depth at a node exceeds the maximum available depth as described by Rossman (2008). Thus, to simulate backwater effect the dynamic wave routing method with a time steps less than 60 s can be accounted. A time series of June 2014 with time steps of 30s was selected for the model simulation.

Model outcomes

The statistical analysis of flood in SWMM

The statistical analysis in SWMM can be performed for any hydrologic events. This analysis includes any event variable such as flood, runoff, precipitation, lateral inflow, discharge volume etc. The mean statistical analysis was conducted on each node with assigning a depth as variable. The maximum depth as threshold limit of each node was selected so that the number of over flow event and frequency during June 2014 can be determined (Figure 4). The water logging duration in June 2014 can also be obtained from simulation results (Figure 5).

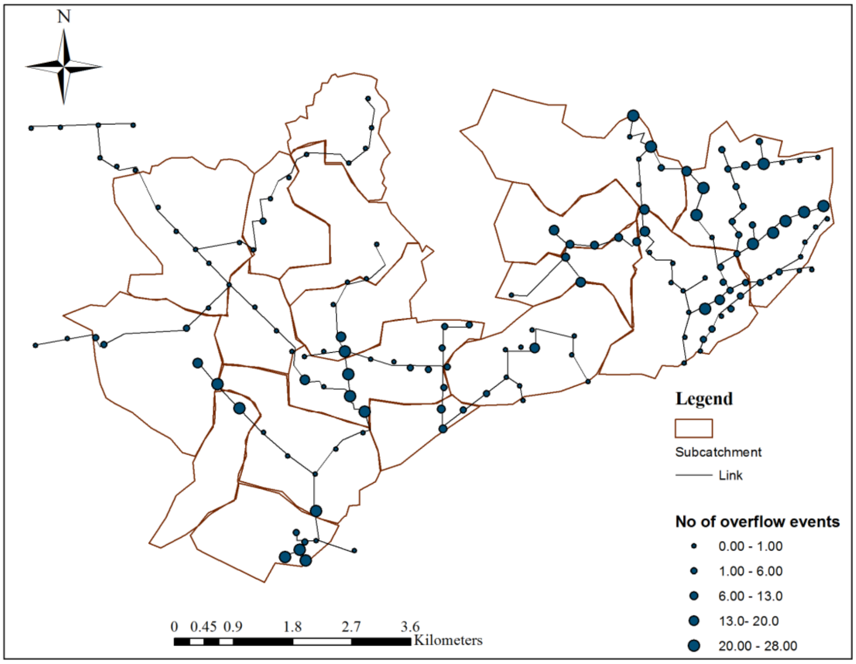

Figure 4. SWMM statistical analysis (numbers of overflow events during June 2014)

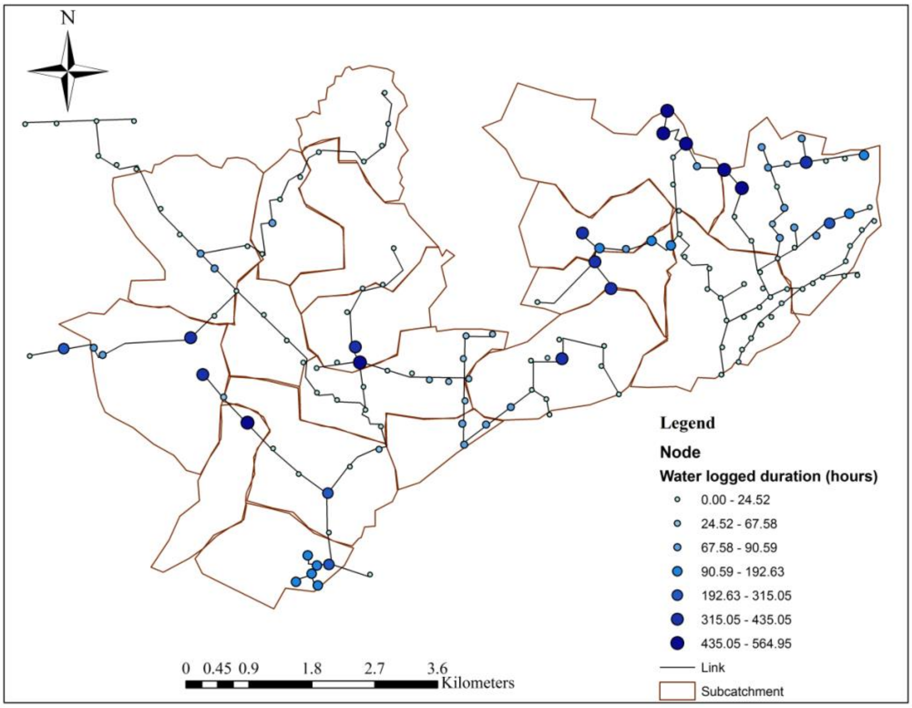

Figure 5. Water logging durations at different nodes during June 2014

Flood hazard analysis method

There are multiple approaches of flood hazard mapping, among them three main methodologies are available, those are: paleohydrological methods, hydrogeomorphological methods, and hydrological–hydraulic methods (Baker 1988; Benito et al. 2004). In addition to these the recent appearance of dendrogeo-morphological methods also engaged in this regard (Díez-Herreroet al. 2008). However, in most hazard analysis they are complementary of each other. Paleohydrological methods are suitable for statistical analysis regarding overflow events and can provide expected information while hydrological–hydraulic methods require large amounts of hydrological data those are integrated with SWMM.

Flood hazard analysis requires information regarding hotspot areas i.e., the place experiences overflow frequently. The basis of identification might be either field survey or numerical modeling. The database for statistical analysis must contain location of overflow nodes and number of overflow events in the nodes. A numerical modeling study i.e, SWMM integrated with GIS is more convenient to represent and gather such information. The statistical analysis in SWMM is detailed below.

Kernel density estimation for flood hazard

A flood is usually caused by a channel that has over flowing banks during high runoff period which can be predicted using annual stream flow study. Thus, for flood hazard analysis it is essential to determine the number of overflow events and this can be determined using statistical analysis in SWMM. Most common probability distribution used in hydrologic extremes modeling is Gumbel Probability Distribution. But, this probability distribution underestimates largest rainfall amounts. Kernel Density Estimation (KDE) plays an important role in the probabilistic characterization of phenomena through reducing identification difficulties of a well-defined probability density function (PDF) in the parametric sense (Tehranyet al. 2014)

Spatial analysis of flood hazard

To identify potential flood hazard zone it is difficult to rely on a well-defined probability distribution function. In such cases researchers suggested for Kernel Density function to determine spatial distribution of flood hazards (Cherqui et al. 2015; Baah et al. 2015; Camarasa et al. 2011; Kaźmierczak and Cavan 2011, Caradot et al. 2011). The hazard density (D) was calculated at each pixel in the territory (Cherqui et al. 2015):

$$D = \frac{\sum_{i=1}^{n} H_i . c_i}{S} \qquad \text{eq. (1)}$$

H_i corresponds to value of the hazard score for sewer flooding i, that is to say the number of events observed for each pixel of the map; c_i is a decreasing smoothing coefficient Eq. (2) and S is the area of a circle with radius R containing n sewer flooding events.

$$c_i = \left(1 - \left(\frac{r_i}{R}\right)^2\right) \text{If } r_i < R$$
$$c_i = 0 \quad \text{If } r_i > R \qquad \text{eq. (2)}$$

The Kernel density approach has been successfully utilized in many fields such as road accidents, health disease. Visual surveillance of photogrammetry, remote sensing based flood assessment etc. (Branko 2008). Usually a grid based representation provides a better representation of flood hazard than point based database. However if the point based flood hazard density able to estimate than it is convenient to interpolate the hazard density to the surrounding pixel. Thus at first, KDE was obtained at each node considering the SRTM 30m as unit pixel depth. Further, KDE with surrounding pixel are interpolated with overflow node KDE using the inverse distance weighted (IDW) surface interpolation method. IDW surface interpolation method takes the concept of spatial auto correction laterally. As statistical analysis carried out over 144 nodes in the study area it is expected that after interpolation KDE in the link network will represent reasonable value as distance among nodes varies from 100m to 500m. Thus obtained flood hazard map is shown in Figure 5

Flood hazard analysis scale

Depending on the needs of the water utility manager, the hazard density can be represented into two different scales:

- A strategic scale (i.e., Kernel density radius of 5 km) which represent a strategic overview of the main flood areas at city scale (Caradot et al. 2011); and

- An operational scale (i.e., Kernel density radius of 500 m) which is a complementary representation shows the disparities within each main flood area. This operational overview makes it possible to identify the network components responsible. This strategic scale is used for hazard mapping in the study area.

Identification of urban flooding hotspot

Based on Kernel hazard density, the location of study area can be classified in three categories i.e., most vulnerable, moderate vulnerable and less vulnerable (Figure 6). The details on urban flooding prediction note down in Table 3.

Table 3. Model predicted hotspot of urban flooding

Vulnerability class	**Locations**	**Kernel hazard density at strategic scale 500 m radius**
Most Vulnerable	Bakalia (West and South), some part of Chawkbazar (adjacent to Bakalia) Chittagong port, Probortak, Uttar Moddaya Halishahar	0.25-0.47
Moderate Vulnerable	South Agrabad, Sadarghat	0.17-0.24
Less Vulnerable	Rest of the parts of Study area	0-0.16

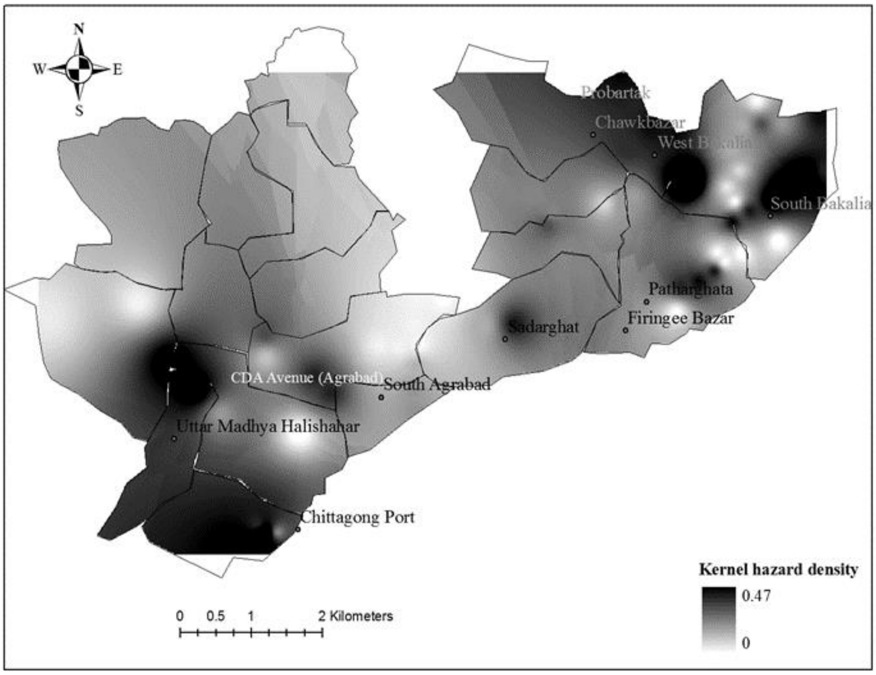

Figure 6 . Strategic representation of potential flooding based on kernel density function (radius 0.5 km) at nodes

Validations of model outcome

The predicted hotspots are verified with an intensive study of CWASA (2015) and Mohit et al. (2014). The vulnerable locations identified from those studies are Bakalia, Halishahar, Agrabad and Sadarghat in the study area. Several validations need to be justified from the study:

- Given the marginal sensitivity of topography and Manning's n value the model outcome might be influenced. The terrain elevation collected based on 30 m SRTM DEM might be fail to mimic the channel topography thus there might be influence on flow direction in link network;

- Manning's 'n' was taken in a range of 0.03-.04, a value in the theoretical range of roughness specification. Whilst a uniform roughness value of 0.03 simplifies the representation, given the scenario-based nature of this study, it is regarded as an adequate assumption;

- Inadequacies of rain gauge make the rainfall amount uniform over the study area. This might be overcome using Tropical Rainfall Measuring Mission (TRMM) precipitation or Next Generation Weather Radar (NEXRAD) following Wen et al. (2013) but those were not available during study period; and

- Determination of soil hydraulic conductivity from the SCS-CN method may not accurate representation for overall catchment area. But model assumptions required one constant value throughout the sub catchment.

Concluding remarks

The reported study is a part of existing study. Observing serious urban flooding in June 2014, this study was focused on Chittagong city based on available rainfall data from Patenga rain gauge. The GIS based SWMM could reasonably identify the flood hazard locations compare to the earlier field studies. In addition to the identification based on the Kernel hazard density, the validated model could estimate the intensity of flood hazards and thus the most vulnerable spots are spreads over Bakalia (West and South), some part of Chawkbazar (adjacent to Bakalia) Chittagong port, Probortak and Uttar Moddaya Halishahar. The validated model supposed to provide guidance once the model estimation could be verified by the available field survey.

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RIVER RESPONSE IN THE SELECTED REACH OF JAMUNA RIVER DUE TO DREDGING

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Abstract

Dredging in a river is certainly a human intervention need to be analyzed for better planning and decision processes. In this study, an attempt has been made to analyze the river responses due to dredging on a selected reach of the Jamuna River by preparing a morphological model of this river. The study reaches covers from 30 km upstream of Bangabandhu Bridge to 20 km downstream of this bridge. Various important hydraulic structures are situated around this area like East Guide Bund and West Guide Bund of Bangabandhu Bridge, Sirajganj Hard Point and Bhuapur Hard Point. Among all, Sirajganj Hard Point area has been focused here as a prime concern. To setup these morphological models, MIKE 21C, 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 fulfill the study objectives. From analysis, it has been found that with the increasing of dredging depth, dimensionless velocity increases along the dredged channel and decreases along the bank. Moreover, the bed materials along the bank stay at the threshold point of erosion during average flood year of the Jamuna River while at higher dredging depth condition, the velocity along bank decreases in such an amount so that it becomes lower than the critical velocity and the bank becomes non erosive zone. Due to dredging, bed scour near Sirajganj Hardpoint decreases maximum 33.4% for 7 m of dredging depth and average rate of decreasing of bed scour is 5.5% per meter of dredging depth. However, if dredging executed near Sirajganj Hardpoint, the channel became wider as the dredging depth increases. Finally, a technique has been introduced using the relationship curve between Dimensionless Velocity and Relative Dredging Depth to evaluating optimum dredging depth for planning dredging.

Introduction

Getting idea on river response due to dredging is a complex task. Two-dimensional morphological model can be treated as a useful tool to overcome this complexity. Change in a river due to any interference is a time depending morphological process in nature (Vries 1993). Rivers always try to achieve a stable state of equilibrium throughout it reaches over a period of time. One of the major river in Bangladesh, Jamuna is very dynamic in nature and the sufferings it causes to the people along with damages to national properties; the river has drawn the attention of researchers and planners. Several researchers like Coleman (1969) and Bristow (1987) have carried out comprehensive studies on the Jamuna River. Later in the 1990s, a number of studies were carried out to understand the behavior of the river in relation to the construction of the Jamuna Bridge. Besides that many researchers have conducted numerous experimental works, analytical studies and numerical modeling in the field of river response due to dredging. As example, Lagasse (1986) shape can retard the movement of bed-load sediments through a river system. In this context an attempt has been made to understand the river response due to dredging for a braided river like Jamuna using 2D morphological model.

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Methodology

In connection with this study, a two-dimensional morphological model has been setup in the selected reach of Jamuna River. To setup this model several related data including bathymetry, banklines, historical water level and discharge, sediment data and other information have been collected from different sources. To evaluate the response due to dredging, a two-dimensional morphological model has been setup using MIKE 21C software. Several steps have been followed to setup the model that includes grid generation, bathymetry preparation, boundary generation, calibration and validation. In this regard, cross-section of Jamuna River of the year 2010 has been used for the model bathymetry. The grid and bathymetry are shown in Figure 1. The upstream discharge boundary at the 30 km u/s from Bangabandhu Bridge (generated by flood routing using the Bahadurabad discharge data) and downstream water level boundary at the 20 km d/s from Bangabandhu Bridge (generated by analyzing the slope and distance) have been set up according to the model area and finally simulations have conducted for average flood event (hydrological year 2005).

The base model has been calibrated and validated against the year of 2010 and 2011, respectively. The calibration plot of water level and the validation plot of a cross-section of Jamuna River near Sirajganj Hardpoint have been shown in Figure 2 and Figure 3. The following parameters have been used for simulation of the 2D morphological model of Jamuna River.

Chezy C = $20h^{0.5}$, Eddy viscosity = 1, Sediment transport equation: Van Rijn
Bank erosion parameter: Slope = 1:10, Erodobility coefficient = 0.04

After setting up the model for Jamuna River, a suitable dredging alignment has been fixed keeping morphological phenomena in mind, also shown in Figure 1. Such an alignment has been fixed so that flow passes through the proper channel and causes no adverse effect in Bangabandhu Bridge as well as its adjacent hydraulic structures. Simulation have been conducted for different options which are made by varying the depth of the dredging section, shown in Table 1. A 15 km long channel has been dredged following the design dredged section near the Sirajganj Hard Point (Figure 1). Bottom width of the dredged section and the side slope of the section is 500 m and 1:3, respectively. Six different options have been carried out by varying the dredging depth from two to seven meter at one meter interval.

Finally, river response in the selected reach of Jamuna due to varying dredging depth have been analyzed by comparing the relative dredging depth with the dimensionless velocity. Moreover, a relation between bank erosion with the dredging depth is also developed.

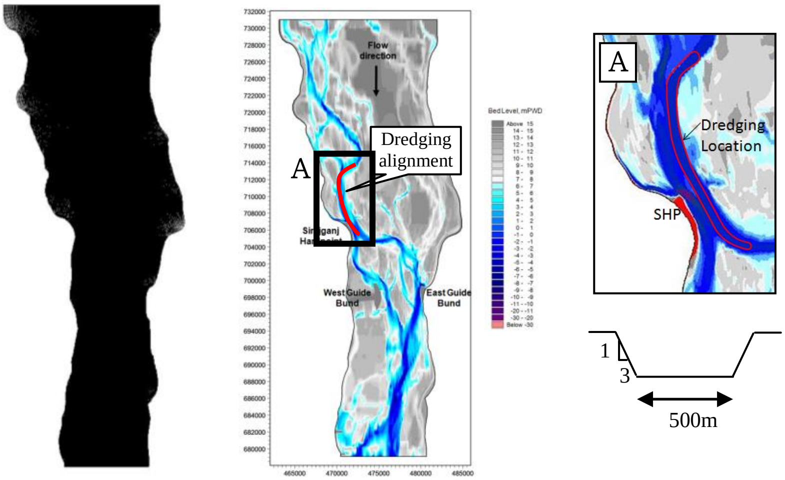

Figure 1. Computational grid, generated bathymetry and design dredge section of the dredging alignment for the study reach of Jamuna River.

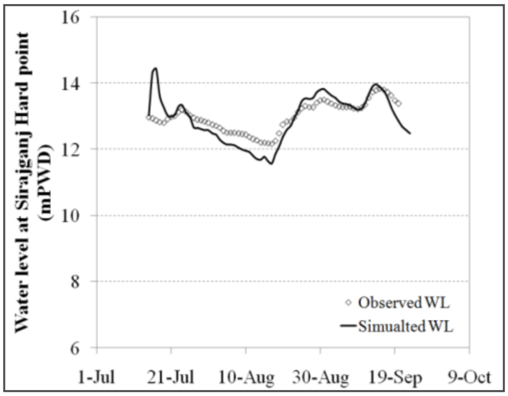

Figure 2. Comparison of model simulated and observed water levels of Jamuna River at Sirajganj

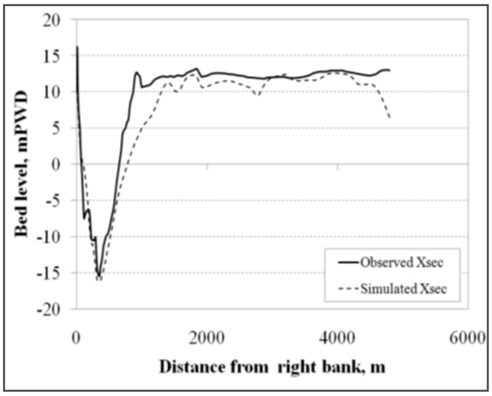

Figure 3. Comparison of model simulated and observed cross-section of Jamuna River near Sirajganj Hardpoint

Table 1. Different option simulations

Options	Dredging width (m)	Dredging depth (m)
Option 1	500	2
Option 2		3
Option 3		4
Option 4		5
Option 5		6
Option 6		7

Results and discussion

The simulated morphological change regarding planform of channel, bed scour are also extracted from the model to understand the relation with dredging.

Channel layout

Plan view of Jamuna River for different dredging depths has been extracted (Figure 4) from model and it is seen that in base condition, the left anabranch near Sirajganj Hardpoint became silted up at the end of monsoon. However, if dredging executed along the dredging alignment, this channel became widened and getting deeper as the dredging depth increases. Here, Option 6 represents the deepest dredging depth among all the options, though dredging width in all options are same. Thus, Option 6 generates deeper channel (Figure 4). Moreover scour depth in front of Sirajganj Hardpoint decreases with the increase of dredging depth.

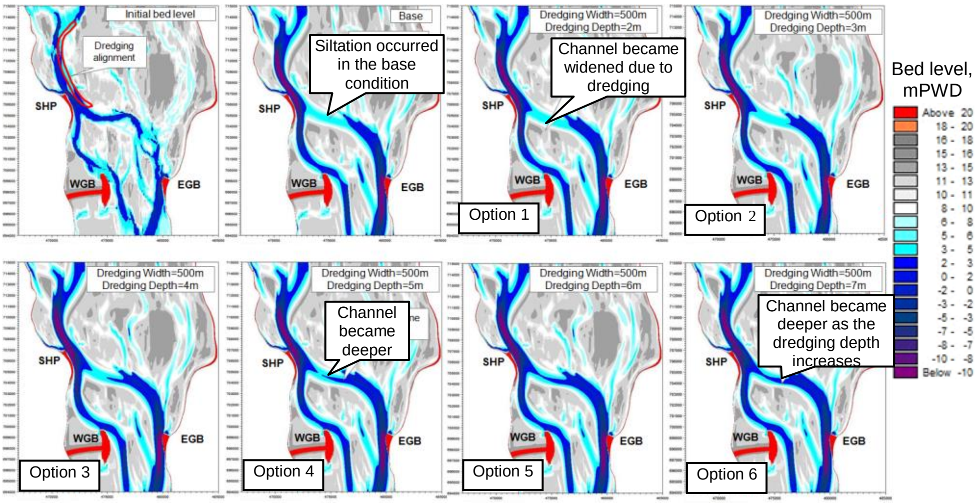

Figure 4. Simulated bed level of Jamuna River for different dredging depth

Bed scour

Cross-section of Jamuna River for different dredging depths also shows that the channel became wider due to dredging (Figure 5). Moreover, scour depth in front of Sirajganj Hardpoint decreases with the increase of dredging depth. Maximum bed scour near Sirajganj Hardpoint decreases 33.4% for 7 m of dredging and on average the rate of decrease of bed scour is 5.5% per meter of dredging depth. The conveyance area also becomes higher as the dredging depth increases. It is worth mentioning here that the dredged channel is silted up more than 30% after the monsoon. So, frequent maintenance dredging would be required.

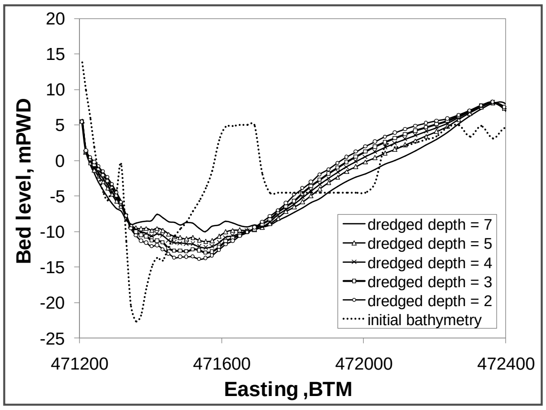

Figure 5. Simulated cross-section in front of Sirajganj Hardpoint for different dredging depth

Relationship between dimensionless velocity and relative dredging depth (for selected reach of Jamuna River)

Velocity and other required parameters are extracted from the simulated model at different locations (at the starting point, middle point and tailing point) along the dredging alignment. It is seen that dimensionless velocity increases with the increase of relative dredging depth along the dredge channel. On the other hand, near the bank location dimensionless velocity decreases with the increasing of relative dredging depth though the rate of decreasing of dimensionless velocity is very low. At the starting point of dredging alignment, numerically 8.8% dimensionless velocity increases at dredged channel location and 1.4% dimensionless velocity decreases near the bank location for 7 m of dredging. These values are more or less similar for the middle position of dredging alignment. At the tailing point, almost 17.3% dimensionless velocity increases along the dredge channel and 15.1% dimensionless velocity decreases near bank location for 7 m of dredging.

The relationship between dimensionless velocities with the relative dredging depth at different locations of Jamuna River are represented in Figure 6. All the plots are showing concave downward shape, the slope of the curve is increasing for along the dredged channel and decreasing for near bank location. Only at the tailing point at near bank location (Figure 6f), the shape of the curve is concave upward.

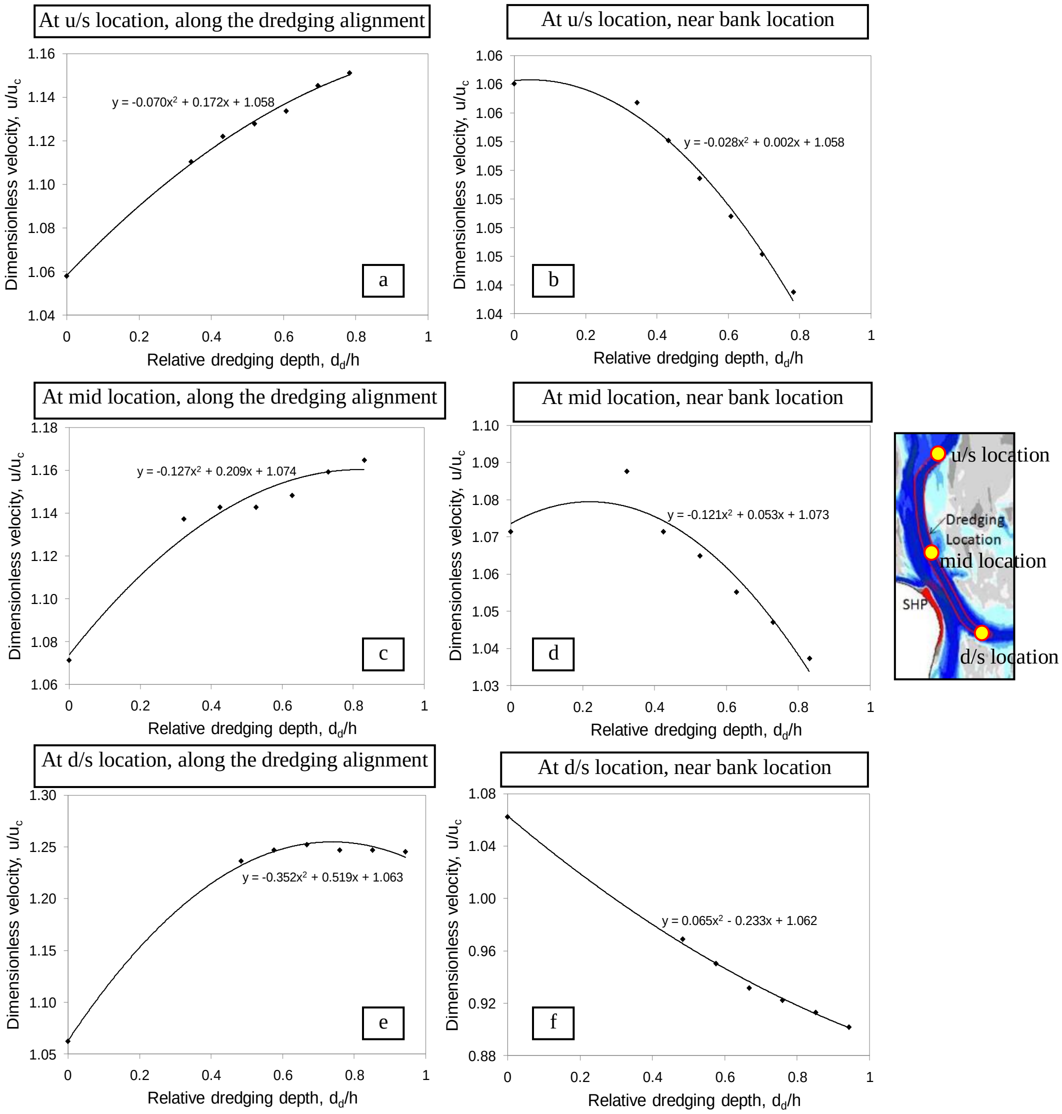

Figure 6. Dimensionless velocity Vs relative dredging depth curve for Jamuna River at different location of dredging alignment

As Jamuna River has the erosion tendency, hence a relationship has been generated between bank erosion with the dredging depth. It is found that model bank erosion decreases with the increase of dredging depth, Figure 7. In some point the bank erosion decreases maximum 50m.

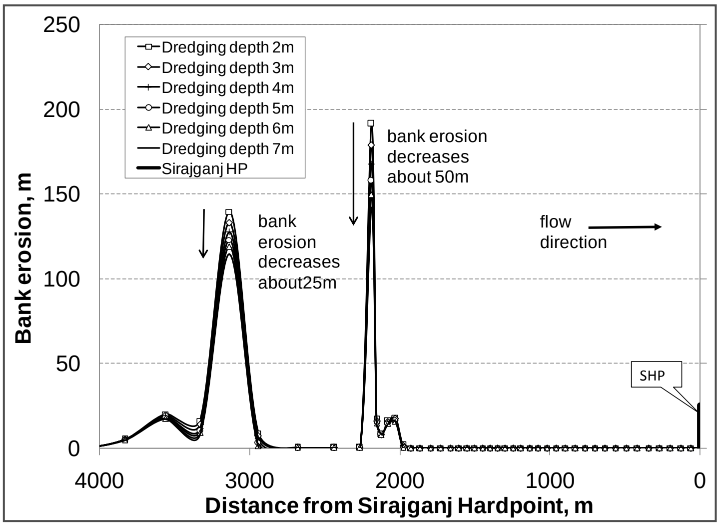

Figure 7. Simulated bank erosion for Jamuna River

In this study, bank erosion is also calculated by Mosselman Equation (FAP 24 1996) at the few kilometers upstream of SHP. It is seen from the calculation that with the increasing of dredging depth bank erosion decreases (Figure 8). So, the Mosselman equation also support the model results. It is worth mentioning here that the coefficient for the Mosselman equation has been fixed by comparing the simulated bank erosion results. In doing so, the time averaged erosion coefficient E_u is considered here as $10.6*10^{-5}$ for Sirajganj (calculating by interpolation). From FAP 24, it is found that the value of E_u is $9.5*10^{-5}$ for Bahadurabad and $7.2*10^{-5}$ for Kamarjani. Both of the locations are 75 km and 100 km upstream from Sirajganj Hardpoint, respectively.

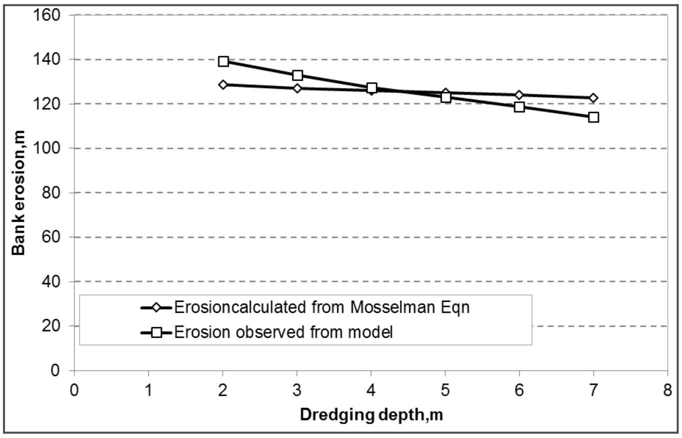

Figure 8. Comparison of simulated and calculated bank erosion

A sample calculation for optimum dredging depth

Dredging is a massive work, huge money and labor is involved in a dredging project. So, an optimum solution is required for allowable bank erosion. In this study, maximum bank erosion is found 140 m near Sirajganj. It is also seen from both the simulated and measured equation that with increasing of dredging depth, the bank erosion decreases. Now, if the maximum bank erosion is allowed 20 m near Sirajganj, the bank velocity is found 0.49 m/s from Mosselman equation (FAP 24 1996). Then the dimensionless velocity at the bank becomes 1.04 m/s. Now from Figure 6d, for dimensionless velocity 1.04, the corresponding relative dredging depth is found 0.82. As the average water depth at dredging location is 8.18 m, so the allowable dredging depth should be 10m near Sirajganj.

Conclusions

These are the conclusions that have been made from this study:

i. On Jamuna River, it has been observed that velocity increases along dredged channel and decreases along bank with increasing dredging depth. Numerically in some point dimensionless velocity increases maximum of 17.3% for 7 m of dredging depth. This is happened due to attracting flow toward dredged channel on increasing dredging depth. On the other hand, along the bank dimensionless velocity decreases maximum of 15.1% for 7 m of dredging depth.

ii. In Jamuna River bed scour near SHP decreases maximum 33.4 % for 7 m of dredging depth and the average rate of decrease of bed scour is 5.5 % per meter of dredging depth.

iii. From plan view analysis it is seen that in base condition, the left anabranch near Sirajganj Hardpoint became silted at the end of monsoon. However, if dredging executed along the dredging alignment, this channel became widened and was getting deeper as the dredging depth increases.

iv. Finally bank erosion is calculated using the model simulated data. It has been found that the bank erosion is decreases with the increasing of dredging depth. Mosselman equation is also used for comparing the bank erosion result found from simulation. A slight deviation is found between the two results due to some valid reasons. To overcome this deviation, a value of time averaged erosion coefficient E_u in Mosselman Equation is suggested for Jamuna River near Sirajganj area which is $10.6*10^{-5}$. Using Mosselman Equation further erosion along the bank could be calculated. Moreover, this bank erosion value and developed relationship curve between the dimensionless velocity and relative dredging depth would be helpful for planning the optimizing dredging depth.

Acknowledgement

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POST-EVALUATION OF THE GANGES LEFT BANK EROSION PROTECTION PROJECTS FROM PANKA NARAYANPUR TO INDO-BANGLADESH BORDER

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Abstract

River training and bank protection is an important responsibility of Bangladesh Water Development Board (BWDB) as the damage caused by bank erosion disaster every year is colossal. So far BWDB has taken up a number of such projects some of which are a great success while some others have failed to serve the intended purposes. Post project evaluation allows recognizing project achievements and identifying techniques and methods that worked or not worked. A lack of post evaluation may inhibit appropriate project and policy adjustments. This paper describes post evaluation of two important BWDB bank erosion mitigation projects which were implemented at Panka Narayanpur to Indo-Bangladesh border along the left bank of the Ganges during 2002-2005. The aim of the evaluation is primarily to compare the actual outcomes of the project with the projections made at the appraisal stage. The investigation of different aspects of the project can provide important lessons derived from field experience which would be beneficial for the successful implementation of new projects. The overall impact of the completed project will result in a number of effects, which can be classified in the form of costs and benefits, direct and indirect or tangible and intangible. It is identified from the post evaluation that the negative results of the first project have been successfully nullified by implementing the second project through close monitoring of the developments after implementation of the first project. The post evaluation also indicates that site specific solutions have to be devised for successful project implementation together with proper planning and ensuring of construction as per design and within the stipulated time frame.

Introduction

Water resources planning and evaluation activities have usually emphasized future cost and benefits of potential projects. Detailed planning documents, the principles and guidelines are available to Bangladesh Water Development Board (BWDB). However, no comparable document exists to guide retrospective, post reviews of river bank protection projects and progress. Preferences may change considerably after execution of such type of project. Broadening BWDB water resources management to integrate a greater degree of post evaluation may help resolving some of the nation's water resources controversies. Post evaluation takes place after the completion of the project and often more in-depth since it focuses analysis of impacts. Besides, it is time-consuming, costly and calls for persons with special skills. So far systematic post evaluation of implemented projects and use of the lessons learned in formulation of future projects is limited. However, the need for

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such post evaluation of projects is gaining more and more importance for sustainable water resources development. A post project evaluation is a very useful and powerful way of adding a continuous improvement mechanism. This continuous improvement mechanism helps make each succeeding project more successful. Post project evaluation typically involve the project team and major stakeholders meeting together and reviewing what went well and what went badly during the project. This input can help participants make the right decisions and plans so that the next project runs better. It can also help clear up misunderstandings and other issues.

BWDB, whose one of the important responsibility is to arrest bank erosion throughout the country. The organization has implemented a number of river training and bank protection projects on the Ganges which is a trans-boundary river. After traveling about 2240 km through India, it enters Bangladesh at Panka Narayanpur, near Shibgonj Upazilla under Chapai Nawabganj district and18 km downstream of Farakka Barrage and runs for about 100 km as a boundary between India and Bangladesh. The travelling length through Bangladesh is about 230 km. It is wide wandering river with a bank full width of about 5 km.

The hydrology of the Ganges river is very intricate especially in the Ganges Delta region. The hydrologic cycle in the Ganges basin is governed by the Southwest monsoon. About 84% of the total rainfall occurs in the monsoon from June to September. Consequently, stream flow in the Ganges is highly seasonal. The average dry season to monsoon discharge ratio is about 1:6, as measured at Hardinge Bridge, an important monitoring point where India and Bangladesh jointly monitor water level and discharge under water sharing treaty, 1996. The maximum peak discharge of the Ganges as recorded at Hardinge Bridge point in Bangladesh, recorded 70,000 m^3/s. The minimum-recorded discharge at the same was about180 m^3/s in 1997. Prior to constructing Farakka Barrage (1975), minimum recorded discharge was about 1297 m^3/s in April 1966. The water surface slope of the river is about 5 cm per km (FAP 24 1996a).

The bank material of the Ganges within Bangladesh territory consists of loosely packed sand and silt. These materials are highly susceptible to erosion. The bank erosion process along the Ganges is controlled by its wandering plan form characteristics. Maximum bank erosion, however occurs in the meandering reaches, near the outer bend can still migrate laterally within the corridor. In the period 1984-1993, the maximum observed rate was 665 m/year. Along the right and left bank of the Ganges, erosion rate are 56 m and 20 m per year respectively. About 45 km^2 area of valuable land has been eroded by the Ganges for the period of 1990-2000 (BWDB 2010). The width of the river varied from 1.7 to 10 km in 1984 and 1.9 to 11.7 km in 1993. The average width of the river in 1984 was 4.37 km, which increased to 4.69 km in 1993. The widening rate of the river is 36 m/year. Some rivers cause erosion in large scale and high frequency due to their unstable character. Ganges river assumed a braided pattern consisting of several channels separated by small islands in their courses.

The study area (Panka Narayanpur to Indo-Bangladesh Border) is located on the left bank of the Ganges river immediately downstream of the Indo-Bangladesh border that falls under Shibganj and Nawabganj Sadar Upazila under Chapai Nawabganj district. The Latitude and Longitude of the project area are $24^{0}30'$ to $24^{0}38'$ and $88^{0}24'$ to $80^{0}28'$ respectively shown in Figure 1.

Huge amount of agricultural land engulfed by the progressive bank erosion in the study area led BWDB executing two projects in the study area namely Panka Narayanpur Project and Babupura to Indo-Bangladesh border Project. The length of the study area along the left bank is 14 km at Panka Narayanpur and 9 km from Babupura to Indo-Bangladesh border. The river is very dynamic and erosion prone within the reach between Panka Narayanpur and Indo-Bangladesh border.

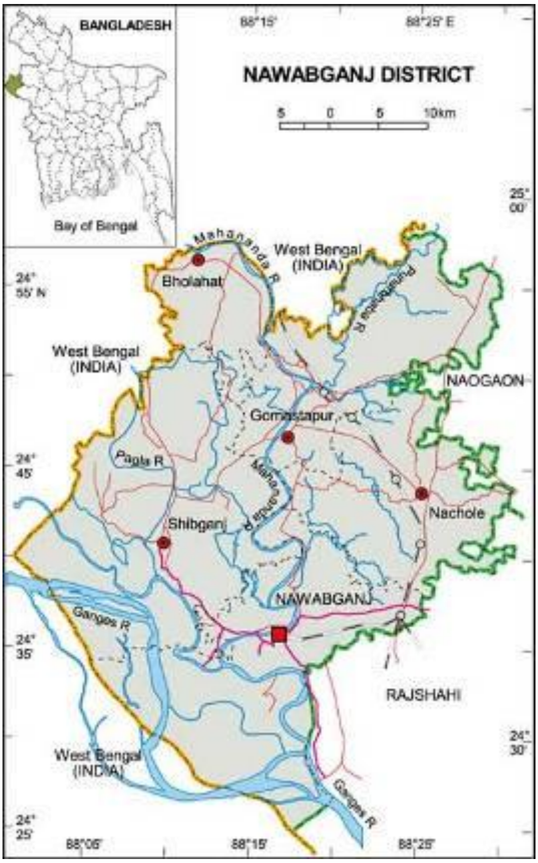

Figure 1. Location of the study area

Methodology

The following steps have mainly been followed to extract data and information for the purpose of this study:

- Collection and review of relevant study reports, documents, publication books, scientific journals etc.;
- Collection and analysis of time-series satellite images of the study area;
- Stakeholder consultations that include BWDB officials and local people on whom there is direct impacts of the project.

In the course of evaluation, standard evaluation criteria have been used under the same matrix where two sets of responses have been explored. First set characterizes BWDB's endeavors and consequences while other reveals people's voices.

Findings from the study

The major findings from the study based on literature review, image analysis and data analysis have been presented hereafter.

Historical trend of morphological changes

In 1990s, the Ganges river increased its width by causing serious erosion along the left bank in the reaches upstream of Chapai Nawabganj district. A study (IWM 2006) showed that the bank line moved up in the north direction up to 6 km at the rate of 600 m per year. This movement had been continued and the bank line was shifted up to 7 km within the period 1990-1999. This continued bank erosion posed the threat of causing merger of the Pagla river with the Ganges in the near future. It is predicted that the merger of the Pagla with the Ganges may cause merger with the Mohananda river in the long run.

These critical morphological situations of the river systems may put the Rajshahi-Chapai Nawabganj region at risk. Lateral bank shifting (of about 7 km) due to progressive erosion is shown in Figure 2.

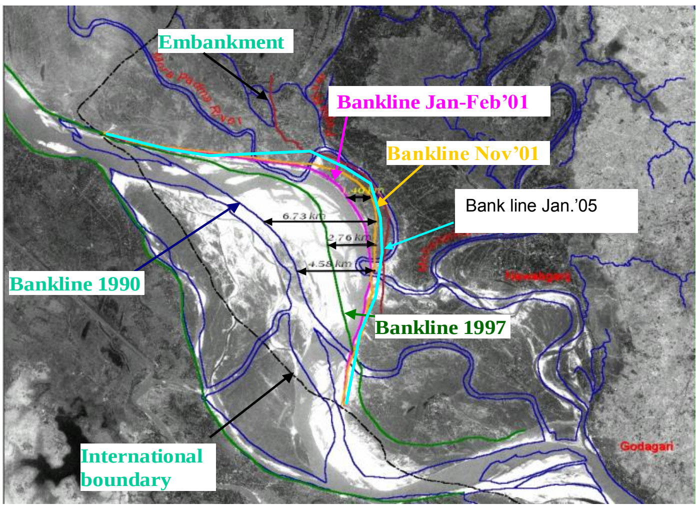

Figure 2. Bank line shifting of the Ganges River at Panka Narayanpur to Indo-Bangladesh border (Source: IWM 2006)

Model studies and implemented measures

The threat arisen out of such unabated bank erosion in the studied area led BWDB to take up a project during the year of 1996 aiming at combating the bank erosion problem. Due to complexity in dealing with erosion problem, BWDB commissioned River Research Institute (RRI) to carry out a physical model study for the erosion prone area under this project. RRI recommended some structural measures for the protection of vulnerable area after accomplishing physical model study. In the following monsoon, the bank line for the modeled area shifted towards north and the erosion rate per year was more than 500 m. Under such conditions, it was not possible for BWDB to take structural measures recommended by RRI. The severity of bank erosion was continued and the BWDB formed a technical committee in 1999 in order to protect the most vulnerable Panka Narayanpur area. The committee recommended construction of 8 (eight) nos. of spurs and the length, location and orientation of 8 spurs were finalized by Institute of Water Modeling (IWM) and also verified by RRI. As per recommendation of the technical committee, IWM carried out 2D mathematical model study to meet up the objectives set forth by BWDB. RRI conducted physical model study accordingly with a view to observe bank retreat between the spurs finalized by IWM.

Seven spurs (except spur #1) out of eight, constructed fully in 2002 working season under the project taken by BWDB in the study area. After 2003 flood, about 66 m RCC (Reinforced Cement Concrete) portion out of 150 m was completed but during flood season (July) earthen shank of the spur 1 failed and detached from the concrete tail. At that time about 4 km of bank line, both at upstream and downstream of spur #1 was severely eroded. About 100 m of the RCC head of completed spur #8 was uprooted in late September of the same year. During the following flood, about 3km bank at upstream of spur #2 was under severe erosion attack due to concentration of main flow along the left bank. The lateral shifting of bank line towards countryside by this erosion was 500 m to 1500 m. Due to this embayment, 100 m concrete head of spur #2 and spur #7 including some portion of shanks was damaged in August-September 2004. Shanks and concrete heads of spur #2, #7 and #8 were partially damaged in 2003-2004 floods and were destroyed by 2005 flood. Required amount of hard material and sand filled geo-bags were dumped in the affected areas aiming to protect the spur field and their anticipated effectiveness. Prior to flood in 2005, 900 m bank revetment between spur #3 and #4 along with 1080 m bank revetment works at both upstream and downstream of damaged spur #2 was constructed as an emergency basis in order to arrest further embayment of the river.

During monsoon in 2004, the erosion continued along the left bank of the Ganges consequent with about 1500 m bank line shifted locally near the failed spur. Thus created a renewed threat of merger of the Ganges with the Pagla river near the bank of the damaged spur #2. In December 2004, the most vulnerable location was the point where the minimum set back distance between the Pagla and the Ganges was only 150 m. The bank erosion within this reach still showed increasing trend and propagated towards upstream direction i.e. Babupura to Indo-Bangladesh border. Addition to this, the area between spur #6 and spur #8 (cross dam, embankment, and link channel) became vulnerable due to the damage of spur #7 and spur #8.

In 2005, BWDB planned to implement adaptive mitigation measures on the Ganges left bank erosion in order to arrest this continued erosion and to prevent further damage of existing spurs. In this context, it became obvious that the Ganges left bank from Babupura to Indo-Bangladesh border, upstream of spur #3 and the area between spur #6 and #8 should be protected by adopting appropriate protective measures. Before implementing such type of adaptive mitigation measures, it was necessary to carry out both feasibility and detail study to decide about the optimum solution of the erosion problem. Accordingly, IWM carried out feasibility and detail study in which numerical modeling was a component to find out the probable solutions mitigating existing bank erosion problem and strengthening previous protective measures. IWM recommended bank revetment (more detailed descriptions in IWM 2006) at different locations within the project area. Concurrently, RRI suggested bank revetment (detailed in RRI 2006) to protect the vulnerable area. Both the model results were quite similar to each other excepting only 1000 m increasing length of revetment for proposed revetment 1 towards upstream direction.

Post-project conditions

Field visit revealed that spur #3, #4, #5 and #6 are still existing in the field and functioning well some of which (spur #5, #6) can be evident from Figure 5 and Figure 6. Emergency basis constructed revetment between the damaged spur #1 and #2 and another revetment between existing spur #3 and #4 are in good shape and functioning well. The revetments constructed at the downstream of existing spur #6 are also in good condition. Different types of bank protective structures such as spur and revetment constructed under two projects have positive bearing to arrest bank erosion for a 23 km long stretch of left bank of the Ganges river.

Figure 3. The pictorial view of head of Spur #5

Figure 4. Photographic view of head of Spur #6

BWDB's monitoring program

Monitoring is an essential task principally for riverbank protection project. It means to observe a situation for any change, which may occur over time. Close monitoring of the evolution of bank line is, therefore, necessary to have forewarning of impending danger. It will help taking appropriate measures on time.

Damage and failure of bank protective works are very common phenomena particularly in large rivers. Governing factors of such impairment are improper design and construction method, lack of regular monitoring and unavailability of funds in time (BWDB 2010). Therefore, routine monitoring is an imperative tool for any successful water resources project especially during and after implementation. Since, the project was completed successfully and the constructed spurs and revetment are functioning well at present field condition, it would be an imperative task for BWDB to take necessary steps regarding close monitoring of the structures so that the project objectives would be achieved profitably without any major damage and failure in future.

Having considered this situation, BWDB has undertaken close monitoring program for the studied project through monthly and weekly field visit during dry and flood season. The concerned BWDB's official informed that frequent field visits were made when they have got message regarding bank erosion of the project area. However, proper repair and maintenance works have not been made due to lack of funds.

Post-evaluation of the implemented projects

An evaluation is a rational and objective assessment regarding the relevance, effectiveness, efficiency, sustainability and impact of activities in the light of specified objective. An important goal of evaluation is to provide recommendations and lessons to the project managers and implementation components that have worked on the projects and for the ones that will implement and work on similar projects. Evaluation can also be used to promote new projects get support from governments, raise funds from public or private institutions and inform the general public on the different activities. According to the United Nations Development Programme (UNDP), an outcome indication has two components: the baseline, which is the situation before the project begins and the target, which is expected situation at the end of the project. Through the evaluation process, using same matrix, two sets of answers were searched out. One set represents BWDB's attempts & consequences and other set represents people's views. Major comparative Matrix between two sets of views is shown in Table 1. It can be fairly said that, some specific differences are found between them.

Table 1. Major comparative matrix for the evaluation of implemented projects

Evaluation criteria	Project name: River bank protection of Panka Narayanpur area		Project name: River bank protection of Babupura to Indo- Bangladesh border	
	BWDB's Attempts and consequences	People's voice	BWDB's Attempts and consequences	People's voice
Relevance	As per people's demand and restoring the environmental degradation of the vulnerable area, BWDB had undertaken a project.	Experiencing serious bank erosion of the Panka-Narayanpur area and comprehending possible merger of the Ganges river with the Pagla river people raised their voice to take necessary action.	Owing to the failure of 4 nos. of constructed spurs under the previous project and progressive erosion at upstream and downstream of the existing spurs, BWDB had undertaken another project in 2005 at Babupura to Indo-Bangladesh border which encompassed the previous project area.	People's perception became optimistic since their demands had not been reflected fully in the previous project.
	In 2002, BWDB erected 8 nos. of spurs under the project.	People's tones were, bank protective structures like spurs were not adequate to combat the erosion. Their demands were to construct embankment with revetment.	About 4.05 km and 8.00 km revetment at upstream and downstream of the existing Spurs were constructed under this project. Also repair works were done to strengthening the existing Spurs.	

<table>
<tr><th rowspan="2">Evaluation criteria</th><th colspan="2">Project name: River bank protection of Panka Narayanpur area</th><th colspan="2">Project name: River bank protection of Babupura to Indo- Bangladesh border</th></tr>
<tr><th>BWDB’s Attempts and consequences</th><th>People’s voice</th><th>BWDB’s Attempts and consequences</th><th>People’s voice</th></tr>
<tr><td>Effectiveness</td><td>In one year later (In 2003) 2 nos. of spurs at upstream and 2 nos. of spurs at downstream were fully or partially damaged and were not working effectively.

Constraining factors
Improper design and construction of these spurs
Quick morphological changes at upstream
Inflexibility of budget allocation and lengthy approval process
Unavailability of fund in time</td><td>Populace felt unsecured. They demanded pragmatic solution to arrest bank erosion since their demand were not fulfilled.</td><td>Newly constructed revetment works along with existing spurs were sufficient to arrest the bank erosion.

In addition to that, restoring environmental degradation was also achieved.</td><td>Public felt protected and satisfied as their demand were fulfilled.</td></tr>
<tr><td>Efficiency</td><td>All the constructed spurs were not functioning properly due to changes of flow field at the head of individual spur.</td><td>People shouted repeatedly to take necessary measures to protect bank erosion.</td><td>The combined effect of revetment and spurs were positive to produce smooth flow field without endangering the bank.</td><td>People were convinced . They felt more confident and their attitudes were positive.</td></tr>
</table>

Evaluation criteria	Project name: River bank protection of Panka Narayanpur area		Project name: River bank protection of Babupura to Indo- Bangladesh border	
	BWDB's Attempts and consequences	**People's voice**	**BWDB's Attempts and consequences**	**People's voice**
Sustainability	Project's objectives were not achieved completely considering environmental aspects since the constructed spurs failed to protect the homestead aside the bank along with mango garden.	Numbers of distressed people were increased alarmingly and they moved to another places for their settlement and livelihood as benefits were unlikely to be maintained for an extended period.	The project's objectives were fulfilled considering all aspects including environment.	Inhabitants aside the bank had stayed in their parental homestead and were satisfied for maintaining their livelihood smoothly.
Impact	After implementing the project, employment opportunity, income, social wellbeing of the affected people had been increased to some extent.	There was adverse environmental impact around the project area. General people of that area were not benefitted.	After executing the project, employment opportunity, income, social wellbeing of the affected people had been augmented notably.	People hadn't seen any adverse impact of the implemented project and felt benefitted.

It is depicted from Figure 5 (Satellite image 2015) that, sedimentation occurred among the spurs. Main channel has been shifted from left bank to the right bank due to the combined effect of revetment and spurs erected in the exposed area. Other area remains with the left bank. At the downstream of spur #6, the bank line has been shifted to the left in comparison with bank line 2006. The bank line at upstream of spur #6 is found to be almost same for 2006 and 2015.

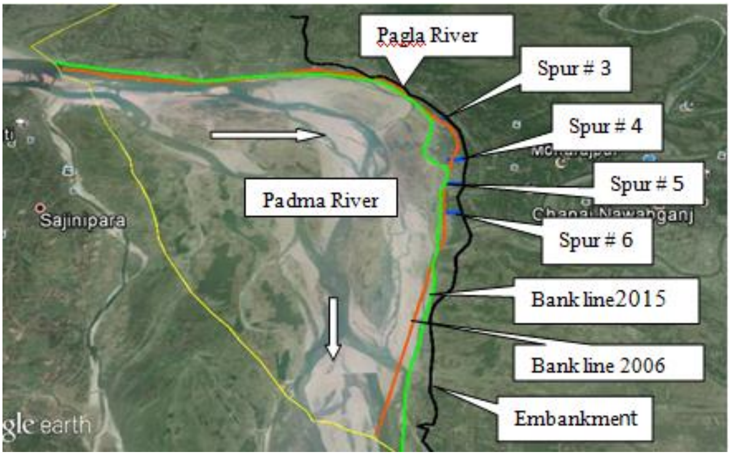

Figure 5. Bank line of the Ganges River at Panka Narayanpur to Indo-Bangladesh border (Source: Satellite image 2015)

Concluding remarks

Two projects were commissioned by BWDB in 2002 and 2005 asynchronously to arrest the bank erosion at Panka Narayanpur area and from Babupura to Indo-Bangladesh border under Chapai Nawabgonj district. The second project aims to strengthen the constructed structures under the first project and to mitigate exposed bank erosion in the upstream of the first project location. Investigation of the present status of the projects in terms of structural integrity and efficacy for preventing bank erosion and also the results of the public consultation and beneficiary investigation have shown local people's contentment with the benefits of the latter project. The project has also contributed to the upgrading and enrichment of the people's living environment as well as the economic and social advancement. In light of this, the latter project is considered to have reverted positive efficacy and impacts in many ways whereas preceding project was unavailing at some points with the outcome made at the appraisal stage.

Bank protection works should be planned and designed based on thorough understanding of geo-morphological and ecological processes of the exposed area, rather than merely imitation of form, as in blind application of a classification scheme. In addition to that, employment of physical as well as numerical model studies needs to be considered prior to implementing bank protection project since they are complementary to each other and have been proved to be very essential tools for finding safe and less expensive solutions of different water related problems. Most of the river bank protection projects in Bangladesh implemented by BWDB have not been subjected to objective post-project evaluation due to lack of evaluation program and unavailability of necessary fund and manpower. As a consequence, opportunities to learn from past experience to improve future project planning and design have been lost.

Summarizing the overall evaluation of projects, subsequent definite recommendations can be made:

- In a mighty river like the Ganges, construction of a series of spurs may be considered to facilitate formation of a stable river course safely away from the bank without endangering the structures. The appropriate placement, dimension and orientation of the structures should be finalized by employing physical and numerical studies;

- Preferably all spurs of the series should be constructed in the same hydrological year or a sequence of construction should be devised through hydraulic modeling studies;

- There should be a mechanism for third party inspection and monitoring of the construction works thoroughly by forming a team and effectiveness should be checked by both field inspection and discussion with the local people;

- The existing practice of design for revetments, groynes and spurs has to be reviewed for each specific project and design should be modified according to the site condition, if necessary; and

- Provision of necessary funds for operation and maintenance and also for monitoring and evaluation of the implemented project should be included in the project proposal.

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A GEOTECHNICAL PROFILE OF HAOR AREA OF BANGLADESH

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Abstract

Analyzing soil parameter is an utmost important job 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 making available information on foundation soils to the design engineer, a study is made to investigate the engineering soil parameters around the haor areas of the country. Accordingly, an effort is made to study on haor area soils of Bangladesh focusing soil profile along with its physical and index properties and to disseminate them to the geotechnical engineers. The data have been collected from the Soil Mechanics Division of River Research Institute (RRI) by testing of relevant engineering parameters of haor areas soil of Bangladesh. As River Research Institute has been testing disturbed and undisturbed soil samples collected mainly from the Bangladesh Water Development Board (BWDB) through accomplishment of soil borings by the geologist of the same throughout the country. In this study it is found that brown and grey in colour soils are existed in those area up to the maximum depth of about (0-102´). Their natural moisture content of cohesive soils is varying from (25%-752%). At the same time their plasticity indices are varying from (5%-153%), which has been seen in observation. Of course, their strength is different, which varies from 0 to 383.04kN/m^2.On the other hand, relative densities of those soils are varying from very loose to dense. The findings of the study are in general that cohesive and non-cohesive soil layers exists almost every region of the haor areas of Bangladesh. 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 haor areas of the country.

Introduction

Bangladesh is a riverine country with low lying land. It has about 8, 58,000 hectare of haor areas wherever agriculture and fisheries are the main economy. It has about 2 crores of population. We know haor-baor is an abandoned meander isolated from the main stream channel by deposition and filled with water. Accordingly, baors/haors are situated in the North-Eastern part of Bangladesh and they are in the districts Sunamgonj, Habigobj, Sylhet, Moulovibazar, Netrokona, Kishoregonj and Brahmanbaria. The problems of the haor areas are enormous. Flash and normal monsoon flood and lack of communication are the major problems in that area. The first one hampers production of crops and another makes suffering to the public lives. For these reasons, about 28% people are in poverty line. Considering such situation,

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Bangladesh government has undertaken sustainable development project to mitigate these problems, at which construction of structure is one of the main component. In order to accomplish the structures, project authority is conducting different tests of soil samples to collect design parameters. This attempt has been made to facilitate the construction works of haor areas through exploring the engineering soil properties of such areas. To accomplish this study, a number of soil testing parameters are collected from the soil mechanics division of geotechnical research directorate of River Research Institute (RRI). These analyzing parameters will focus an indication to the engineering properties of soil such as, index and physical properties of the haor areas from which a design engineer may know the soil conditions. It will also help the design engineer to design structures in the same area.

Literature review

Geologically, Bangladesh is a part of the Bengal Basin, one of the largest geosynclines in the world (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. Table1 shows the ages, lithology and characters of those formations and their accessible locations.

Table 1. The lithology of Bangladesh

Ages	Formation	Lithology and Character	Location
Holocene	Alluvium	Aquifers of Sand Silt and Clay	Flood Plains
Pliestocene	Modhupur	Red Clay, Ferruginous Nodules	Barind
Pliocene	Dihing	Sandstone	Deep
	DhupiTila	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
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 and Metamorphic rocks	Dinajpur

(Source: Haque 2008)

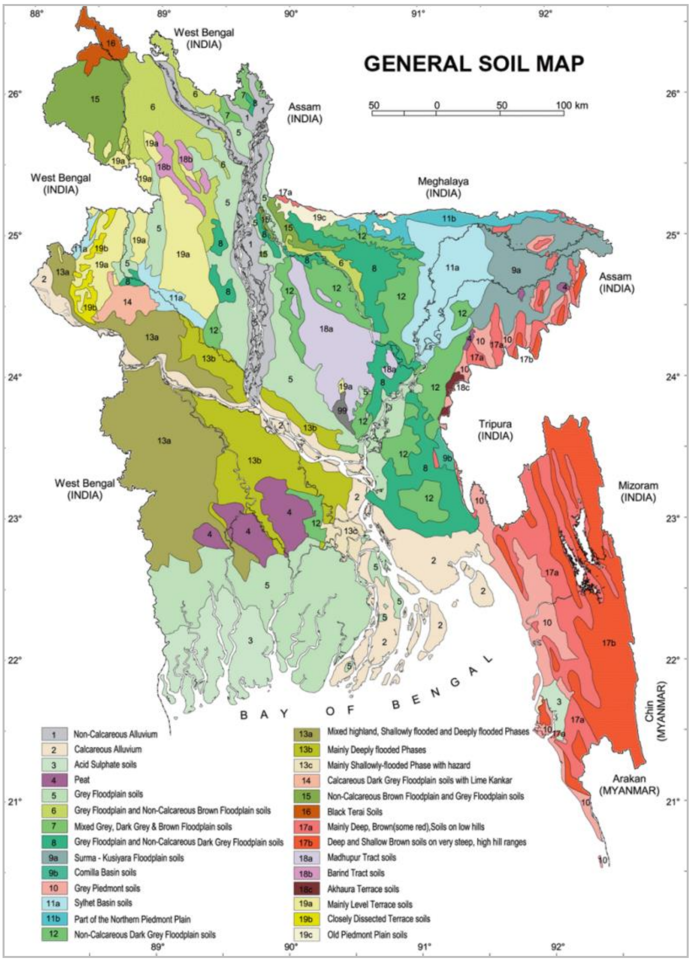

Figure 1. Ggeneral soil map of Bangladesh (Source: Google images, 2014)

The Eastern part of the floodplain is generally smooth, comprising broad ridges and extensive basins. The soils are mainly clay loams on the ridges and clays on the basins or depressions. The Sylhet Basin is a vast depressed area mainly comprising high river levees surrounding extensive basins (haors), the centers of which remain wet in the dry season. Even though the basin is located some 300 km from the coast, the lowest parts of the basin are less than five meters from mean sea level. The relief is locally irregular due to erratic nature of sedimentation during flash floods. Clay soil predominates in this area. Most of the land experiences deep to very deep flooding in the wet season, when the area often resembles an inland sea with substantial waves generated by monsoon winds. The haors remain wet for most or all of the dry season (Sajjadur 2008).

One of the large parts of the country in Sunamgonj, Netrokona, Kishoregonj and Habigonj districts have peat soils near the ground or just few meters below the ground. Peat soils are also found at the fringes of some other lowlands, under a layer of silt or sandy topsoil. All these happened because of morphological changes in the adjacent rivers, when they started carrying sand and silt during floods, and deposited them over the subsiding fossil soil (Haque 2008).

Methodology

Data of the geotechnical profile and its physical and index properties of the study area are collected from different soil testing reports of RRI. The whole vertical profile of the bored soil is considered in this study for the haor areas of Bangladesh such as, Sunamgonj, Sylhet, Moulovibazar, Habigonj, Brahmanbaria, Kishoregonj and Netrokona. Field survey data as well as laboratory testing data are collected and analyzed in 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. Geologists of BWDB collected the disturbed soil samples in the polythene bag and undisturbed soil samples in the Shelby tube with SPT value and ground water level for each specific location.

Skilled scientists and technicians of RRI tested the samples with great care. Calculations are done 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. Preliminary idea about soil strength is developed through SPT value and settlement characteristics are 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 (%) (Arora, 2010)

Table 2. 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

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

Table 3. 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 explore the soil properties around haor areas of Bangladesh in brief, seven haor districts are considered such as, Sunamgonj, Sylhet, Moulovibazar, Habigonj, Brahmanbaria, Kishoregonj and Netrokona districts, which have been shown in Table 4.

Table 4. Locations of different districts

Locations	Thana	District
Basuri Shologhar Dohar	Jagannathpur Sunamgonj Dharmapasha	Sunamgonj
Birakhali Khal Bosonter Khal, Debottor Khal and Tirashar Khal	Jokigonj Kanaighat	Sylhet
Halkata Khal Ranimura Shawnchara and Hazipur Chautraghat and Mritinga	Moulovibazar Kulaura Sreemongol Kamalgonj	Moulovibazar
Khowai Bridge (Chowdhury Bazar) Sherpur, Makal Kandi and Jatour Haor	Hobigonj Nabigonj	Habigonj
Sharma Singer Beel (Primary School)	Bijoynagar Brahmanbaria	Brahmanbaria
Jiul Khal and Shizli Khal Batilonga (Gobinda)	Itna Astagram	Kishoregonj
Baganir Khal, KallerBeel and Ranichapur Bhabanipur&Meharjani Khal Balia Dubail Khal	Khaliajuri Durgapur Purbodhola Madan	Netrokona

Relevant parameters such as depth, soil type, colour, natural moisture content, plasticity, SPT value, particle size, stiffness and relative density of soils are presented in Table 5. Some comparison graphs of depth, SPT, plasticity index, natural moisture content and particle sizes with depth are presented district wise in Figure 2 and Figure 3 respectively.

Table 5. Soil testing parameters of haor areas of Bangladesh

Location	Depth in ft(')	SPT value	Colour	N.M.C in%	L.L in%	P.I in %	Particle Size in mm	Strength in kN/m^2/Relative Density	Remarks
Sunamgonj	0-67'	0-13	Brown and grey	25-100	29-72	5-37	0.0014-0.074	0-191.52 (very soft to stiff)/Loose to medium dense	Particle sizes of Dharmapasha soils are about 4.76 mm
	67'-72'	6-30	Grey	Non cohesive soils			0.074-4.76		
Sylhet	0-102'	0-30	Brown and grey	17-752	28-113	8-66	0.0014-0.074	0-383.04 (very soft to very stiff)/Very loose to dense	Some soils are reddish brown
	Occasional layer	4-46	Brown and grey	Non cohesive soils			0.074-0.84		
Moulovibazar	0-72'	3-38	Brown and grey	10-267	28-177	6-92	0.0013-0.84	0-383.04 (very soft to very stiff)/Loose to medium dense	Some soils are black
	Occasional layer	5-16	Brown and grey	Non cohesive soils			0.074-2.0		
Habigonj	0-72'	2-19	Brown and grey	12-153	27-129	6-153	0.0014-0.074	0-383.04 (very soft to very stiff)/very Loose to dense	Fine Sand layers are found in its depth column.
	Occasional layer '	3-45	Brown and grey	Non cohesive soils			0.074-4.76		Cohesive soil layers are found in its depth column
Brahmanbaria	0-37'	4-17	Grey	25-36	35-55	8-27	0.0013-0.074	47.88-383.04 (soft to very stiff)/Loose to dense	Fine Sand layers are found in its depth column
	37'-72'	8-36	Brown and grey	Non cohesive soils					
Kishoregonj	0-28'	0-16	Brown and grey	20-64	35-62	9-43	0.0013-0.074	0-191.52 (very soft to stiff)/Very loose to dense	Exception found in Astagram Thana in where clay layer is upto 50'
	28'-72'	4-90	Grey	Non cohesive soils					
Netrokona	0-28'	1-10	Grey	18-210	18-66	10-58	0.0014-0.074	0-191.52 (very soft to stiff)/ Very loose to dense	
		2-40	Grey	Non cohesive soils			0.074-0.82		

N.B.: SPT- Standard Penetration resistance for Test; N.M.C.- Natural Moisture Content; L.L.- Liquid Limit; P.I.- Plasticity Index

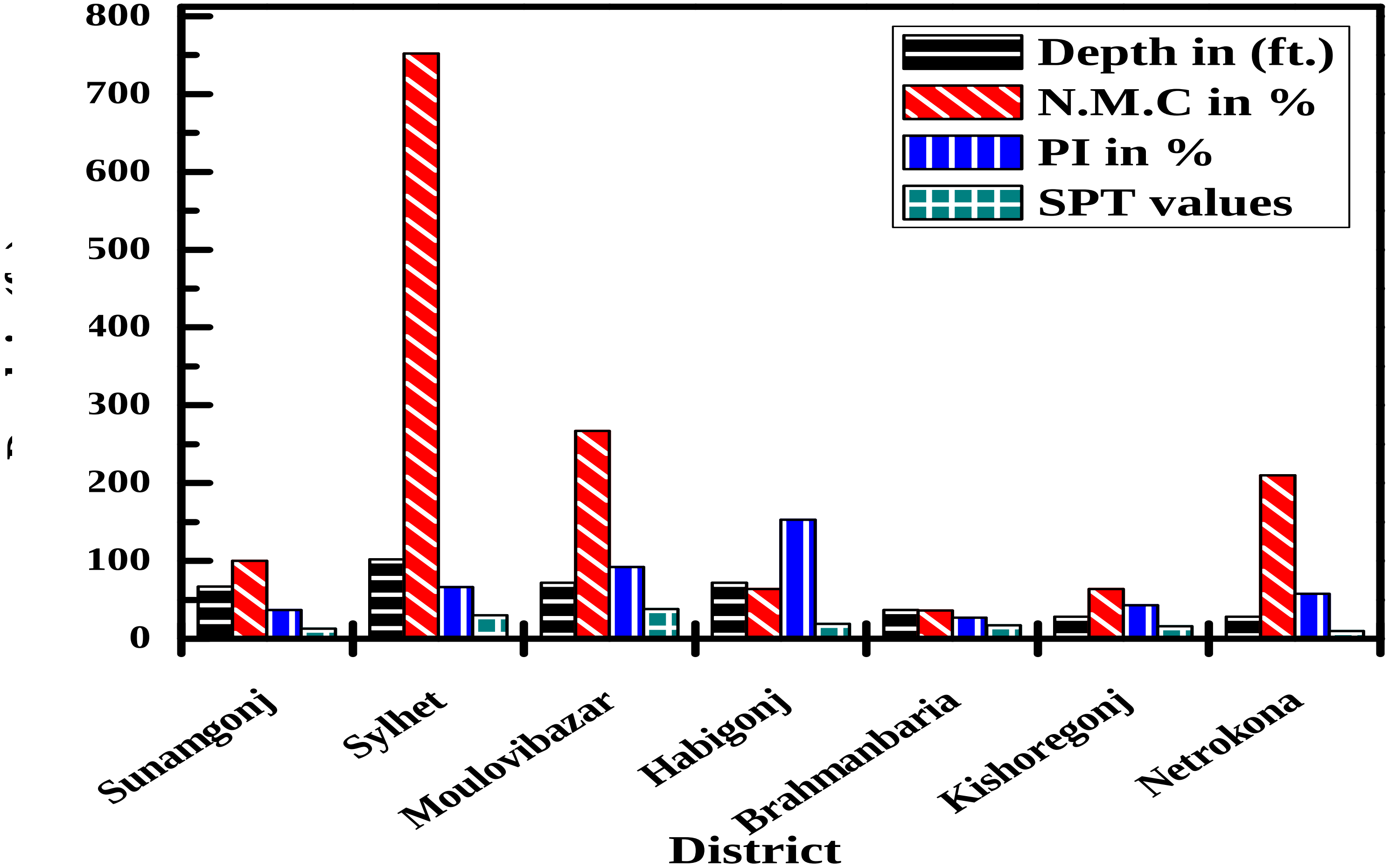

Figure 2. Comparison graph of Depth, NMC, PI and SPT values (district-wise)

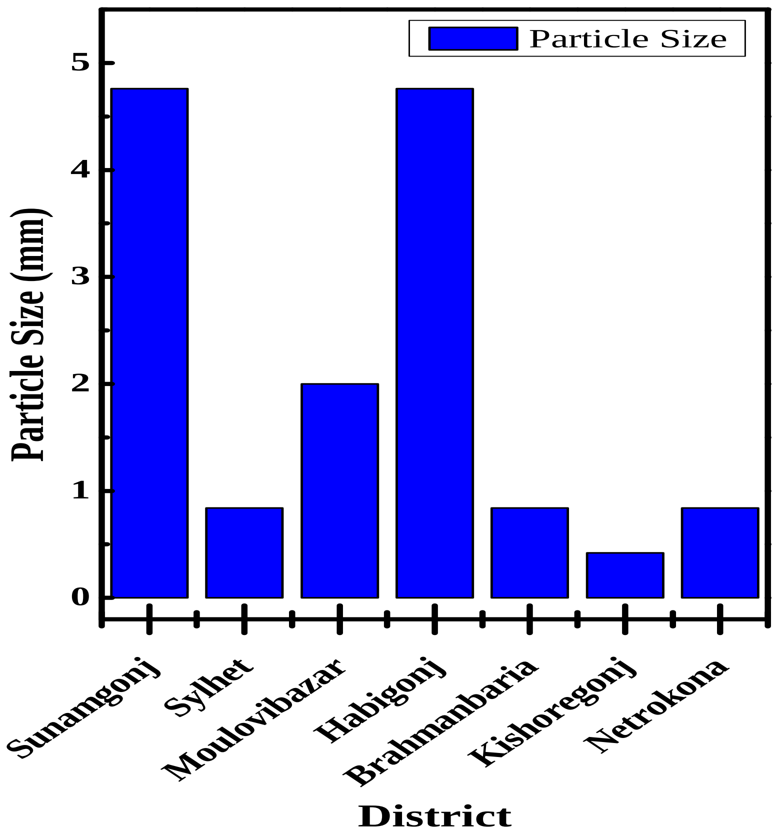

Figure 3. Comparison of Particle Sizes (district-wise)

Soil profile has been described briefly in both tabular presentation and graphical presentation.

In Sunamgonj district, brown and grey in colour cohesive and grey non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-67′), whose sizes vary about (0.0014-0.074) mm and stiffness varies from very soft to stiff and then there are non-cohesive soils up to depth 72′ whose sizes vary from (0.074-4.76) mm and the relative density varies between loose and medium dense. But exception has been found in Dharmapasha Thana's soil layers, whose particle sizes vary up to 4.76 mm (RRI 2014-15).

In Sylhet district, brown and grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-102′), whose sizes vary about (0.0014-0.074) mm and stiffness varies from very soft to very stiff and the occasional non-cohesive soil layers are observed around the hole, whose sizes vary about (0.074-0.84) mm and the relative density varies from very loose to dense. But exception has been found in colour because some soils are reddish brown (RRI 2009; RRI 2010; RRI 2013-14; RRI 2014-15).

In Moulovibazar district, brown and grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-72′), whose sizes vary about (0.0013-0.074) mm and stiffness varies from very soft to very stiff and the occasional non-cohesive soil layers are observed around the hole, whose sizes vary about (0.074-2.0) mm and the relative density varies between loose and medium dense (RRI 2013-14).

In Habigonj district, brown and grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-72′), whose sizes vary about (0.0014-0.074) mm and the 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 (RRI 1995; RRI 1996; RRI 2012-13).

In Brahmanbaria district, grey in colour cohesive and brown and grey non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-37′), whose sizes vary about (0.0013-0.074) mm and the stiffness varies from soft to very stiff and then they are non-cohesive soils upto the maximum depth of about 72′, whose sizes vary about (0.074-0.84) mm and the relative density varies from loose to dense (RRI 2010).

In Kishoregonj district, brown and grey in colour cohesive and grey non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-28′), whose sizes vary about (0.0013-0.074) mm and the stiffness varies from very soft to stiff and then they are non-cohesive soils upto the maximum depth of about 72′, whose sizes vary about (0.074-0.42) mm and the relative density varies from loose to very dense. But exception has been found in Astagram Thana, in where cohesive soil layer is up to (0-50´) (RRI 1992; RRI 2011).

In Netrokona district, grey in colour cohesive and non-cohesive soil layers are observed. Cohesive soil layers are found upto the maximum depth of about (0-28′), whose sizes vary about (0.0014-.074) mm and the stiffness varies from very soft to stiff and then the non-cohesive soils upto the depth of about 72′, whose sizes vary about (0.074-0.84) mm and the relative density varies from very loose to dense (RRI 2015-16).

Conclusions

In this study, a large number of soil testing data are analyzed at different haor areas of Bangladesh. Finally, a range of results are presented in this paper district wise. From the results it is observed that brown and grey in colour soils are existed in those area up to the maximum depth of about (0-102´). Their natural moisture content of cohesive soils is varying from (25%- 752%). At the same time their plasticity indices are varying from (5%-153%), which has been seen in observation. Of course, their strengths are different, which varies from 0 to 383.04 kN/m^2.On the other hand, relative densities of those soils are varying from very loose to dense.

The presented soil properties and layers would assist the design engineers to develop and visualize about the geotechnical information around the haor area. It is found in most cases that cohesive as well as non-cohesive soils are found in every region with exceptions. The particle sizes of (0.0013-.074) mm cohesive soils are observed in most of the haor districts up to the 102´, and there are some exceptions found in Brahmanbaria, Kishoregonj and Netrokona at which cohesive soil layers are observed up to the maximum depth of about (0-28´). Apart from that, exception is found in Astagram upazilla of Kishoregonj district, wherever cohesive soil layer is observed up to (0-50´). The findings and information of soil characteristics of haor areas are expected to help the design engineer for initial assessment of the proposed structure.

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ANALYSIS ON HYDRODYNAMIC AND MORPHOLOGICAL CHARACTERISTICS OF UPPER GORAI RIVER USING DELFT3D

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Abstract

Owing to the reduction of upstream water flow after construction of Farakka Barrage on the Ganges River, huge amount of sediment loads are settling down on Gorai river bed, hindering the safe passage of flow which contributes to the change in hydrodynamic and morphological characteristics of the Gorai River. Therefore the objective of this study is to investigate the hydrodynamic and morphological behavior of the river Gorai by Delft3D. Salinity parameter is also included in this study. A 25 km reach of the Gorai River, subdivided into 5 monitoring points for the year 2010 was selected for the model simulation. For the analysis, simulation period was divided into two seasons; dry period and flood period. Results reveal that during dry period, simulated velocity, discharge and sediment flow had been 0.1 m/s - 0.25 m/s, 200 m^3/s - 400 m^3/s, 0.07 kg/s - 0.125 kg/s respectively. The water level showed decreasing trend towards downstream and found to be 2 m-3.5m. Modeling results also reveal that in the flood period, velocity ranged from 0.7 m/s-0.9 m/s, discharge from 2800 m^3/s to 4200 m^3/s and water level from 8 m to 9.5 m PWD. It was found that during flood period sediment transport rate increased almost 50% than that of dry period. On an average cumulative erosion of the deep channel within the study area ranges from 0.05 m - 0.10 m and the sedimentation of the sides of the channel ranges from 0.20 m-0.25 m. Chloride concentration showed seasonal decreasing trend from 0.22 ppt during dry period to 0.07 ppt during flood period. It is hoped that this study will help in understanding the hydro-morpho dynamic nature of the river and will help the river regulation authority to undertake appropriate future developments projects.

Introduction

The Gorai River catchment area is 15160 km² and is located between 21° 30′ N to 24° 0′ N latitude and 89° 0′ E to 90° 0′ E longitude, covering partly or fully areas of Pabna, Chuadanga, Kushtia, Rajbari, Faridpur, Gopalgonj, Jessore, Jhenaidah, Magura, Norail, Pirogpur, Borguna, Bagerhat, Khulna and Sathkhira districts of South-Western region of Bangladesh. The river takes off from the Ganges at Talbaria, north of Kushtia town and 19km downstream from the Hardinge bridge and discharges into the Bay of Bengal through the Madhumati and Baleswar Rivers (Islam and Gnauck 2011). The river course is wide, long, meandering and is known to adjust its slope, width, depth and velocity to achieve stable conditions at a specified supply of water and sediment (BWDB 2011).

Being a riverine country, Bangladesh is bestowed upon by the innumerable resources from the rivers. From the time immemorial, rivers have played their part in forming the lifeline for the country, which only recently is facing problems due to natural and anthropogenic reasons. In general diversion of river flow in the upstream, salinity intrusion, excessive sedimentation causing navigability

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disturbance and consequent flooding are the major problems relating to rivers in our country (Shamsad et al. 2014). Most rivers in South-Western region (SWR) of Bangladesh depend on water flow from Ganges River. Many of the branches of Ganges River are blocked off from Ganges River due to the water intake at the upstream Indian Farakka Barrage that was built in 1975. Thus the flow volume declines during the dry-season and it impacts the region including coastal areas and Sundarbands (Banglapedia 2015).

Gorai River, a branch of Ganges River, is one of the major sources of freshwater supply to SWR and is the only one remaining branch river. However, in at least these twenty years, its flow volume in the dry-season (December – April) has been declining considerably (Islam 2005). It has a serious environmental impact: especially along the coastal areas around the sanctuary forests where the salty water has increasingly been intruding. This study has been conducted being motivated by the fact that the Gorai river is immensely important for water supply in the south-western region of Bangladesh and the brewing problems of low flow and increased sediment deposition.

Being an important watercourse for Bangladesh, Gorai River has drawn attention of different national and international researchers and organizations. Islam and Karim (2001); Clijncke (2001) and Sarker (2002) tried to predict the downstream hydraulic geometry of the Gorai river, morphological changes due to dredging and morphological changes in response to the declining flow. Islam and Gnauck (2012) studied on water shortage in the Gorai river basin and damage of mangrove wetland ecosystem in sundarbans. Horeet (2013) under Centre for Environmental and Geographic Information services (CEGIS) focused in his study on the morphological development of Gorai river off-take for the restoration of the river flow. Several modelling studies have been conducted to understand the river behaviour. Kader (2000) studied on effectiveness of pilot dredging in the Gorai River in which effectiveness of the pilot dredging has been studied on the basis of pre dredging and post dredging bathymetry using MIKE 21C. Biswas and Ahammed studied on hydrodynamic characteristics of the river Gorai using CCHE 2D.

Delft3D is a powerful tool for understanding and predicting the river hydro-morpho dynamic behaviour. It is a fully integrated modelling framework that can be used for the computation of flow, sediment transport, waves, water quality including particle tracking such as oil spill and dredging plume modelling. Delft3D solves the two-dimensional depth-averaged flow equations. As Delft3D is essentially comparable to many other hydrodynamic models, these analyses can be used to understand the capabilities of numerical models to simulate river hydrodynamics.

In this study, understanding the predicament of river Gorai in particular, efforts have been ensued to address the hydrodynamic, sediment and salinity analysis of the river with the specific objectives of

- Development of a hydrodynamic, morphological and salinity model of a 25 km reach of the River Gorai.
- To assess the Hydrodynamic parameters namely velocity, discharge and water level
- To assess morphological parameters including the total sediment load and the cumulative erosion-deposition and
- To assess the salinity parameters including only the chloride concentration for the study reach.

Methodology

Selection of study area

Figure 1 shows the Google map of the study area. The study area covers about 25 km reach of the Gorai River flowing from 10 km downstream from the Ganges-Gorai offtake within the kushtia district to the 5 km upstream of the Kamarkhali transit within the Kumarkhali upazilla.

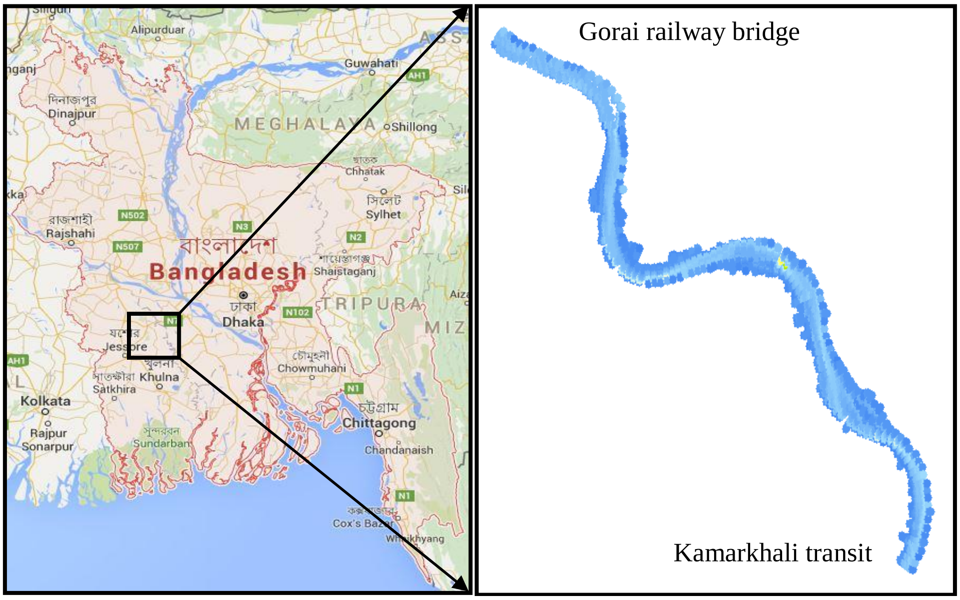

Figure1. Map of the study area (Source: Google images 2015)

Data collection

Data collected for conducting the study are summarized in Table 1.

Table 1. Types and sources of data

Data	Location/ID	Period	Source
Cross-section	25 km reach (GOR-1 to GOR-150)	2010	BWDB
Discharge	Gorai Railway Bridge (SW-99)	2000-2014	BWDB
Water Level	Kamarkhali (SW-101) and Kamarkhali transit (SW-101.5)	2010	BWDB
Sediment	Hardinge bridge (SW-91) and Kamarkhali transit (SW-101.5)	2000-2010	BWDB
Salinity	Amalsar (SW-100) and Kamarkhali transit (SW-101.5)	2000-2010	BWDB

Analysis of flow, salinity and sediment trend of the Gorai River

The average flood flow of the river is 4,500 m^3/s (monsoon period) and the annual average sediment transport is about 50 million tons in which about 40% are fine sand and the rest amount consists of silt and clay. (GRRP-Phase-II, 2014), Discharge data has been processed to obtain the mean annual discharge from 2000 to 2014. It showed that the mean annual flow volume has decreased from 2000 m^3/s to below 500 m^3/s over the last ten years. Due to reduction in flow, sediment concentration shows an increasing trend, which is indicative to riverbed siltation. Salinity data of last ten years show that the maximum Chloride concentration at Gorai Railway Bridge is 260 ppm. Though the concentration is well below the sea salinity (35000 ppm), reduced dry season flow and clogging of the River at off-take, salinity along the Nabaganga-Rupsha-Passur system has largely been influenced. Figure 2(a) shows a clear declining trend of mean annual discharge and Figure 2(b) shows an increasing trend of Gorai river salinity over time. Figure 3(a) and (b) shows increasing trend of sand concentration at two stations namely Gorai railway bridge and Kamarkhali transit for the year 2002 and 2005 respectively.

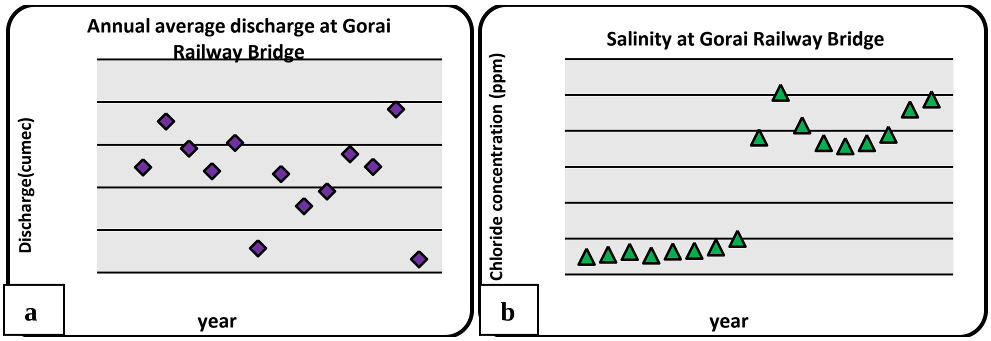

Figure 2. (a) Declining trend of mean annual discharge and (b) Increasing trend of Gorai river salinity

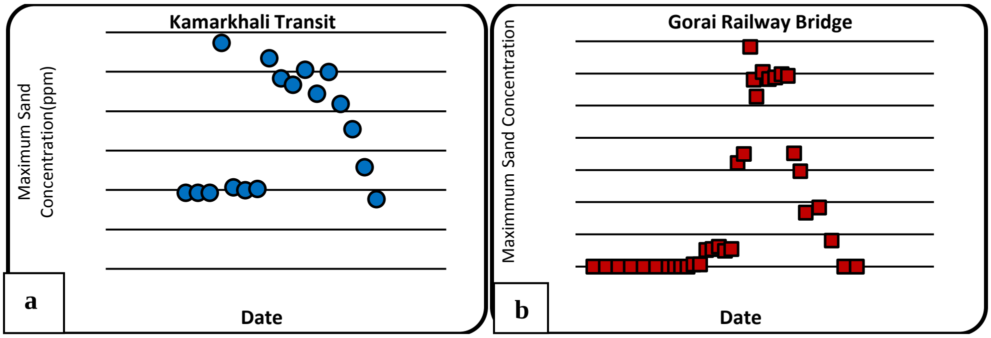

Figure 3. (a) Increasing trend of sand concentration at Kamarkhali transit and (b) Increasing trend of sand concentration at Gorai Railway Bridge

Model schematization

Grid generation

Figure 4 shows the generated grid of the model. In this study a curvilinear grid has been created by simulating a numerical model for 25 km river reach with an average width of 800 m; started from 6 km downstream from the Ganges-Gorai off-take and ended at the kamarkhali transit. The reach was discretized by 720×36 (m×n) grid cells. The average dimension of each grid cell was approximately 40m×40m.

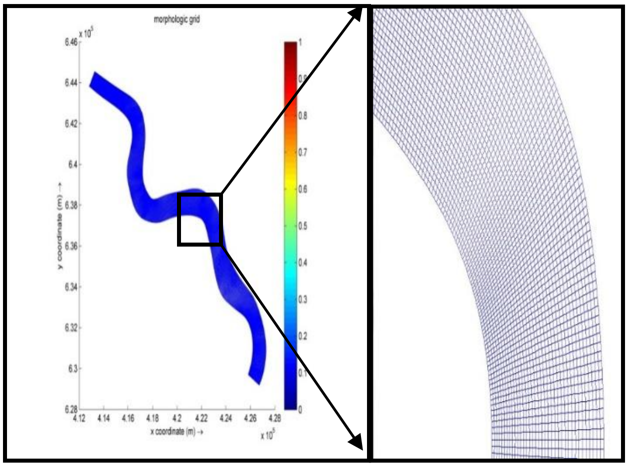

Figure 4. Hydro-morpho dynamic grid of the model

Depth generation

Figure 5 shows the depth and initial bathymetry of the model. After developing the study area with good quality grids, collected cross section data were processed to prepare sample and were imported into the mesh nodes, afterwards it has been interpolated and diffused using triangular interpolation and internal diffusion toolbars to obtain a spatially varying depth file. Bathymetry data was collected during the monsoon period of the year 2010 measured with respect to the PWD datum. 120 cross sections at approximately 200 m interval have been used for the setup of the initial bathymetry of the model.

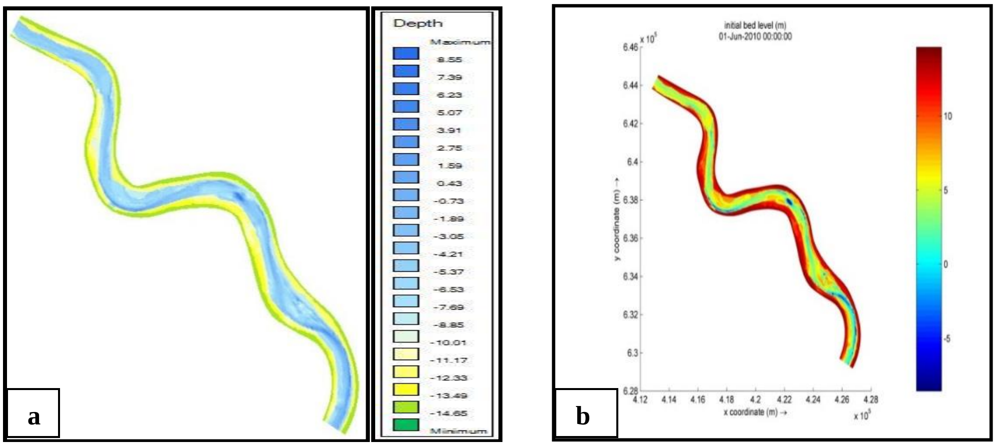

Figure 5. (a) Depth of the study area (b) Initial bed level of the model

Boundary conditions for calibration and validation

Figure 6 shows the upstream and downstream boundary conditions for the calibration and validation of the hydrodynamic model setup. Both flow peaks attained in that particular year during the month of September.

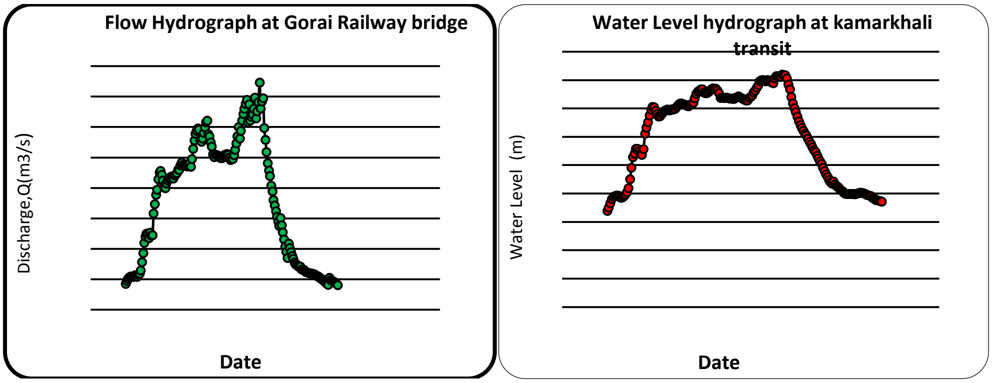

Figure 6. Boundary conditions of the model

The model was simulated for 6 months only from the 1st of June, 2010 to 30th of November, 2010. As the upstream boundary the discharge data of Gorai railway bridge (SW-99) was considered and the water level of Kamarkhali Transit (SW-101.5) was chosen as the downstream boundary.

Calibration and validation of the hydrodynamic model

Figure 7 shows the water level calibration result of the model. For hydrodynamic calibration, computed water surface elevations have been compared with the observed water surface elevations at Kamarkhali station (SW-101). Roughness and eddy viscosity are the parameters that have been used to play to obtain an adequate match with the observed field conditions in the present study. Manning's roughness coefficient has been adjusted after several trial of the model during calibration to an average value of n = 0.025, the value of eddy viscosity has been considered as 10.0 m^2/s.

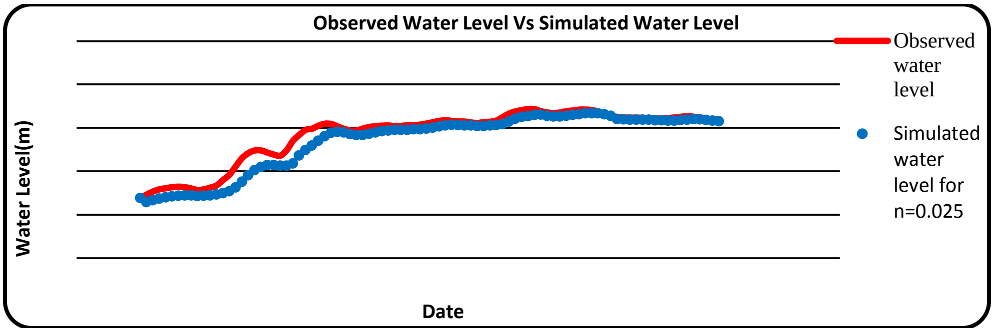

Figure 7. Calibration of the numerical model

Figure 8 shows the validation graph. The model was validated at the Kamarkhali for the period 1st of September to 30th of November that shows a good agreement with the observed data. This result indicates that the model predicted the water level well for the lower discharge condition rather than the peak of the hydrograph.

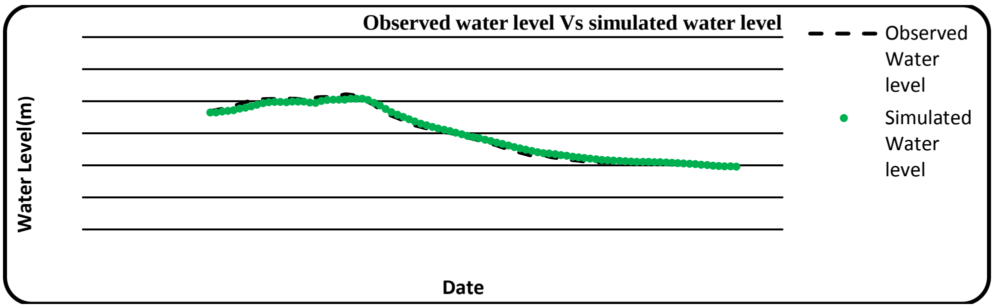

Figure 8. Validation of the numerical model

Calculation of model efficiency

In this study the Root Mean Square Error (RMSE) has been used for the calculation of error and the Nash Sutcliffe coefficient (E) formula has been utilized to calculate the model efficiency. Simulation has been performed for various values of manning's roughness, n and corresponding degree of error of the simulation to the observed data has been plotted to obtain the trial value with minimum error and maximum efficiency. Both of the graphs of Figure 9 shows that the % efficiency is maximum and the RMSE value is minimum for the trial of n = 0.025.

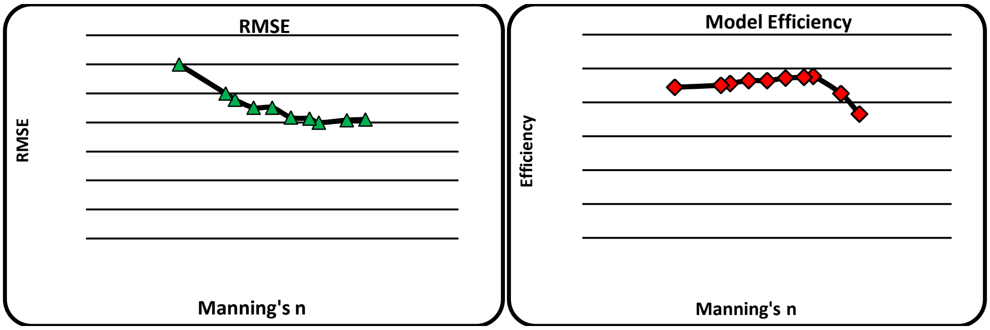

Figure 9. Model error and efficiency

Sediment transport modeling

In this section Delft3D Flow and the DELFT 2D-MOR module have been used for morphological simulation for the year 2010 (1st January to 31st December). Mean sediment diameter (D50) has been taken assumed as 0.150 mm and as the sediment boundary the monthly average sediment data for the year 2010 was given as the input which is shown in Table 2 and Table 3.

Table 2. Upstream and downstream boundary sediment load

Month	**Total sediment Load(kg/s)**	
	Upstream boundary	**Downstream boundary**
January	29.579	1.2
February	13.606	2
March	17.148	2.05
April	15.105	2.35
May	22.468	2.4
June	41.512	5.176
July	69.888	12.325
August	89.815	48.434
September	67.884	53.824
October	31.009	40.79
November	17.967	10.73
December	13.474	1.26

Salinity modeling

Delft3D Flow module with salinity process has been used for salinity modeling for the year 2010 (1st January to 31st December). Table 4 and Table 5 shows the boundary conditions for the salinity modelling

Table 3. Upstream and downstream boundary sediment load

Month	Monthly average Chloride concentration (ppt)	
	Upstream boundary	**Downstream boundary**
January	0.1792	0.183
February	0.182	0.161
March	0.2013	0.18825
April	0.2205	0.18925
May	0.2013	0.187
June	0.1556	0.1863
July	0.08	0.13
August	0.07	0.09
September	0.065	0.07
October	0.09	0.092
November	0.177	0.16875
December	0.1796	0.1842

Defining the monitoring points for analysis

Figure 10 defines the monitoring points which have been used for observing simulated velocities, discharge, water level, total sediment transport, cumulative erosion and deposition as well as the salinity concentration. The monitoring points are named as bend 1, bend 2, bend 3, bend 4, bend 5 from upstream to downstream respectively as shown in Figure 10.

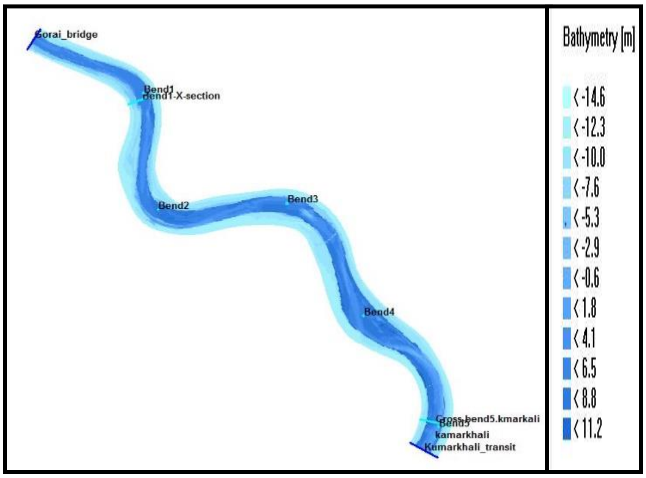

Figure 10. Monitoring points and cross sections

Results and discussion

Analyses of hydrodynamic parameters of study reach

Velocity

Figure 11 and Figure 12 shows the spatial distribution of the simulated depth averaged velocities in m/s. The Figures visualize the changing pattern along the reach during the dry and wet season respectively.The month of january represents the dry period and the month of September represents the wet period.

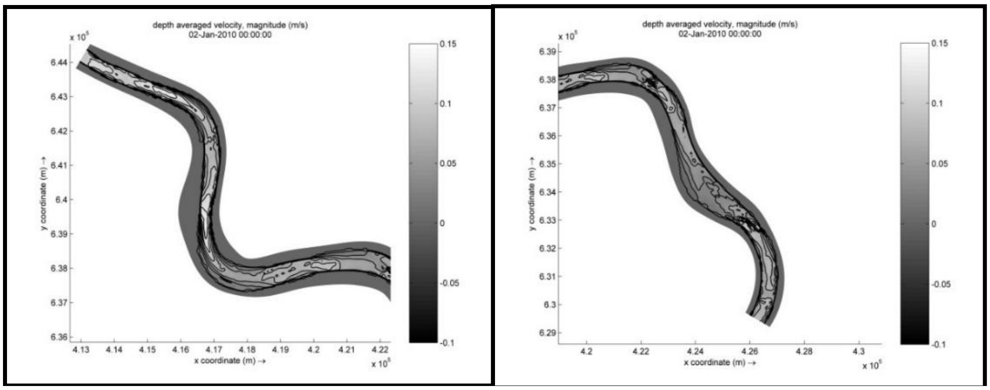

Figure 11. Simulated depth averaged velocity for dry period (January)

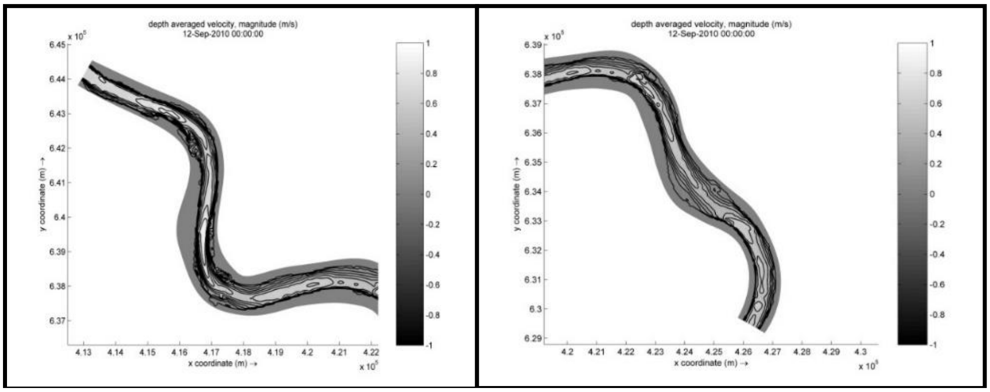

Figure 12. Simulated depth averaged velocity for flood period (September)

Simulated velocity distribution depicts that the peak velocity ranges from 0.7 m/s to 0.9 m/s during the flood period and the magnitude lies between 0.1 to 0.25 m/s during the period of lean flow. Trough and peak magnitudes of the velocities showed that change in velocity along the reach varies from 4%-7% indicating the higher velocity at the upstream areas and decreasing towards downstream. Flow velocity along the study reach decreases due to the changing fluvial processes but the percentage change is not so significant due to small length of the study area.

In progress report number-26 of the feasibility study under Gorai River Restoration Project (GRRP-II), Institute of Water Modeling through comprehensive model test in MIKE 21C found that the flow velocity in upper Gorai river ranging from Goraioff-take to Gorai Railway Bridge was 0.943 m/s for the year 2012-2013 during the month of September and the velocity falls to 0.15 m/s during the dry season for the same time consideration.

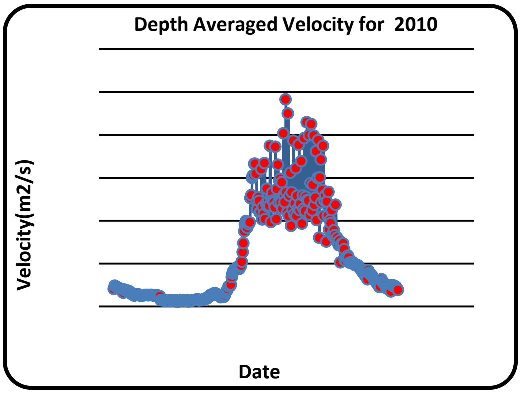

Figure 13. Simulated depth averaged velocity for 2010

From this observation it can be concluded that taking an average of the simulated velocities of the monitoring points to represent the average velocity of the reach of 25 km would not cause much deviation in accuracy. Figure 13 shows simulated velocity profile averaged for the 25 km reach including the monitoring points for the year 2010.

Discharge

Figure 14 (a) and (b) shows the spatial distribution of the depth averaged unit discharge during dry and wet season respectively. Simulated depth averaged discharge distribution depicts that the magnitude ranges from 0.5 m^2/s to 1 m^2/s during the dry period of the year 2010 and 8 m^2/s to 10 m^2/s during the flood period.

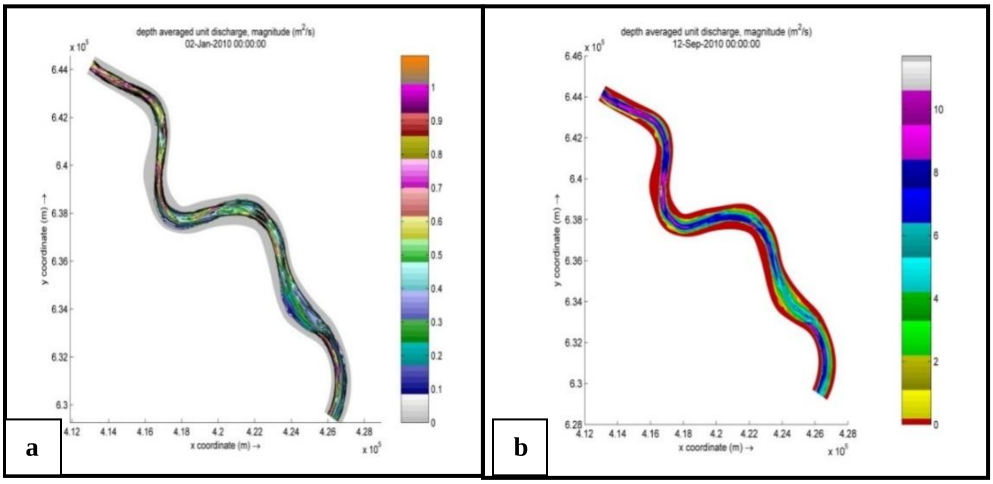

Figure 14. Depth averaged unit discharge (a) during dry period; (b) during wet period

Simulated discharges at 5 different monitoring points are plotted to visualize the changing pattern and the length-wise difference in magnitude of discharge. Figure 15 shows the simulated instantaneous discharge along the study reach at 5 monitoring points. Simulated discharge during the flood period ranges from 2800 m^3/s to 4200 m^3/s. Trough and peak magnitudes of the discharge show that change in discharge along the reach varies from 12%-17% with maximum discharge at the bend 1 (1st monitoring point) and minimum at bend 5 (5th monitoring point). In progress report number-26 of the feasibility study under Gorai River Restoration Project (GRRP-II), Institute of Water Modeling through comprehensive model test in MIKE 21C found that the flow discharge in upper Gorai river ranging from Gorai off-take to Gorai Railway Bridge was 3343 m^3/s for the year 2012-2013 during the month of September and the value falls to 250 m^3/s during the dry season for the same time consideration.

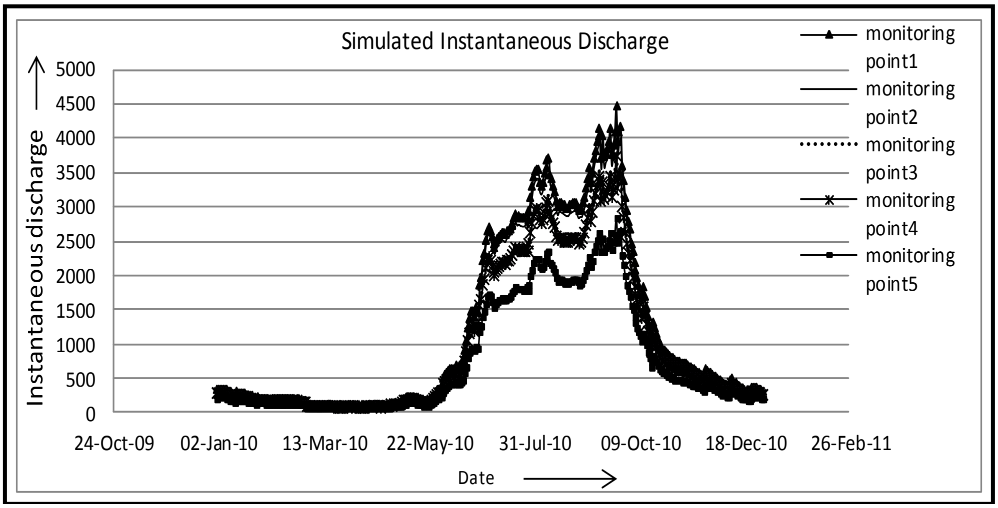

Figure 15. Simulated instantaneous discharge along the study reach at 5 monitoring points

Water level

Simulated water levels at 5 different monitoring points were plotted to visualize the changing pattern and the length-wise difference in magnitude. Figure 16 shows the simulated water levels at 5 monitoring points. Water depth increases along the reach as the flow velocity and discharge decreases along the reach. But the simulated water levels showed a decreasing pattern along the reach as the bed level lowers along the bathymetry at an average slope due to land topography. During dry period simulated water level varies from 2 m to 3.5 m and the level rises to 8 m to 9.5 m during flood period.

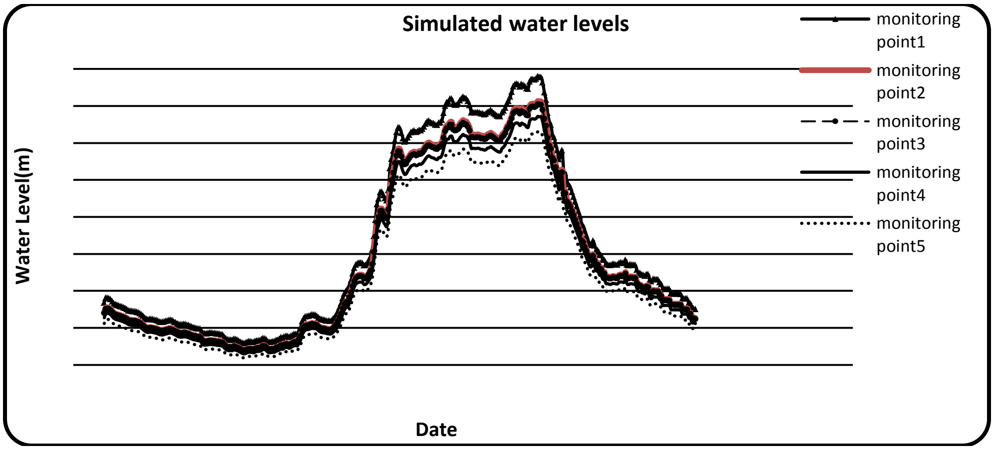

Figure 16. Simulated water levels at five monitoring points

Analyses of sediment transport of study reach

Total sediment transport

Figure 17 shows the simulated sediment transport rate along the study reach. Discharge and flow velocity are of major importance to determine whether deposition or erosion will occur. A general increase can usually be observed in suspended sediment concentration with increasing water discharge (A Guide to Use of Biota, Sediments and Water in Environmental Monitoring 2ndedition). Along downstream the channel decreases due to retarding flow velocity, which causes the decrease of total sediment transport of the channel. The simulated sediment transport rates follow the decreasing pattern along the reach. During the period of lean flow sediment transport rate varies from 0.07 kg/s to 0.125 kg/s and the transport rate rises to 0.22 kg/s during the peak flow season

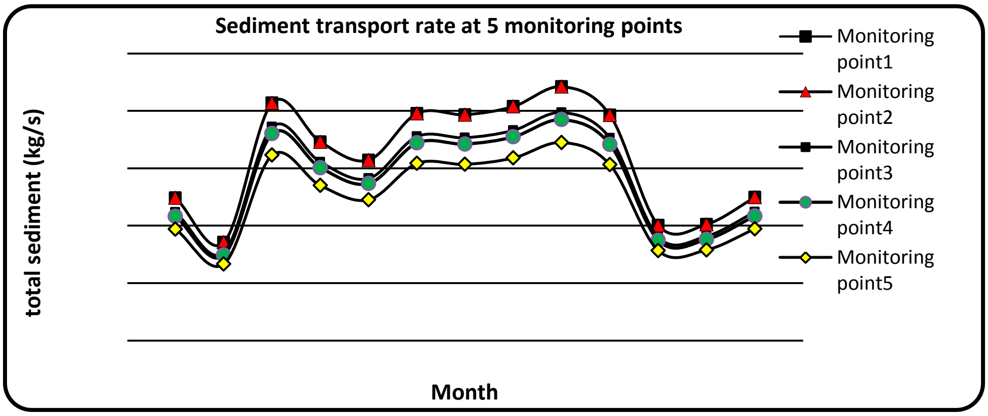

Figure 17. Simulated sediment transport rate along the study reach

It is evident from Figure 19 shown below that in case of natural bathymetry, erosion occurs along the deepest channel (thalweg) and the deposition of sediment occurs near the bank lines. On an average cumulative erosion of the deep channel within the study area ranges from 0.05 m - 0.10 m and the sedimentation of the sides of the channel ranges from 0.20 m - 0.25 m. During the dry period of the year 2010 (mostly in January) change in cumulative sediment deposition along the study reach is minor due to very less discharge. It is evident from the figures of cumulative erosion and deposition that the zones with higher velocity carry more sediment than the other zones.Table 6 shows the cumulative erosion and deposition during the wet and dry season.

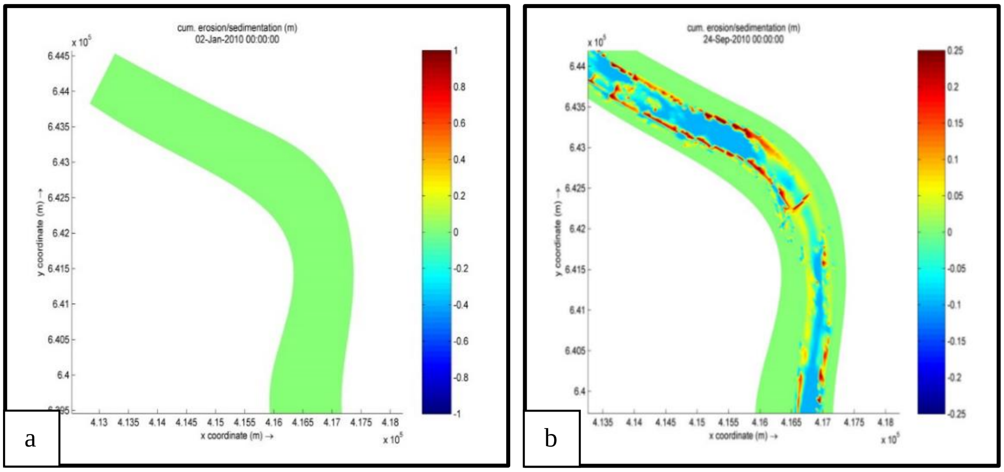

Figure 18. Cumulative erosion sedimentation (a) Dry period; (b) Wet period

Table 6. Net erosion-sedimentation

Season	Cumulative erosion (m)	Cumulative sedimentation (m)
dry	0.0	0.18
Wet	0.15	0.15

Salinity concentration

Chloride concentration

To represent the salinity along the study reach only the chloride concentration has been considered. Figure 19 shows that the reach wise variation in chloride concentration indicating that the values are well below the sea salinity. Chloride concentration is the maximum (0.22 ppt) during the month of January and February and falls to the value of 0.07 ppt during the monsoon flow. Spatial variation of the concentrations at 5 monitoring points for the year 2010 is plotted in Figure 20 that depicts that the concentration variation is negligible at the upper Gorai reach.

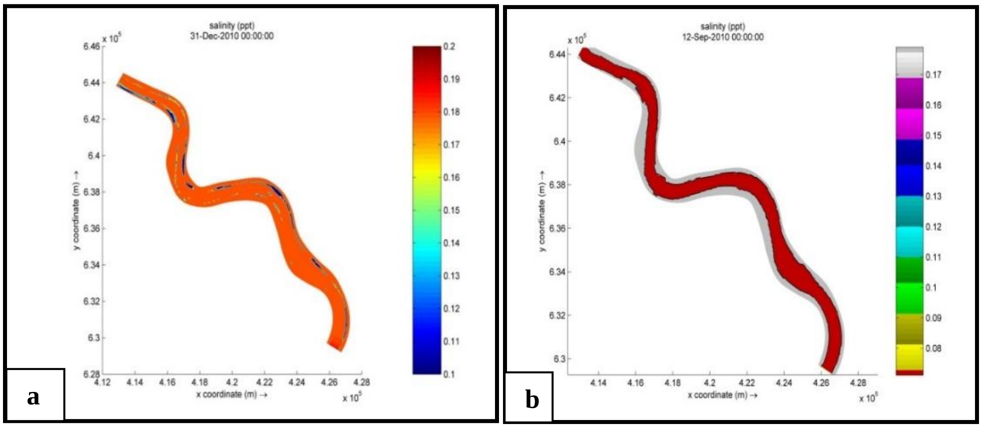

Figure 19. Chloride concentration (a) Dry period; (b) Wet period

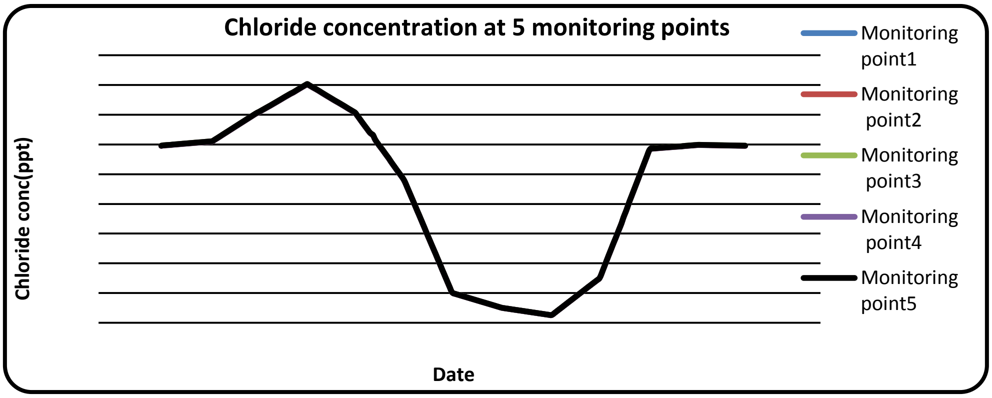

Figure 20. Simulated chloride concentration at five monitoring points

Conclusion

Numerical modeling technique of Delft3D has been applied in this study in case of a 25 km reach of Gorai river extending from Gorai Railway Bridge to Kamarkhali. April 2010 field data of bed level has been taken as reference bed level and the discharge, water level, sediment and salinity data of the corresponding year have been taken as the boundary conditions. The model has been verified by calibrating and validating with the help of iteration of the calibration parameters manning's n and morphological scale factor, Morfac. A value of n = 0.025 and Morfac = 10 was found to be most efficient while comparing the simulated and observed data set. At 5 different monitoring points, different simulated hydrodynamic parameters (Velocity, Discharge and Water level), morphological parameters (Cumulative sedimentation- erosion, total sediment load) and salinity parameter (Chloride concentration) were observed and recorded. The hydrodynamic, morphology and salinity analysis showed that simulated velocity, discharge and sediment flow had been 0.1 m/s-0.25 m/s, 200 m3/s - 400 m3 /s, 0.07 kg/s - 0.125 kg/s. The water level showed decreasing trend towards downstream and found to be 2m-3.5m. Modeling results also reveal that in the flood period, velocity ranged from 0.7 m/s-0.9 m/s, discharge from 2800 m3/s to 4200 m3/s and water level from 8m to 9.5 m. It was found that during flood period sediment transport rate increased almost 50% than that of dry period. On an average cumulative erosion of the deep channel within the study area ranges from 0.05 m - 0.10 m and the sedimentation of the sides of the channel ranges from 0.20 m - 0.25 m. Chloride concentration showed seasonal decreasing trend from 0.22 ppt during dry period to 0.07 ppt during flood period. It is hoped that this study will help in understanding the hydro-morpho dynamic nature of the river and will help the river regulation authority to undertake appropriate future developments projects.

Acknowledgements

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DETERMINATION OF ORGANIC CONTENT IN DIFFERENT BOREHOLES IN VARIOUS REGIONS IN KHULNA DIVISION

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Abstract

The study was conducted to find out the organic matter in different boreholes in various regions in Khulna division. Organic content influences the engineering properties of soil. Soil sample were collected from Khulna, Jessore, and Bagerhat up to the depth of 50 feet from the existing ground surface. Organic content was determined by loss-on-ignition method. A compression was made between organic content with Standard Penetration Test value (N-value). Graph was plotted organic content vs. depth. Organic content is also very important for top soil of agricultural lands for well growth of plants. Top soil samples were collected from seven sites of Khulna region up to the maximum depth of 30 cm from the ground surface. From the study it could be shown that (1) from engineering point of view, top layers contain more organic content than subsequent bottom layers and amount of organic content of Khulna district was higher than that of Jessore and Bagerhat, (2) from agricultural point of view the agricultural top soil should have at least 5% organic content. But most of the agricultural soils contain less than 5% organic content in Khulna region.

Introduction

Soil organic content is one of the most vital properties that influence other properties of soil either directly or indirectly. Soil organic matter protects the top soil against erosion and supplies cementing substance for desirable aggregate formation. Excessive amounts of organic residues produce different phytotoxins during their decomposition. Generally more than 5% organic content considered as excessive in nature which may sometimes cause difficulties (Reddi and Inyang 2000). Organic substance in soil range from microscopic incompletely decomposed plant to animal residue dark colored humus. Humus include product of decomposed of organic residue, precipitation of dissolve organic compound and organic molecules in solution. Organic substances are composed of mainly carbon, oxygen and hydrogen. However, different organic parent materials various aerobic and anaerobic condition of degradation and different degrees of humification produce organic substances with a wide range of molecular structure and particle morphology. A high peripheries and flexible cellular structure is the most important characteristics of organic coarse particles which are granular. Organic fine substance usually smaller than 100µm consist of irregular shape such as cell fragment and tissue parts, as well as of globular organic precipitation and smaller than 1µm. Organic fine substances are negative charged and display substantial cat-ion exchange capacity, which increase with degree of humification and strongly influenced by the hydrogen ion in the pore water.

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Objective of this study

- To determine the organic content of soil sample in different boreholes in Khulna region
- To compare the results with known engineering parameters, and
- To determine the organic content in the top soil of agricultural lands

Literature review

Many soil properties impact soil quality, but organic matter deserves special attention. It affects several critical soil functions such as soil structure, particle size distribution etc. It is well established that addition of soil organic matter (SOM) can not only reduce bulk density (D_b) and increase water holding capacity, but also effectively increase soil aggregate stability (Jalal 2014).

Generally good soil conditions are associated with dark brown colours near the soil surface, which is associated with relatively high organic matter levels, good soil aggregation and high nutrient levels (Peverill 1999).

The presence of organic matter in soil is often ignored, although it influences some important properties. Organic matter, although relatively small in volumetric production, significantly affects the water absorbing capacity of soils. In terms of mechanical properties of soils, the organic matter is known to reduce the maximum dry unit weight and cohesion of soils (Franklin et al. 1973).

According to Russel (1960), soil organic matter consists of a whole series of productions which ranges from undecayed plant and animals tissues through ephemeral production of decomposition to fairly stable brown to black materials bearing no trace of the anatomical structure from which it was derived (it is latter material that is normally defined as humus)

The term organic matter generally used to represent the organic constituents in the soil excluding undecayed plant and animal tissue and their partially decomposition products and the soil biomass. The end product of organic matter decomposition is humus (Gupta 1999).

Soil organic matter defined as the totality of organic matter in soil also includes the organisms that live in soil, the soil biomass, although they usually account for less than 5% of the soil organic matter (Russel 1960).

Soil deposits of organic origin

Organic deposits are due to the decomposition of organic matters and usually in topsoil and marshy place. A soil deposit of organic origin is said to peat if it is at the higher end of the organic content scale (75% or more according to some authors), organic soil at the low end, and muck in between. Peat soil deposit is usually formed of fossilized plant materials and characterized by fiber content and lower decomposition or humification. However, there are many criteria existed to classify the organic deposits and it remains still as a controversial issues with numerous approaches available for varying purposes of classification. Soil from organic deposit refers to a distinct mode of behave different than traditional soil mechanics in certain respect. A possible approach is beings considered by the American Society for Testing and Materials (ASTM) for classifying soils having organic contents (OC) which may stated as follows (Edil 1997).

I. OC<5%; little effect on behavior, considered inorganic soil.
II. OC in between 6-20%; effects properties but behavior is still like mineral soil, organic silts and clays.
III. OC in between 21-74%; organic matter governs properties; traditional soil mechanics may be applicable; silty or clayey organic soils.
IV. OC>75%; Display behavior distinct from traditional soil mechanics especially at low stresses; peat.

Peats have certain characteristics that set them apart from most mineral soils and require special considerations for construction over them. These special characteristics include:

I. High nature moisture content (up to 1500%)
II. High compressibility including significant secondary and even tertiary compression.
III. Low strength in nature condition.

Soil formation

Soils are formed by the disintegration of rock material of the earths relatively deeper crust, which itself is formed by the cooling of volcanic magma. The stability of crystalline structure governs the rock formation. As the temperature falls, new and often mare stable minerals are formed. Most natural soils are composed of the breakdown products of rocks, which have been attack by physical, chemical or biological weathering processes (Reddi and Inyang 2000). The weathered materials have been transported and deposited elsewhere as sediments, or may remain in situ as residual soils. The properties of soil deposition depend on the soil forming factors. In general, five independent variables may be viewed as governing soil formation:

i. Climate,
ii. Organisms present,
iii. Topography,
iv. The nature of the parent material and

v. Time.

It is generally established in the soil science literature that any property of soil is invariably linked to these five fundamental soil forming factors (Jenny 1941).

Phase composition of soils

As the result of the interactions among the parent rocks, atmospheric agencies (primarily water and air), and organisms, during their formation, soils consist of four components or phases such as mineral matter, organic matter, water and air. Nature introduces "fluidity" to the inert weathered rock mass through the water and air phases, and this is where the challenges of predicting the engineering behavior of soils arise. The myriad factors responsible for the co-existence of the four components during soil formation impact a great variety of properties to soils. The need to know the relative proportions of these components of soils property leads us to the soil composition at a scale finer than that of soil profiles. The typical volumetric composition of the four phases of soils is shown in the following Figure 1. Among the four components, organic matter occupies the least amount of volume, and its quality decreases with depth below the ground surface.

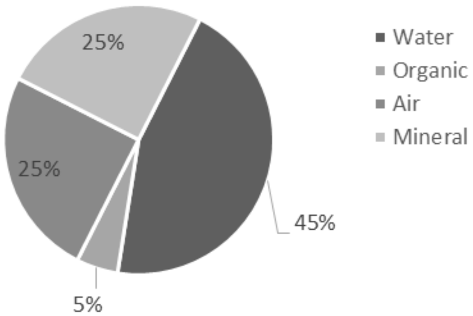

Figure 1. Typical volumetric composition of the four components of soils (Source: Islam 2006)

Significance of organic soils

Any soil containing a sufficient amount of organic matter to influence its engineering properties is called an organic soil. The amount of organic matter is expressed in terms of organic content, which is the ratio between the weight of organic matter and the oven dried weight of sample. The weight of organic matter can be determined by heating the sample to ignite the organic substances (McFarland 1959).

Natural soil deposits may contain a very small percentage of organic matter. Generally a relatively small percentage will contribute sufficient undesirable characteristics (Teng 1997). In some special applications (e.g. soil-cement) only a fraction of one percent may render the soil undesirable.

Organic matters are derived principally from plant life and occasionally from animal organisms. They are found in the following forms:

i. Top soil (loam): the upper layer of ground, usually several inches deep;
ii. Leached stratum: organic matter accumulated on an impervious layer from leaching through upper previous soil; and
iii. Organic deposits: peat, swamp, lignite, coal, etc. In Engineering literatures the term muskeg is sued in Northern United States and Canada to denote a terrain consisting of swamp, bog, or other peat deposits.

Soils containing high organic matter will, evidently, have the following undesirable characteristics:

i. Low shear strength;
ii. High compressibility;
iii. Spongy structure which deteriorates rapidly; hence, results in subsidence without external load; and
iv. Acidity and other injurious characteristics to construction material.

Importance of organic matter on soil properties

The presence of organic matter in soils is often ignored, although it influences some important properties. Organic matter, although relatively small in volumetric proportion, significantly affects the water absorbing capacity of the soils. In terms of mechanical properties of soils, the organic matter is known to reduce the maximum dry unit weight and cohesion of soils (Franklin et al. 1973). The cation-exchange capacity may increase significantly when the organic matter is present in soils. On an average, the cation-exchange capacity of the soil increases by 2mEq/100 g for each 1% of well-humified organic matter (Lyon et al. 1952). Organic matter is also a very good source of nutrients, an important consideration in bioremediation of contaminated sites.

- Organic matter is the source of 90-95% of the nitrogen in unfertilized soils.
- Organic matter can be major source of both phosphorus and available sulphur when soil humus is appreciable amounts.
- Organic matter supplies directly or indirectly microbial action the major soil aggregate forming cements particularly the long sugar chain called polysaccharides.
- When left on top of soil as a mulch, organic matter reduce erosion and surface runoff, prevent rapid moisture loss, keeps the soil cooled in hot weather and warm in the winter.
- Aids growth of crops by improving the soils ability to store and transmit air and water as measured by improved porosity.
- Organic matter binds soil particle into structural units called aggregates. These aggregates help to maintain a loose, open granular condition.

Methodology

The holes were made by driving the casings of 10.16 cm (4″) and 7.62 cm (3″) diameter and the drilling was advanced by chopping method. The disturbed samples were collected by driving standard split spoon sampler of 3.49 cm ($1^{3}/_{8}$″) inner diameter with a 63.5 kg (140 lbs) hammer dropping freely a height of 76.2 cm (30″) in an average and the number of blows required to drive the sampler for every 15.24 cm (6″) penetration over 0.61 m (2′) depth was recorded as a measure of standard penetration resistance-N per 0.3 m (1′) depth. All the samples were collected at an interval of 5′depth. The layer to layer organic content was determined by selecting two boreholes spaced between 10 ft to 30 ft from each borehole. Soil samples were collected from different boreholes maximum depth of 50 ft. Those samples were kept in the oven for 24 hours at 105°C to determine the moisture content. After that the dry sample were kept in the muffle furnace at 550°C for 5 hours. Then the organic content was found out.

Estimates of total organic carbon (OC expressed as C) are used to assess the amount of organic matter in soils. The method measures the amount of carbon in plant and animal remains, including soil humus but not charcoal or coal. Levels are commonly highest in Surface soils (Reddi and Inyang 2000) but wide variations from almost zero to above 15% C are possible. This variation occurs due to soil formation and geologic condition of that particular area. During the soil formation, heavy metals moves downward and the organic content moves up due to their light weight.

Sampling sites for soil layers

For this investigation soil samples are collected from different boreholes of different places. The places are Rajbandh, Khulna Medical College, Judge Court in Jessore, Jibon Nagar in Chuadanga, shrimp investigation center in Bagerhat, Sonaganga residential area and KDA in Sonadanga. In these locations the parties or clients requested for soil testing to the CRTS of KUET. The samples are then collected by making boreholes.

Sampling for agricultural top soil

For agricultural top soil, at first sites were selected. The sites are Rupsha, Khulna, University, Teligati, Daulatpur, Maheshwarpasha and KUET campus. The Site selection represents the uniform distribution of Khulna city. Generally root of crops enter 30 cm into the ground. Three bore hole was done at each location, three sample was collected from each bore hole. First sample as collected the surface of ground. Second sample was collected at 15 cm depth. Third sample was collected 30 cm depth. Those three samples were mixed inside. The sample was packed. Then organic content was determined in lab.

Determining organic content

Organic content are determined by the following steps. Those are given below.

- Disturbed sample was collected from different bore holes at various depths.
- Sample was kept in oven at 105ºC to remove water content. Duration of heating is 24 hours.
- Dry hard and large particles are made to fine soil hammer. For uniformly heated the sample.
- Weight of dry sample was taken
- Samples were kept in Muffle furnace.
- Sample was kept in room temperature for cooling.
- Taking the weight again.

Analysis of the results and the interpretation

The study work was concerned with the variation of organic content in different boreholes of different locations of Khulna, Jessore and Bagerhat. The type of variations was obtained with compared to the shape of standard curves which is elaborated in this chapter. Moreover, variations in organic content in horizontal directions are also observed in agricultural top soil at different locations of Khulna region.

Table 1. Organic content and N-value (KMC, Khulna)

Bore Hole 1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic content (%)	55	21	1.6	15.6	4.8	14.6	6.2	5.7	16.2	14.7
N-value	2	2	3	7	4	7	3	18	8	9

Bore Hole 2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic content (%)	1.7	21.4	5	21	3.1	6.5	5.6	3.1	11.3	6.1
N-value	5	2	3	8	6	5	3	15	11	23

From the above table, it is observed that, the maximum organic content is 55% at the top layer.

Table 2. Organic content and N-value (Judge court, Jessore)

Bore Hole 1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic content (%)	5	5.9	5.1	2.6	19.1	13.2	1.9	6.1	5.3	7.4
N-value	4	2	5	4	4	6	12	9	3	4

Bore Hole 2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic content (%)	7.3	15.4	36.7	6.1	5.3	7.1	9.3	18.1	7.1	9.3
N-value	5	3	2	8	4	4	8	6	5	5

From the above table, the maximum organic content 36.7% lies between 10 to 15 ft layers. The layer to layer variation was very small.

Table 3. Organic content and N-value (Bagerhat)

Bore Hole 1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	4.3	9.0	6.4	5.1	5.5	4.2	3.8	2.8	2.4	2.1
N-value	3	3	2	2	2	2	3	4	4	4

Bore Hole 2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	12.0	7.5	3.1	3.1	3.3	5.5	4.1	2.2	4.1	2.1
N-value	3	2	3	2	2	2	2	3	3	4

From the above table, it was shown that, the maximum organic content exist on the top layer. In both boreholes the idealized variation was seen.

Table 4. Organic content and N-value (Sonadanga, Khulna)

Bore Hole-1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	60	17	9.4	8.1	3.6	16.1	1.2	3.2	4.2	9.3
N-value	4	3	5	5	4	2	3	32	16	9

Bore Hole-2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	35.2	11.4	12.3	5.8	6.9	13.4	5.6	7.1	5.7	7.2
N-value	4	2	3	7	3	3	4	27	21	21

At site Sonadanga, the maximum organic content (60%) was found in the top layer, which is excessive for both structural and agricultural point of view. The graph also show idealized diagram.

Table 5. Organic content and N-value (Jibonnagar, Chuadanga)

Bore Hole 1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	1.5	3.5	2.6	8.6	10.8	2.7	4.4	27.7	5.6	5.9
N-value	4	3	4	5	5	4	11	4	4	6

Bore Hole 2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	2.64	1.9	1.5	4.8	8.5	6.2	3.6	3.0	3.3	4.8
N-value	5	5	4	5	4	11	6	4	4	6

The maximum organic content 27% lies and idealized variation was not seen.

Table 6. Organic content and N-value (Rajbandh)

Bore Hole 1

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	11.7	23.4	44.6	26.66	15.6	18.6	9.2	13.0	5.6	8.3
N-value	4	5	5	4	4	3	2	3	2	2

Bore Hole 2

Depth (ft)	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50
Organic Content (%)	6.8	13.3	26.8	5.8	5.1	6.9	5.8	6.1	3.8	4.1
N-value	3	4	5	4	4	3	5	5	4	3

The maximum organic content was found in the top layer. The graph also showed idealized diagram.

Table 7. Organic content of agricultural top soil in Khulna region

Site name	NO. of boreholes	Organic Content (%)	Average of Organic Content (%)
Dighulia	01	2.40	2.06
	02	1.70	
	03	2.10	
Rupsha	01	3.04	2.70
	02	2.52	
	03	2.81	
Khulna University	01	2.89	3.70
	02	4.70	
	03	4.16	
Teligati	01	3.43	3.57
	02	5.20	
	03	2.10	
KUET	01	1.70	3.80
	02	3.24	
	03	2.65	
M. Pasha	01	2.10	1.70
	02	1.78	
	03	1.22	
Daulatpur	01	4.75	4.02
	02	5.02	
	03	2.99	

Discussion

From our data and observation one type of graph can be established which represents the idealized shape. The results obtained in agricultural top soil is less than the ideal one should be increased.

According to Reddi and Inyang (2000), organic content of soil decreases with the increases of depth. In our observation, the data shows the similar type of result which represents the idealized variation at Jibon Nagar in Chuadanga, there is some variation from the idealized condition. It may be occurred due to the formation of soil and other factors such as temperature, climate etc. Khulna and Bagerhat is situated near the Mangrove forest. This may be one reason for having higher organic content in those regions. For agricultural topsoil, seven sites have been analyzed. All these sites contain less than 5% organic content which should be increased to an extent for the natural growth of plants.

Conclusion

Engineering point of view

From the study, we can found the following information based on our data and observations.

- Amount of organic content in Khulna region is higher than that of Jessore and Bagerhat.
- There is no linear relationship between organic content and depth.
- Generally the top layers contain more organic content than subsequent bottom layers.
- Each layer has at least some amount of organic content. Because Khulna is situated in the southern part of Bangladesh where Mangrove Forest is close at hand. In the soil formation, these regions have some impacts.
- The amount of organic content ranges from 5% to 10% in the most of the soil layers.

Agricultural point of view

The agricultural top soil should have at least 5% organic content. Most of the agricultural soils contain less than 5% in Khulna region. The organic content ranges from 1.70 to 4.02. In a consideration of the maintenance of soil organic content in top layer, the amount of crop residues that must be returned to maintain a certain organic content in soil.

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