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ASSESSING RIVERBANK EROSION AT UPSTREAM AND DOWNSTREAM SIDE OF PADMA BRIDGE BY USING SATELLITE IMAGES

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Abstract

Riverbank erosion is one of the critical and unpredictable natural calamities in Bangladesh that took place in almost every year. This phenomenon takes into account the quantity of rainfall, soil properties, structural stability, river morphology, topography of river floodplain and monsoon floods. In this study an attempt was made to assess the riverbank erosion at upstream and downstream side (Naria Upazila) of Padma Bridge and tried to identify the possible remedial measures. With the aid of satellite images and GIS software, the bank line over the past four decades (1980 - 2019) were delineated and analysis were made to identify probable impact on life and livelihood at Naria Upazila, Shariatpur, Bangladesh due to river bank erosion. Analysis shows, erosion at downstream is more prominent rather than the upstream of the selected reach. During this period total erosion was occurred at downstream side (Naria Upazila) 2823 meter by length (4456 ha by area) where the maximum erosion was happened 2159 meter in between the year 1988 – 1990. On the other hand, total 1995 meter (2823 ha by area) erosion was measured at the upstream side of the Padma bridge in that time (1980 – 2019) where the maximum erosion was happened in between the year 1990 – 1995. Present trend of river bank shifting will disappear the Naria region within a very short period. In this regards, management should take initiative immediately to adopt techniques that work with the natural processes. To sustain the life and livelihood and reducing the riverbank erosion, structural and non-structural measures should be adopted.

Key words: *Riverbank erosion, satellite images, GIS software, remedial measures*

Introduction

Bangladesh is one of the most disaster prone countries around the world with severe cyclone, destructive flood and associated river bank erosion. River bank erosion is one of the natural disasters that cause displacement of inhabitants who previously lived near river banks. (Das, 2010). Impacts of river bank erosion are multifarious: social, economic, health, education and sometimes political. The first and foremost impact is social, i.e., homelessness due to land erosion which compels people to migrate (Iqbal, 2010). More than 700 rivers, with their tributaries and distributaries have criss-crossed the country forming a network of river system (Islam and Rashid, 2011). Shariatpur District (Dhaka division) located in between 23°01' and 23°27' north latitudes and in between 90°13' and 90°36' east longitudes. Naria Upazila is situated in the district of Shariatpur whose total area of 203.58 square kilometres. It borders Zajira Upazila to the west and north, Munshiganj District to the north, Bhedarganj Upazila to the east and south, and Shariatpur Sadar Upazila to the west. The Padma River flows through the northern part of this upazila.

Bangladesh is a disaster prone country (Hossain and Ferdousi, 2004). The Population is 160 million with growing rate of 1.33 per annum (UNDP, 2009) and more than 75 percent of the population lives in the rural areas (Agarwal and Bina, 1990). In recent years, river bank erosion has become a common natural disaster in Bangladesh. More than 310 rivers and tributaries have made this country a land of rivers (Siddique et al., 2014; RIC,2008). A large number of people become homeless due to river bank erosion (Das, 2011). Padma riverbank erosion at Naria upazila, Shariatpur is not a recent phenomenon, from the last few decade people of the Naria upazila has been experiencing high riverbank erosion. Riverbank erosion undoubtedly poses a significant threat to inhabitants of Naria and consequently the economy of surrounding regions. The main focus of this study is to assess the erosion trend and changes of shoreline of Padma River in Naria upazila and identify the probable impact on the local structures and livelihood due to the river

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bank erosion. Also it will be tried to find out the remedial measures to counter the morphological changes at the study area.

Objectives

The specific objectives of this study is given below.

i) To assess the erosion trend and changes of shoreline of Padma River in Naria upazila, Bangladesh.

ii) To identify the probable impact on the local structures and livelihood due to the river bank erosion in the study area.

Methodology

A methodology is the systematic, theoretical analysis of the methods applied to a field of study, or the theoretical analysis of the methods and principles associated with a branch of knowledge. Methodology cannot provide solutions but offers the theoretical underpinning which method can be applied to a specific case. The methodological flow chart of this study is given in the fig. 1

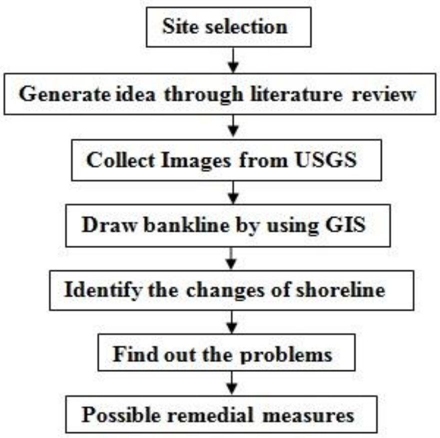

Fig.1: Flowchart of key steps of the study

Study setting and sample collection

For this study upstream and downstream side of Padma Bridge in the Shariatpur district was considered. The special focus was given for finding the trend of riverbank erosion at Naria upazila which is situated in the downstream side of Padma Bridge. Total 9 (nine) figures were collected after 5 (five) years interval from the United States Geological Survey (USGS) in between the year 1980 - 2019. After collecting the images, ArcGIS 10.4 software was used to analyze that figures. The properties of the collected images are given in the table 1.

Table 1. Details of the collected images of Padma River from USGS

Year	Month	Resolution	Satellite
1980	February	60m × 60m	LANDSAT 1
1988	January	60m × 60m	LANDSAT 3
1990	January	30m × 30m	LANDSAT 4
1995	January	30m × 30m	LANDSAT 4
2000	January	30m × 30m	LANDSAT 4
2005	January	30m × 30m	LANDSAT 4
2010	January	30m × 30m	LANDSAT 5
2015	January	30m × 30m	LANDSAT 7
2019	January	30m × 30m	LANDSAT 7

The overview of ArcGIS working process for completing this study was like this

Data Adding ⟹ Creating Shape file ⟹ Bankline Delineation ⟹ Erosion or Deposition Measurement ⟹ Area Calculation ⟹ Mapping

Results and Discussions

Erosion or Deposition at Naria Region

Padma River flows through the northern part of Naria Upazila, Shariatpur. From the analysis of USGS images of Padma riverbank at Naria Upazila, maximum erosion was found 2823 meter and deposition 1787 meter in the time 1980 – 2019. For observing the erosion or deposition at the study area ArcGIS 10.4 software was used. Total scenario of erosion/deposition (in the fixed time interval) at downstream side of Padma Bridge is tabulated in table 2.

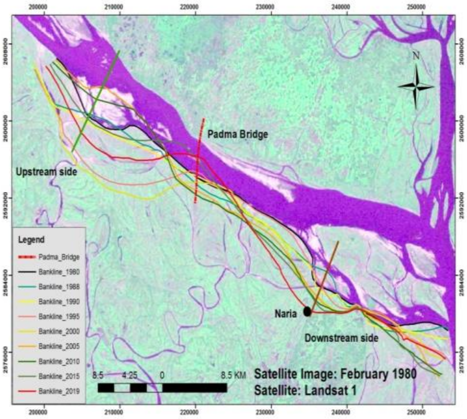

Fig. 2. Changes of Coast line at the study area from 1980 – 2019

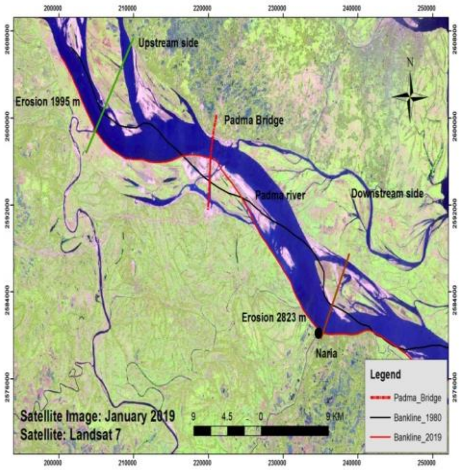

Fig. 3. Erosion at Upstream and Downstream Side of Padma Bridge (1980 – 2019)

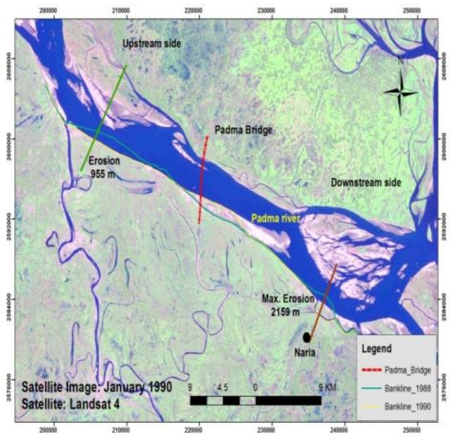

Fig. 4. Maximum Erosion at Downstream Side of Padma Bridge (1988 – 1990)

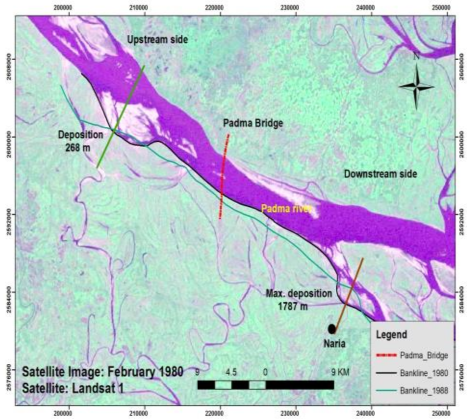

Fig. 5. Maximum Deposition at Downstream Side of Padma Bridge (1980 – 1988)

Table 2. Erosion or Deposition at the downstream side of Padma Bridge (Naria Region)

Erosion/Deposition from 1980 – 2019			Erosion/Deposition with fixed time Interval		
Year	Erosion (m)	Deposition (m)	Year	Erosion (m)	Deposition (m)
1980-1988	…..	1787	1980 – 1988	…..	1787
1990	385	…..	1988 – 1990	2159	…..
1995	…..	158	1990 – 1995	…..	541
2000	1039	…..	1995 – 2000	1198	…..
2005	1117	…..	2000 - 2005	75	…..
2010	1525	…..	2005 - 2010	406	…..
2015	1712	…..	2010 - 2015	180	…..
2019	2823	…..	2015 - 2019	1118	…..

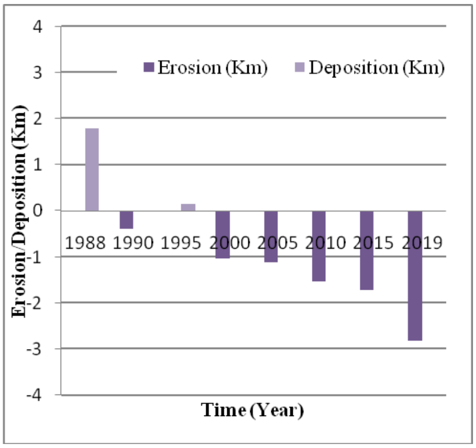

Fig. 6 (a). Riverbank Erosion or Deposition at Naria Upazila from 1980 – 2019.

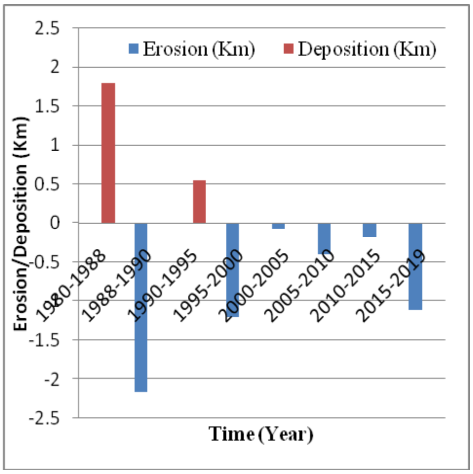

Fig. 6 (b). Riverbank Erosion or Deposition at Naria Upazila from 1980 – 2019.

Erosion or Deposition at the upstream side of Padma Bridge

After collecting the Riverbank images from USGS, ArcGIS 10.4 software was used to calculate the amount of erosion or deposition at the upstream side of Padma Bridge during 1980 – 2019. From the analysis, maximum erosion and deposition was found 3814 meter and 3526 meter respectively. It was also observed the total scenario of erosion/deposition at the fixed time interval which is tabulated in the table 3.

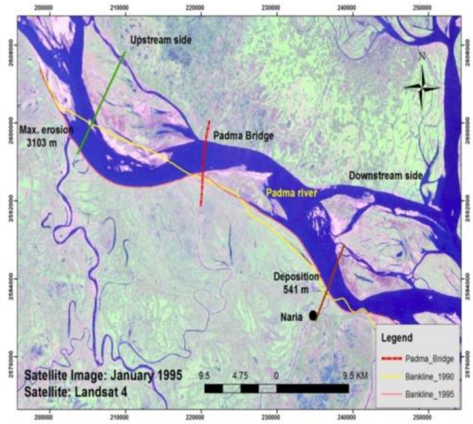

Fig.7. Maximum Erosion at Upstream Side of Padma Bridge (1990 – 1995)

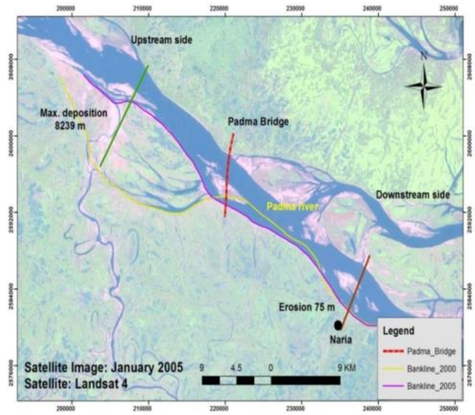

Fig. 8. Maximum Deposition at Upstream Side of Padma Bridge (2000 – 2005)

Table 3. Erosion or Deposition at the upstream side of Padma Bridge

Erosion/Deposition from 1980 – 2019			Erosion/Deposition with fixed time Interval		
Year	Erosion (m)	Deposition (m)	Year	Erosion (m)	Deposition (m)
1980-1988	…..	268	1980 - 1988	…..	268
1990	688	…..	1988 - 1990	955	…..
1995	3814	…..	1990 - 1995	3103	…..
2000	4730	…..	1995 - 2000	910	…..
2005	…..	3526	2000 - 2005	…..	8239

Erosion/Deposition from 1980 – 2019			Erosion/Deposition with fixed time Interval		
Year	Erosion (m)	Deposition (m)	Year	Erosion (m)	Deposition (m)
2010	…..	2030	2005 - 2010	1500	…..
2015	311	…..	2010 - 2015	2325	…..
2019	1995	…..	2015 - 2019	1688	…..

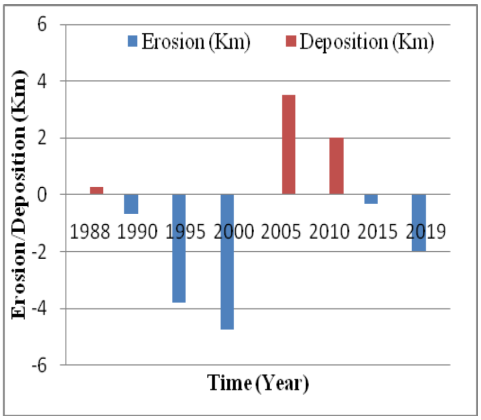

Fig. 9(a). Riverbank Erosion/Deposition at Upstream side of Padma Bridge from 1980 – 2019.

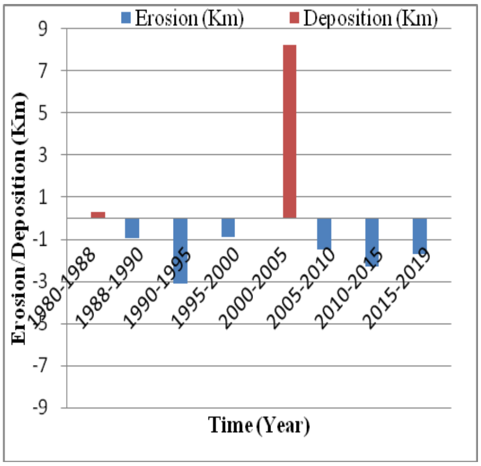

Fig. 9(b). Riverbank Erosion/Deposition at Upstream side of Padma Bridge from 1980 – 2019.

Comparative situation of riverbank erosion or deposition at the study area

Form the bar chart (Fig.10), the trend of erosion or deposition at the study area can be easily understandable. The erosion at the downstream side (Naria) is more than the upstream side of Padma Bridge. In 2005 and 2010 the picture was totally different. In that time deposition happened at upstream side but at the same time erosion was happened at the downstream side (Naria). From 2015 - 2019 erosion happened on both side (upstream and downstream side of Padma Bridge) and the rate of erosion was increasing with time. This trend of erosion is a concerning issue for the inhabitants of that region. If it continues the location of Naria may disappear from his actual position in near future.

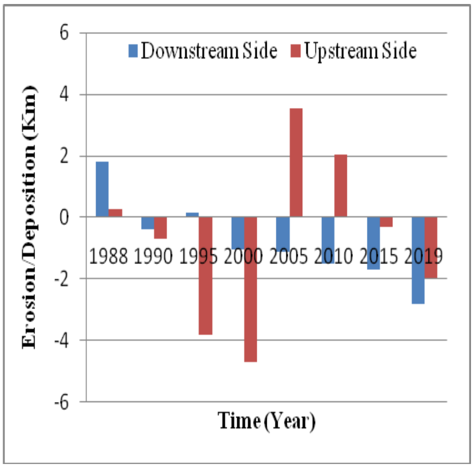

Fig. 10. Erosion or Deposition at the Study Area from 1980 – 2019.

Identification of Vulnerable Location

From the delineated coastlines one vulnerable location was identified in the vicinity of Shariatpur district. The downstream side (Naria upazila) is more vulnerable than upstream side of Padma Bridge which is shown in Fig.11.Over the course of last 39 years total 4456 ha erosion have taken place at the downstream side which is around double from the upstream side of Padma Bridge.

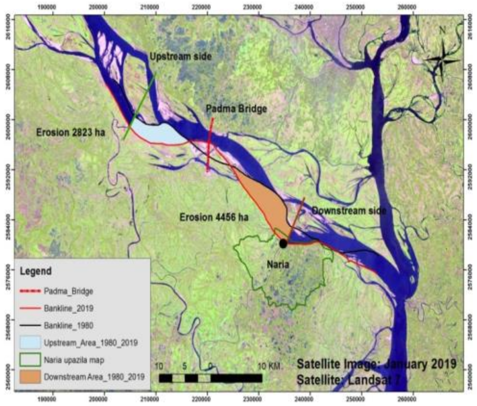

Fig. 11. Eroded area at upstream and downstream side of the study area.

Conclusions

Riverbank erosion of Padma River is not a recent phenomenon. From the last few decade people of the Naria Upazila, Shariatpur has been experiencing high river bank erosion. This type of natural calamities undoubtedly poses a significant threat to inhabitants of Naria and consequently the economy of surrounding regions. Maximum bankline was changed during 2000 – 2019 (near Naria upazila) and situation is continuing. Presently, rate of bank erosion at downstream of Padma bridge is more prominent rather than upstream. Results shows, trend of shifting is rapid and Naria Upazile will disappear in near future. Many families of that area who were totally dependent on agriculture had lost homesteads and agricultural productions which made them socio-economically vulnerable. To sustain the life and livelihood and reducing the riverbank erosion, structural and non-structural measures should be adopted.

Recommendations

The following recommendations can be drawn from this study:

I. Riverbank erosion is a regular phenomenon in the country. Comprehensive river management plan should be made nationally and immediately.
II. Management will need to adopt simply implement techniques to reduce river bank erosion rather than traditional hard engineering.

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PHYSICAL MODELLING OF THE EFFECTS OF STRUCTURAL INTERVENTIONS AND DREDGING AT OFF-TAKE: A CASE SUDY

P. Kanungoe[1*], M. A. A. Moududi[1], M. J. Islam[1], M. Shahabuddin[1], S Afrin[1], Omar Al Maimun[1],

Abstract

The Buriganga is the one of most important river flowing beside Dhaka, capital city of Bangladesh of which water quality has been severely deteriorated due to insufficient river flow, dumping of solid waste, disposal of contaminant effluent from different types of industries, especially effluent of tanneries. Feasibility Study of the Buriganga River Restoration Project reveals that augmentation of dry season flow in the Buriganga River is possible by diverting Jamuna river flow through the New Dhaleshwari River. To this end, structural interventions at the New Dhaleshwari will be needed together with capital and maintenance dredging. In order to augment flow in the Buriganga River during the dry season, a physical model study has been undertaken to determine the efficacy of the sedimentation basin, off-take structure and proposed dredging. To fulfil the objectives, a distorted model having a scale of 1:50 for vertical and 1:200 for horizontal is constructed. The model consists of 4km stretch of the (part width) Jamuna River and 10km stretch of the New Dhaleshwari river (full width). The model bed is prepared according to field survey data of Jamuna and Dhaleshwari rivers. The study shows that due to introduction of the guide bunds, intake canal and sedimentation basin, the discharge of the New Dhaleshwari river was increased from 0.85% to 1.71% ($725m^3s^{-1}$) of corresponding dominant discharge of the Jamuna River. However, after lowering (through dredging and as per design) of the river bed downstream of the intervention location New Dhaleshwari discharge became 3.60% (1510 m^3s^{-1}) of the dominant discharge of the Jamuna. It is also found that targeted dry season flow (141 m^3s^{-1}) augmentation in the Buriganga river is not possible without dredging beyond the intervention location as flood discharge increased by proposed intervention alone is not enough to cause expected lowering of the river bed downstream of the same. Moreover, it is found that the intake canal, sedimentation basin and exit canal would get silted up gradually with time. Annual volume of sediment deposition within the intervention location is $557235m^3$ and of this volume of sediment, 33% will be deposited within the intake canal, 59% within the sediment basin and 8% within the exit canal.

Keywords: *Sedimentation Basin, Dredging, Guide Bund, off-take structure, tailgate, launching apron*

Introduction

The Buriganga is the main river flowing beside Dhaka, capital city of Bangladesh. Over the last several decades the flow of Buriganga, Turag, Shitalakkha and Baluriver has been reduced drastically. As a consequence, the water quality of the river Buriganga has been severely deteriorated due to insufficient river flow, dumping of solid waste, disposal of contaminant effluent from different types of industries, especially effluent of tanneries. In addition, continual growth of population, illegal possession of riverbank and changes of the socio-economic conditions have severely encroached the once famous inland navigation route of Dhaka and Narayanganj. This has created a great nuisance and social problem of Dhaka City. The location of the study area is shown in **Fig 1**.

To ensure sufficient flow in the river Buriganga by diverting flow from the Jamuna through the New Dhaleswari River, Institute of Water Modelling (IWM) to carry out a full scale feasibility and mathematical model study (2008). The study revealed that in order to augment 141cumec dry season flow in the Buriganga river, 245cumec of the Jamuna flow has to be diverted through the New Dhaleshwari river.

It is understood that without sustainable management of the New Dhaleshwari off-take it would not be possible to augment flow in the Buriganga river during dry the season due to large scale sedimentation at the off-take and in the distributary river bed.

In order to ensure diversion of 245cumec of Jamuna flow structural interventions at the off-take in the form of guide bunds, sedimentation basin and dredging will be needed. Through this model investigation, the performance of the proposed sedimentation basin and associated works in inducing siltation within the sedimentation basin and thereby, allowing silt-free water to enter into the New Dhaleswari river has been assessed to determine the efficacy of the proposed plan and design of the same and also to find out the appropriate and cost effective layout of the intake canal, sedimentation basin and exit canal and proper dimension and alignment of the guide bunds at the intake canal.

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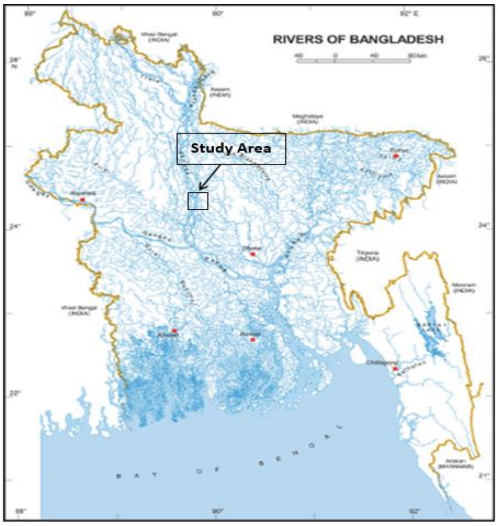

Fig 1. Location of the study area

Objectives of the Study

The objectives of the distorted overall model:

- To verify 141 cumec dry season flow of the Buriganga river by diverting 245 cumec waterfrom the Jamuna river;
- Qualitative assessment of sedimentation and flow distribution at the intake;
- Optimization of location and alignment of sediment basin;
- Performance of sediment basin;
- Morphological assessment at and aroumd the intake qualitatively and
- To get overall idea and sustainability about the system.

Methodology

Study area selection

Keeping the objective of the study in view the study area has been selected. The study area is situated just downstream of the Bangabandhu Bridge and consists of 5km reach (part width) of the Jamuna river and 10km reach (full width) of the New Dhaleshwari river including the off-take.

Data collection and model development

In order to construct the physical model, the primary data include bathymetry and bank line covering the whole study reach, D_{50} of bed and bank materials, water level gradient, point velocities, sediment load and discharges at selected cross-sections both in the Jamuna river as well as in the Dhaleshwari river. Bathymetric survey has been constructed in the selected reach of the Jamuna and the New Dhaleshwari. Discharge has been measured at 3(three) cross-sections (1 in the New Dhaleshwari and 2 in the Jamuna). These measurements have mainly been used for calibration of the model. Therefore, an attempt has been made to measure discharge of the Jamuna River when it is more or less at dominant flow stage. Bathymetric data was collected according to the requirement of the physical model investigation. In the Jamuna river, cross-sections have been measured at an interval of 300m to 400m whereas the same in the New Dhaleshwari river is measured at an interval of 200 m. Some densely spaced cross-sections were measured at the off-take location and wherever necessary for accurate reproduction of prototype bed configuration in the model. GPS survey has been conducted to record the boundary position of the intake canal. The collected secondary data include historical discharges and water levels of the Jamuna recorded at different gauge stations of BWDB. Design drawings of guide bunds, intake canal and sedimentation basin have been supplied by BWDB. Positions of the guide bunds, intake canal and sedimentation basin have also been supplied by BWDB in map form and as numerical data.

An extent of about 5km of Jamuna River extending from 3km upstream to 2km downstream of the New Dhaleshwari off-take has been reproduced in the overall distorted morphological model. The model also includes about 10km reach of the New Dhaleshwari River from the off-take. Partial width of Jamuna River and whole width of Dhaleshwari River have been reproduced in the model. The preliminary layout of the off-take structures has been introduced in the model based on the supplied data. A preliminary layout of the model was given with the reference grid points in the model. Channel planform has been reproduced using these grid points and the bed and bank levels have been fixed up by levelling instrument as per bathymetry using rise and fall method. It has required some cutting and filling of sand from the model. The model was investigated on a mobile bed and hydraulic similarity was established in the model with a distorted scale. The scale ratio was selected as 1:50 for vertical scale and 1:200 for horizontal scale for model construction.

The model is a sand bed morphological model. The model study aims at investigation of the effectiveness of the proposed structural interventions at the off-take to ensure diversion of at least 245cumec of Jamuna river flow into the New Dhaleshwari during dry season. The model has been designed to fulfil both the flow and sediment transport criteria simultaneously. In this physical model, various types of instrument and facilities are needed such as, a sharp-crested weir for measuring flow, point gauge for measuring water level, 3-D current meter for measuring velocity, high resolution camera for taking video and photographic view of model, stopwatch for taking instant time and floats for identifying flow path of flowing water.

The required discharge in the model has been ensured using sharp-crested weir. Flow over the weir has been estimated using Rebock's formula. Model velocity was measured by a 3D velocity meter. Water slope was calculated by analysing the water level measured at different position using point gauges installed in the model. During the model run, flow lines have been identified by dropping floats at the inflow section upstream and by recording their positions from the bank line in the successive downstream sections throughout the entire length of the model and finally catching them at the downstream end of the model. A stopwatch is used to calculate the surface velocity of the flow. Finally, model data have been collected, analyzed and each test results are interpreted.

Similarity condition of the model

The model is designed based on the scale laws and conditions for scale model of the river. In the design of overall distorted morphological model scale conditions related to the three governing processes have to be fulfilled in order to obtain complete similitude between the model and prototype. These processes are (1) flow (2) sediment transport and (3) bed topography. For scaling and design of the model following scale conditions have been taken into account.

i) Roughness Condition

In the model the following roughness condition should be satisfied properly:
$C_r^2 = L_r / h_r$

Where,
C_r = roughness scale
L_r = horizontal scale
h_r = vertical scale
ii) Froude Condition
The scale condition that has to be satisfied reads as:
$F_m < 0.5$ where F_m= Froude number in the model

iii) Minimum Water Depth
Minimum water depth in the model should be maintained for correct measurement of flow velocity.

iv) Sediment Transport Condition:
a) Minimum Sediment transport
The following scale condition should be satisfied to ensure minimum sediment transport in the model.
$V_m > V_{cr}$
where,
V_m= velocity in the model
V_{cr}= critical velocity for sand movement for a particular size
b) Transport Intensity
The following scale condition has to be satisfied for reproduction of the transport intensity when most of the sediment in the prototype is transported as suspended load:
$V_r = C_r D_r^2 \Delta_r$
where,

V_r= velocity scale
C_r= roughness scale
D_r = diameter scale
Δ_r = relative density scale

c) Sediment Transport Direction

The scale condition for transport direction is:

$V^2_r = C_r^2 D_r \Delta_r$

d) Rouse Condition

The Rouse number dictates the mode of sediment transport. It is the ratio of particle settling velocity to the shear velocity (rate of fall versus strength of turbulence acting to suspend particles).

$$P = \frac{w_s}{\kappa u_*}$$

Here, the Rouse number is given by P. The term in the numerator is the (downwards) sediment, the sediment setting velocity w_s. The upwards velocity on the grain is given as a product of the Von Karman's constant, $\kappa = 0.4$, and the shear velocity, u_*. The following table (**Table 1**) gives the approximate required Rouse Numbers for transport as bed load, suspended load, and wash load.

Table1. Approximate required Rouse Numbers for transport

Sl. No.	Mode of Transport	Rouse Number
1	Bed load	$\frac{w_s}{\kappa u_*}$>2.5
2	Suspended load: 50% Suspended	1.2<$\frac{w_s}{\kappa u_*}$<2.5
3	Suspended load: 100% Suspended	0.8<$\frac{w_s}{\kappa u_*}$<1.2
4	Wash load	$\frac{w_s}{\kappa u_*}$< 0.8

In order to reproduce the concentration vertical in the model the following scale condition should be satisfied:

$(V_*/\omega)_r = 1$

where,
V_* = shear velocity
ω = fall velocity

It is evident that different scale conditions will arise in order to satisfy flow and sediment transport conditions discussed above. A compromise should, therefore, be sought that may lead to scale effects to some extent. It is also evident that the model and the prototype roughness play a vital role in selecting the different scale factors. The roughness values may be obtained from previous model studies on the Jamuna river. The design of the model has been made after analyzing the field data. A constant discharge has been considered for the design. Since dominant discharge of a river is responsible for determining its slope and size, the dominant discharge of the Jamuna river is considered for design of the model. Therefore, the same discharge has been considered for model calibration and application tests. Based on the available literature the dominant discharge of the Jamuna river is considered to be 40,000m^3s^{-1}.

e) Shields Condition

The Shields parameters, also called the Shields criterion or Shields number, is a non-dimensional number used to calculate the initiation of motion of sediment in a fluid flow. It is non-dimensionalization of shear stress.

$\theta = \frac{hi}{\Delta d50}$

h = Average depth in meter
i = Slope of the channel/ water surface
d_{50}= Median diameter of the particle in meter
$(s-1) = \Delta = 1.65$
$g = 9.81\ ms^{-2}$

Model Set-up

Outdoor modelling facilities of RRI are used for setting up the planned model. The overall model is constructed in an open-air bed of 100m × 60m. The model setup includes model scale, model discharge, model grid, bathymetry and bank line, re-circulation system, gauging points, tailgates and existing and proposed structures (**Fig 2**).

The sediment has been fed manually at the upstream limit of the model. The design of the existing and proposed structures have been collected from the client and reproduced in the model.

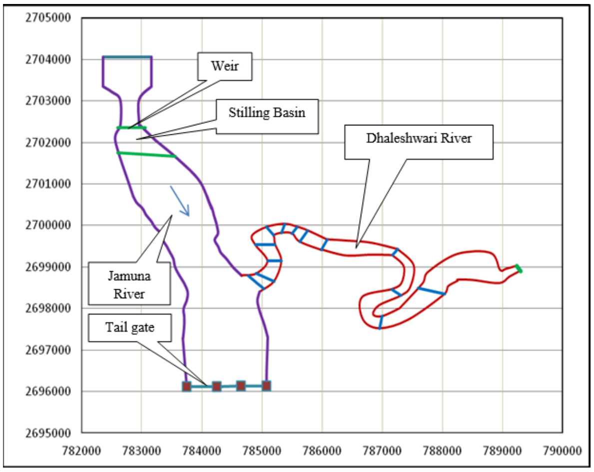

Fig 2. Layout of the model

Calibration of the model and calibration result

Calibration of this model has been conducted in existing condition of the river to ensure that the model is able to reproduce the flow condition, morphological behaviour and sediment transport in the field. The main focus of model calibration has been concentrated on three governing processes namely flow, sediment transport and changes in bed topography.

At the end of the calibration test the discharge through the New Dhaleshwari is found to be as about 0.8% of the Jamuna River. The variation in the New Dhaleshwari discharge with time during the calibration test has been occurred with the developments in the Jamuna bed in response to the imposed conditions for sediment transport similar to that in the prototype.
Calibration of sediment transport depends on the reliable field data of the same. During calibration test sediment has been fed manually at the inflow section of the model. Initially, the rate of sediment feeding has been determined using Engelund and Hansen model. However, at the end of the test the calculated rate has been verified and it is found that equilibrium sediment transport rate in the model is 0.00014 m^3/s for dominant discharge

The necessary measurements taken during the calibration test have been processed and analyzed. The scales of the different basic and derived parameters have been determined based on the calibration test results. **Table 2** presents the characteristics parameters of the dominating processes for the morphological model after calibration. The scale factors for both basic and derived parameters have also been shown in this table. It is noticeable from the table that although the model has been constructed considering a depth scale factor of 50, the actual depth scale ratio after calibration test is found to be 54.64.

Table 2. Characteristics parameters and scale factors obtained after calibration of the model

Parameters	Unit	Prototype	Model	Scale factor
Basic Parameters				
Discharge (Q)	m^3s^{-1}	21172.14	0.39886	53082
Width (W)	m	814	4.07	200
Cross-sectional area (A)	m^2	12454.2	1.1396	10929
Depth (D)	m	15.3	0.28	54.64
Velocity (V)	ms^{-1}	1.7	0.35	4.86
Slope (i)	-	0.00007	0.001	0.07
D_{50}	m	0.00018	0.0001	1.8
Sediment transport (s)	m^2s^{-1}	0.00330	0.000034	97
Sediment transport (S)	m^3s^{-1}	2.686	0.00014	19185
Sediment density	(kgm^{-3})	2650.0	2650.0	1
Relative density	-	1.65	1.65	1
Derived Parameters				
Chezy (C)	$m^{1/2}s^{-1}$	52	21	2.48
Froude Number (F_r)	-	0.14	0.21	0.67
Shear velocity (u*)	ms^{-1}	0.102	0.052	1.96
Critical Shear velocity (u*c)	ms^{-1}	0.0124	0.0124	1.00
Fall Velocity (w)	ms^{-1}	0.022	0.009	2.44
Sheilds parameter (Ө)	-	3.61	1.70	2.12
Critical Shields parameter for motion (Өcrm)	-	0.053	0.095	1.8
Critical Shields parameter for suspension (Өcrs)	-	0.074	0.097	0.76
Rouse parameter (w/kU_*)	-	0.536	0.429	1.25
Rouse (suspension) number (U_*/w)	-	4.66	5.82	0.80
Reynolds particle parameter (Re_*)	-	18.45	5.24	3.52
Reynolds critical particle parameter (Re_*c)	-	2.24	1.24	1.81
Non-dimensional particle paramete (D_*)	-	4.55	2.53	1.8
Critical velocity for motion (Vcrm)	ms^{-1}	0.37	0.28	1.32
Critical velocity for suspension (Vcrs)	ms^{-1}	0.44	0.27	1.63
Critical depth (hcr)	m	0.316	0.0156	20.25
Weigh function of influence of bed slope (fӨ)	-	0.5674	1.0837	0.52
Mode of oscillation	-	1.0000	1.0000	1.00
Flow adaptation length (λ_w)	m	2104	6.24	337.0
Adaptation length for sediment transport (λ_s)	m	2493	6.50	384
Interaction parameter (lP)	-	1.18	1.04	1.14
ID bed celerity (C_{bw})	ms^{-1}	0.00108	0.00091	1.18
ID morphological time scale	days	42.93	0.2542	168.90
2D Morphological time scale	days	376	3.33	113
Aspect ratio	-	53.203	14.536	3.66

The equilibrium water level slope is found to be 0.001. Calibration of morphology has been concentrated on achieving an equilibrium condition in the model bed configuration similar to the initial bathymetry of the model. The observed (initial) *bathymetry* (monsoon 2017) and calibrated bathymetry is shown in **Fig 3** and the observed (initial) velocity (monsoon 2017) and calibrated velocity distribution at prefixed cross-section is shown in **Fig 4.**

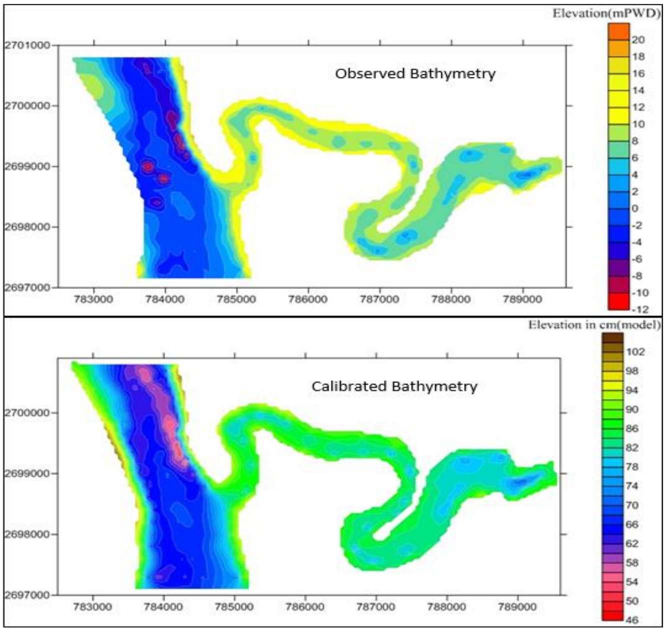

Fig 3. Initial and final bed topography in the model during the calibration test

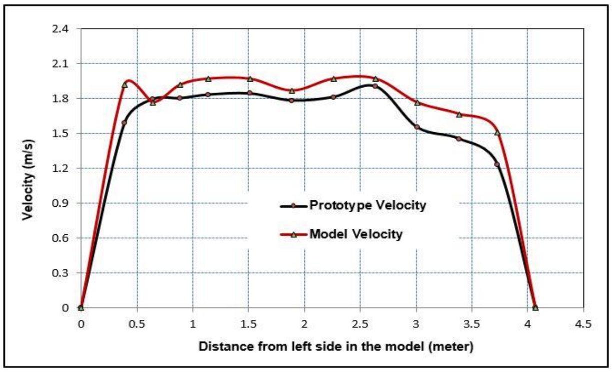

Fig 4. Comparison between prototype and model velocity distribution at prefixed cross-section

Test scenarios

The conducted tests comprise of base run (T0) and application tests (T1-T7). The base run has been conducted without any proposed structural interventions in place for a constant discharge (dominant discharge). The application tests have been conducted with proposed structural interventions in place and for the same discharge as in base run as well as for Jamuna discharge corresponding to its low water level of 6.08mPWD and 5.80mPWD at the New Dhaleshwari off-take. Test conditions of the subsequent tests have been decided based on understanding gained from the prior test results. It has done through interactive communication between RRI and BWDB i.e. taking feedback from the BWDB engineers concerned

Test T1 is the first application test. In this test, the model bathymetry is formed based on the equilibrium bathymetry obtained from T0. This test has been done with dominant discharge of 21,172 m^3s^{-1} (for part width of the Jamuna) with respect to +10.9mPWD water level of the Jamuna river at off-take location.

Test T2 is the second application test. In this test, the same bathymetry as obtained from the calibration test has been used as initial bathymetry. Moreover, the guide bunds, intake canal and sedimentation basin (**Fig 5)** have been introduced in the model as per latest layout and design supplied by the BWDB. New Dhaleshwari river channel downstream of the interventions is kept as it is i.e. no dredging is considered there. The model is run for the same discharge (dominant) as in Test T1.

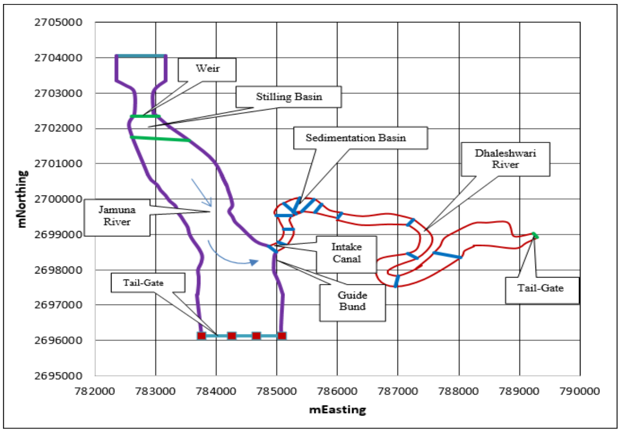

Fig 5. Model layout with proposed interventions at the New Dhaleshwari river off-take area

T3 is the third application test. In this test, the same bathymetry as obtained from the calibration test has been used as initial bathymetry. Moreover, the guide bunds, intake canal and sedimentation basin have been introduced in the model as per Test T2. However, in this test launching apron is placed all along the interventions where scour occurred in the test T2. New Dhaleshwari river channel downstream of the interventions is kept as it is i.e. no dredging is considered there. The model is run for the same discharge (dominant) as in Test T1 and Test T2.

Test T4 is the fourth application test. This test has been done keeping the water level at the New Dhaleshwari off-take at 5.8mPWD and 6.08mPWD. 5.80mPWD is the minimum recorded water level corresponding to the Jamuna flow of 2850 m^3s^{-1} whereas 6,08mPWD is the design water level. During this test potential for dry season flow through the New Dhaleshwari river for Jamuna river channel discharge of 2000 m^3s^{-1}, 3000 m^3s^{-1} and 4000 $m^3s^{-1}$and water level at the New Dhalesjwari off-take at 5.8mPWD and 6.08mpWD has been investigated. The dredged channel at the intervention locations has been considered and bathymetry of the New Dhaleshwari river in the downstream of the interventions is kept as it is in Test T3.

Test T5 is the fifth application test. In this test, in addition to the all arrangements as in Test T4, the bed level of the New Dhaleshwari river channel downstream of the structural interventions has been lowered and provided with the section as shown in **Fig 6** The considered dry season flows of the Jamuna river and corresponding water levels are the same as in Test T4.

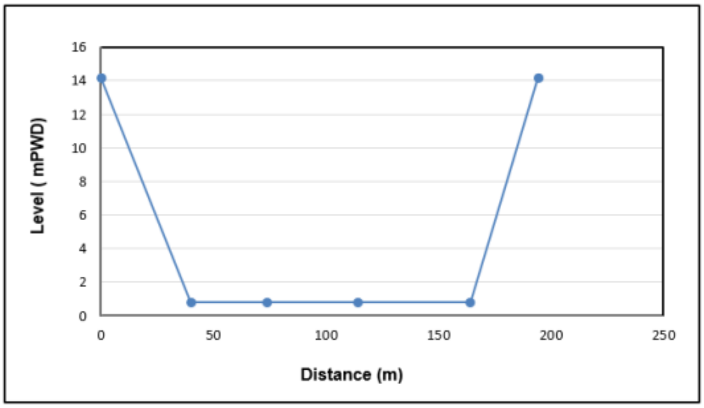

Fig 6. Typical channel section downstream of the intervention in Test T5

In Test T6, the same bathymetry as is in Test T5 has been used as initial bathymetry. Moreover, the guide bunds, intake canal and sedimentation basin have been introduced in the model as Test T2. However, no launching apron is placed along the interventions. The model is run with dominant discharge. The main difference between Test T6 and Test T2 is that the New Dhaleshwari river channel downstream of the interventions has not been dredged for the latter case.

In Test T7, the same bathymetry as is in Test T6 has been used as initial bathymetry. The structural interventions at the off-take are also the same as they are in Test T6. The only difference in test conditions between Test T6 and Test T7 is that launching apron has been provided along the interventions particularly at the scour prone locations as shown in **Fig 7**

Fig 7. Launching apron at the off-take mouth along the right bank

Results and Discussion

Discharge Distribution

Based on the measured discharges in the model it can be concluded that for dominant discharge of the Jamuna river the percentage of discharge through the New Dhaleshwari river is about 0.85% of the Jamuna river which is close to 0.8% as measured in the field. Whereas for T2 the percentage of discharge through the New Dhaleshwari river is about 1.71% (725 m^3s^{-1}) of the Jamuna river which is twice higher than that obtained in Test T1. It is to be noted here that this increase in the discharge through the New Dhaleshwari river may be attributed to the interventions at the off-take. For T3 the percentage of discharge through the New

Dhaleshwari river is about 1.73% of the Jamunariver which is remains almost the same (slightly higher) as is measured in Test T2.

For T4 and T5 the discharge through the New Dhaleshwari river for three probable dry season Jamuna river discharges and two corresponding probable water levels (5.8mPWD and 6.08mPWD) has been measured and the results are shown in **Table 3** and **Table 4** respectively. It is to be noted here that entry of dry season flow of the Jamuna river much depends on the existence of a channel near the off-take as the parent river (Jamuna) follows a braided pattern.

Table 3. Dry season discharge through the New Dhaleshwari river (without dredging in the downstream of the interventions)

$Q_{Jamuna}(m^3s^{-1})$	$Q_{Dhaleshwari}$ (m^3s^{-1})	
	WL = 5.80 mPWD	WL= 6.08 mPWD
2000	Insignificant	16
3000	25	31
4000	33	41

Table 4. Dry season discharge through the New Dhaleshwari river (with channel bed lowering in the downstream of the interventions)

$Q_{Jamuna}(m^3s^{-1})$	$Q_{Dhaleshwari}(m^3s^{-1})$	
	WL = 5.80mPWD	WL = 6.08mPWD
2000	348	410
3000	402	460
4000	455	605

It means together with the proposed interventions the river channel downstream of the same has also to be dredged to augment the dry season flow of the New Dhaleshwari river.

For T6 and T7, the percentage of discharge through the New Dhaleshwari river is about 3.62% (1520 m^3s^{-1}) and 3.60% (1510 m^3s^{-1}) of the corresponding Jamuna river discharge respectively. In both cases, the increase in discharge is due to lowering of the river bed in the downstream of the interventions.

Evolution of Bed Configuration

In equilibrium condition of T2, a sand bar is formed along the left bank of the New Dhaleshwari mouth (**Fig 8**) as in pre-dredge condition and the deep channel is along the right bank throughout the intake channel up to the sedimentation basin. In the sedimentation basin the dredged channel got silted up gradually with time. The same happened to the dredged channel downstream of the sedimentation basin. Initially more sediment is deposited in the intake canal rather than the sedimentation basin.

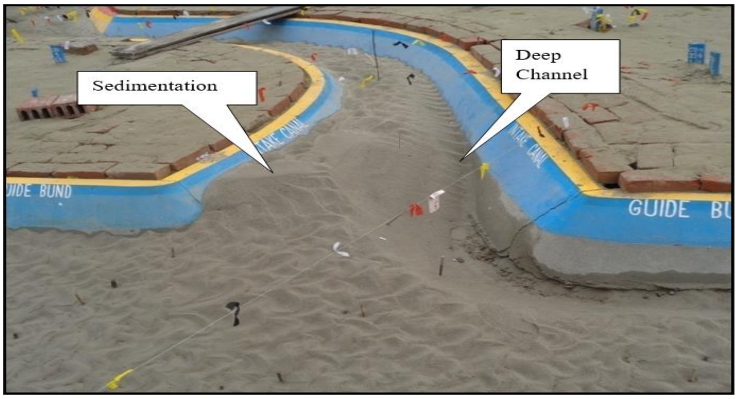

Fig 8. Final bed configuration at the off-take after sedimentation in the dredged channel in Test T2

Similar trend in deposition pattern is observed in Test T3. On the other hand, both in Test T2 and Test T3 somewhat overall lowering of bed level in the downstream of the intervention location is noticeable. Similar developments in bed level in the intake and exit canals have been noticed during Test T6 and Test T7 except the fact that the equilibrium bed level within the intervention location is somewhat lower in later two tests compared to that in former two tests. The morphological developments within the sedimentation basin with time are, however, relatively intricate for later two tests. Sedimentation pattern within the sedimentation basin in Test T6 (**Fig 9**) is noticeably different from that found in Test T6.

Fig 9. Extent of initially formed local scour at the entrance

In test T7, initially, a channel is formed along the right side of the sedimentation basin without endangering the launching apron provided at the sedimentation basin. After passage of time this channel is shifted towards the left side and has started to get filled up with sediment. A large sand bar is formed along the mid part of the sedimentation basin forming a distinct channel along the right side causing partial launching of the provided apron after dynamic equilibrium condition is reached. Scour tendency is observed at the outlet of the sedimentation basin along the left side where launching apron has been provided. The general morphological developments within the sedimentation basin are sedimentation along the left side and consequent formation of a large sand bar. The 2D plot of the initial and final bed level in the model appears in **Fig 10**. It shows sedimentation both within and beyond interventions.

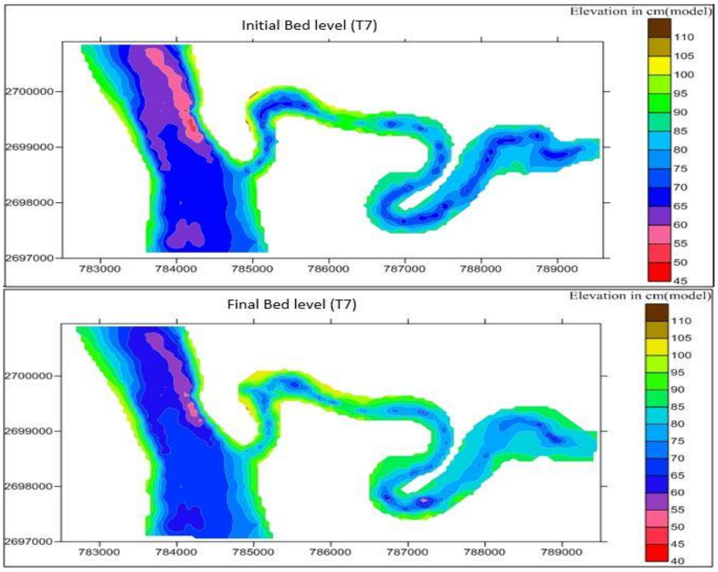

Fig.10. Comparison of bed level in Test T7

It is found from the Test T2 results that for reaching a dynamic equilibrium condition, a total volume of 2228941m^3 of sediment (prototype) is deposited within the intervention location. An analysis shows that 33% of this sediment volume will be deposited within the intake canal, 59% within the sediment basin and 8% within the exit canal. The average annual volume of sediment deposition within the intervention location is 557235m^3. The investigation also shows that due to increased flood discharge in the New Dhaleshwari river, bank erosion potential of the same will be increased particularly at bend locations.

Conclusion and Recommendations

There exists, sedimentation problem at the New Dhaleshwari off-take. The mean bed level as well as the minimum bed level at the mouth of the river is much higher than the dry season water level of the parent river (Jamuna). As a result, no flow situation occurs during dry season in the New Dhaleshwari River. In order to restore the polluted Buriganga River, a flow of 141 m^{3}s^{-1} has to be added to bring up the dissolved oxygen level to a tolerable limit. It could be done by augmenting 245m^{3}s^{-1} of flow from the Jamuna river through new Dhaleshwari River.

In order to augment the targeted flow from the Jamuna river a sustainable solution of the sedimentation problem at the off-take and in the river channel downstream of the same is essential. The proposed interventions at the off-take in the form of guide bunds, intake channel, sedimentation basin and exit channel could be a solution of the existing problem if properly planned and implemented with provision for long-term monitoring and maintenance dredging. Model results suggest that targeted flow augmentation of the New Dhaleshwari river is possible with the proposed interventions at the off-take and dredging as per design. However, river channel downstream of the interventions has also to be dredged to a level of 0mPWD with sufficient width for smooth passage of dry season flow. The extent of such dredging should be

determined based on monitoring survey field data analysis. A technically sound dredging strategy and phase wise implementation plan may be devised.
Implementation of the proposed structural interventions with dredging only within the interventions as per design will increase the flood discharge through the New Dhaleshwari River. Discharge of the New Dhaleshwari River corresponding to the dominant discharge of the Jamuna river will be more than two times higher compared to that in base condition. However, this increased discharge is not sufficient enough to lower the bed level of the downstream channel to a level that allows for the smooth conveyance of targeted dry season flow. The dredged channel will tend to get filled up gradually. Several sand waves will move from upstream to downstream during the filling up process. It may take about 4 to 5 years for the river to reach its dynamic equilibrium state by filling the dredged channel if no maintenance dredging is carried out.

There is potential for forming deep scour hole at the off-take mouth on the right side, at the starting point of the sedimentation basin on the right side and at the end point of the sedimentation basin on the left side. These locations should be protected well against local scour and developments there should be monitored closely. Due to increased flood discharge and consequent increased flow velocity bank erosion potential may increase in the entire river system particularly at the bend locations. In order to cope with this situation outer bank of the eroding bends may be stabilized by undertaking appropriate bank protection measures.

Acknowledgment

The authors like to express their profound gratitude to the Project Director of BRRP, BWDB for financial support and cooperation by providing all necessary data and information available with them to conduct the investigation. Sincere gratitude is also due to RRI authority and officials and staffs concerned for their encouragement and all out support to complete the model study successfully.

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NUMERICAL SIMULATION OF FLOODING IN HAOR AREA TO SUPPORT HYDROLOGIC AND HYDRAULIC DESIGN OF ROAD AND ROAD STRUCTURES: A CASE STUDY

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Abstract

River Research Institute has taken up a numerical model study aiming at establishment of the proposed Dirai-Sullah road link and assessment of the devised options to select suitable road alignment and also to determine appropriate type, location and dimension of the road structures in the low laying haor area. At present there is partially completed road link between Dirai and Sullah upazilla headquarters. Establishment of this road link is hampered by the fact that the project area is situated in complex physical and hydrological settings. The study area is hydrologically influenced by the combination of old Surma River and the Champti-Darain River together with many other drainage routes. Rainfall in the adjacent Indian state of Meghalaya largely affects flooding in the study area and the Surma-Kushiyara basin receives water from the transboundary catchments of the Meghalaya, the Barak and the Tripura. A comprehensive hydro-morphological study has been conducted for assessing the existing Madanpur-Dirai-Sullah road (Dirai-Sullah portion) and road structures to determine the appropriateness and adequacy of the existing road alignment and road structures in terms of their sustainability under critical and design hydrological scenarios to suggest appropriate road alignment and type, location, dimension and hydraulic design variables of the proposed road structures, to assess the need for slope protection works and devise their hydraulic design variables. A field survey campaign has been conducted to collect the recent bathymetric and bank line data of the rivers, nearby road alignment and road elevation data, information on existing road structures and physical features in the study area, water level and sediment data etc. as well as DEM of the study area. The study is based on extensive primary and secondary data. A two-dimensional model covering an extent of about 23km of the Champti-Darain River and parts of the Old Surma and other rivers in the study area together with parts of their floodplains has been developed using modelling software MIKE21C. The initial bathymetry of the model is formed by use of the recently surveyed bathymetric data of rivers and topographic data as well as DEM of the study area. The probable discharges and water levels have been identified based on the hydrological data analysis. The model boundary conditions for different returned period discharges have been determined by flood frequency analysis. At some upstream boundaries where measured data were not available, discharge was calculated by slope area method for different return period of flood (20 year and 50 year). The study results shows that the existing Derai-Sullah road alignment except some portion of road and bridge approaches is found to be suitable under likely hydrological and hydraulic conditions. Some portions of the road, the top level of the road is below the design formation level (8.5mPWD). In order to ensure smooth passage of an extreme field discharge one new bridge (103 m) and five new culverts has been suggested with their appropriate locations and dimensions.

Key Words: *Haor, Drainage, Scour, Wave runup, Flooding*

Introduction

Sunamganj district is located in the north-east region of Bangladesh. Dirai and Sullah are two upazilas under Sunamganj district. These upazilas are naturally resourceful with rice and fish cultivation. At present, there is no smooth road communication between these upazilas as the road link between the Dirai and Sullah upazila headquarters is not yet suitable for vehicular movement. While the Dirai upazila headquarter is connected with national road network by RHD zilla road most of the people of these two upazilas can not avail this opportunity easily. The study area is shown in **Fig.-1**. The Dirai and Sullah upazila are situated in the low-lying haor area where in the monsoon season haor starts to get filled by floodwater and by the month of July most of the area in these two upazilas get deeply flooded. As a result, people of this region have to solely depend on waterway communication to move from one place to another. Therefore, in order to connect these isolated upazilas to national road network it is essential to construct and improve the existing Dirai-Sullah road.

If this road is constructed and improved to the status of RHD zilla road,

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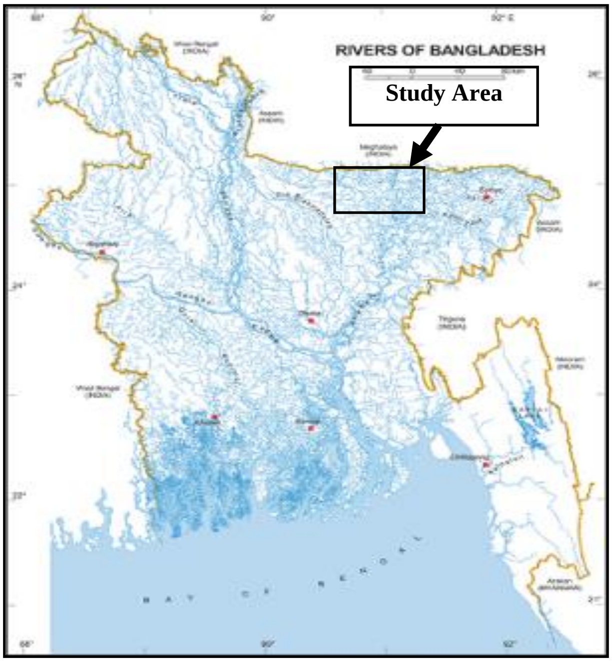

Fig. 1. Location of study area

it will be easier to transport agricultural products from these upazilas to other parts of the country, and people of this region will get transport facilities throughout the year (RHD, Request proposal, 2017). It will connect Sullah upazilla headquarter to the district town and rest of the country. Schools, madrasas and small cottage industries will reap benefit from this roadway communication. As a result, socio-economic condition of the people will improve.

The main purpose of this project is to establish a direct all weather roadway connection between Sullah upazila headquarter and district headquarter Sunamganj as well as rest of the country. The proposed road embankment through the haor will create obstruction to natural flow and will be subjected to wave action on it (BUET, 2008). Bridges often constrict the flow area and bridge piers also enhance this constriction resulting in increase in the velocity, acceleration of scouring process, backwater effect etc.

In this study, MIKE21C 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 hydrodynamics and sediment transport (DHI 2006).

The main objective study is to determine the parameters for s sustainable road alignment along with structures (bridges and culverts) and to provide the hydraulic design variables including the river training and protective work from hydrological and morphological considerations.

Methodology

In order to conduct the study, needed primary and secondary data and other relevant information have been collected. The primary data have been collected by conducting field survey which includes road alignment, road levels, road cross-sections, location and dimension of existing road embankment, slope protection works, cross-sections of rivers, khals and floodplains, soil and sediment samples etc. The secondary data include historical discharge and water level, digital elevation model (DEM) and satellite images of the study area. The collected data have been processed to derive inputs for the developed of two-dimensional model. The initial bathymetry of the model has been prepared based on field survey data together with DEM of the study area. The road structure data (alignment and elevation, opening and other physical features in the study area) have been used to incorporate the existing road and associated road structures in the model.

A two-dimensional model covering an extent of about 23km stretch of the road from Derai upzilla to Sullah Upazilla which also covered the Champti-Darain River and parts of the Old Surma and other rivers in the study area together with parts of their floodplains has been developed. A curvilinear computational grid of the model has a dimension of 238×5049. It means the length and width of the study reach are represented in the model with 238 and 5049 grid points respectively. The grid is generated covering an extent of floodplain on the both sides of the road. The collected data have been processed for model use format. The initial bathymetry has been prepared using recently surveyed bathymetric (2018) data of the rivers and topographic data. The computational grid and initial bathymetry of the model is shown in **Fig. 2** and **Fig. 3** respectively.

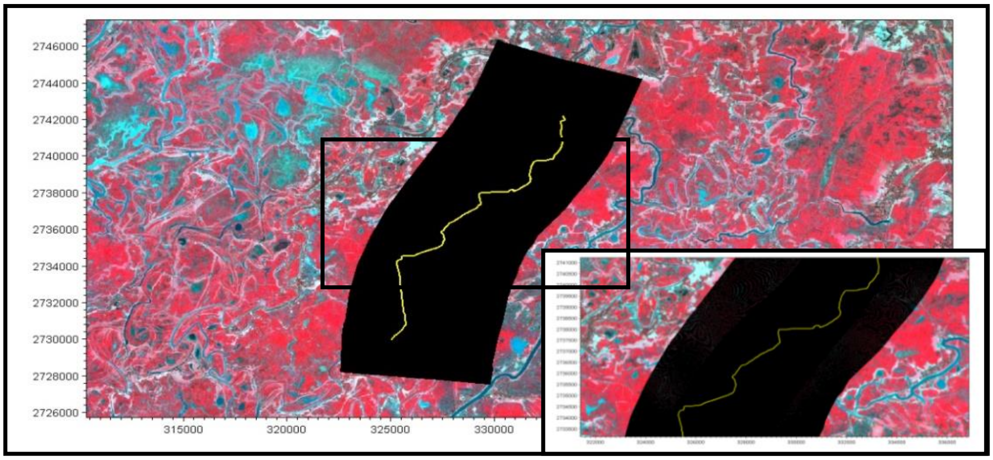

Fig. 2. Computational grid of the model

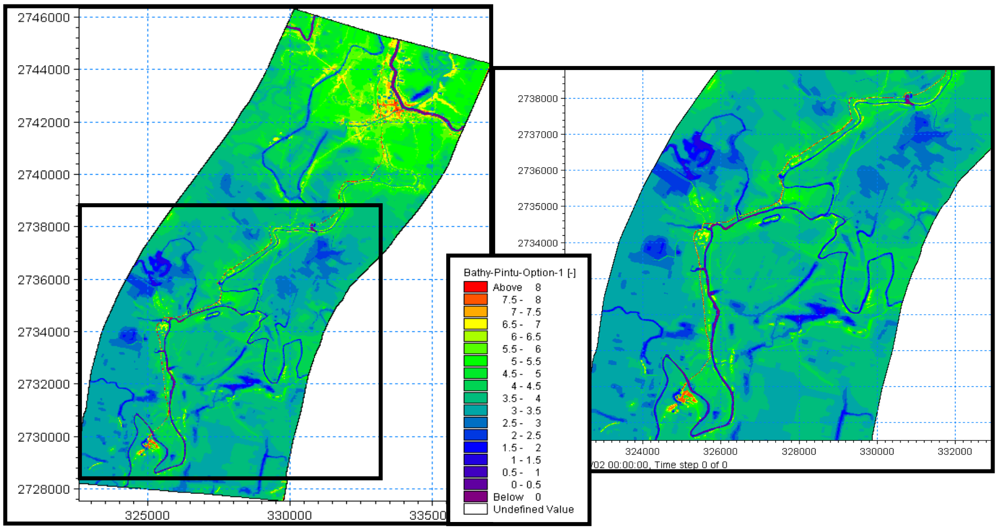

Fig. 3. Initial bathymetry of the model (MIKE 21C)

There is no discharge and water level gauge station within the model area. The flow enters into the model domain through two boundaries and leaves model domain through one boundary. The flow of the Old Surma River enters into the model domain through north and east boundaries. Water leaves the model domain through downstream boundary which covers a long stretch of floodplain. Flow in the study area occurs from northeast to southwest direction. Land slope in the study area also follows the same direction. Since haors act as storage reservoir during monsoon, majority of the flow leaves the model domain through downstream boundary. Downstream boundary data of the model are obtainable from the recorded historical water level data at Markuli and Ajmiriganj that are not much away from the downstream boundary. On the other hand, upstream boundary data of the model are obtainable from the recorded historical discharge data at Sunamganj. At the Old Surma off-take, the discharge of the Surma River gets divided between the Old Surma River and present course of the Surma River. Majority of the Surma river flow occurs through the new course. There is no other discharge gauge station between Sunamganj town and model upstream boundary. Therefore, the discharge records at Sunamganj town have been used to determine the model upstream boundary.The discharge distribution at the bifurcation point has been determined based on measured cross-sections of the two rivers nearby. Steady state simulations have been made with two return period discharges and water levels. 50 year flood is considered as critical hydrological condition for the road project. The probable water levels have been determined from historical record of annual maximum water levels by frequency analysis using GEV and EVI distribution. On the other hand, the discharges corresponding to design and other water levels have been determined using slope area method. This is done because no discharge data of the small rivers that cross the road is available.

Since the road is already in place in incomplete manner and there is not any scope for selection of a better alternative as the existing road passes through the relatively high elevation land along the Champti-Darain river where there are scattered human settlements. However, there is scope for small modifications in existing road alignment. There are two large road gaps at Nayagaon and Kashipur and local people want these gaps to be closed permanently. At present BWDB constructs submersible closures across these gaps every year to prevent the early flood water from entering the crop land. In view of this fact, three options have been devised for hydrological and hydraulic assessment using the developed model. It is to be noted here that all the three options are almost similar except a little variation in road alignment and some variation in the total road opening.

The model investigation is made 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.

In Option-1, the alignment of the road and road structures are kept as they are except a bit modification is made to keep the horizontal curve smooth. However, the road gaps at Nayagaon (226m) and Kashipur (161m) are kept closed according to the public demand. The total length of the road (including road structures) under this option is about 18.4km. The road alignment under this option is shown in **Fig. 4**.

The road alignment in **Option-2** follows the same as in Option-1 except a little shift of the road position towards the west at about 614m (along the road) north from the bridge at Sullah upazila headquarter and to the south of the road gap at Kashipur considering straight connection between two ends of the existing road at two road gap locations. This shift in road position at these two locations has been considered for keeping the road away from the river bank as at present there is little or no setback distance between the road and river bank margin. The road gaps at Nayagaon and Kashipur have been kept open allowing the oncoming flow to pass through these gaps. The total length of the road (including road structures) under this option is about 17.7km. The road alignment under this option is shown in **Fig. 4**.

The road alignment under Option-3 follows the same way as in Option-2 but additional 2 (two) road bridges in two road gaps and 5 (five) culverts have been introduced. It is done to avoid blockage of natural connectivity between the river and the floodplain at these locations. The total length of the road (including road structures) under this option is about 17.834km.The road alignment under this option is shown in **Fig. 4**.

Results and Discussions

The hydrologic and hydraulic analysis of road and associated road structures have been conducted by the use of developed two-dimensional numerical model. With the road and associated road structures under three different options in place, the hydraulic performance of the devised options have been assessed with two different return period discharges (20 year and 50 year).

The model investigation is made to assess the hydrodynamic response of the structures in terms of discharge through the structures, through structure velocity, water level and flow depth at the structures and along the road, afflux caused by the road 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 field at and around the road and associated road structures for different options have been furnished in **Fig. 4**. Due to presence of the road the flow directions have undergone substantial local changes. For all the options, the flow velocity on the floodplain is very low in magnitude (0.0m/s to 0.3m/s) for 50 year discharge. However, through structure velocity is relatively high ranging from 0.28m/s to 0.71m/s (RRI 2018) for the same 50 year discharge. The discharge through the different existing road structures under different options appears in **Table 1**

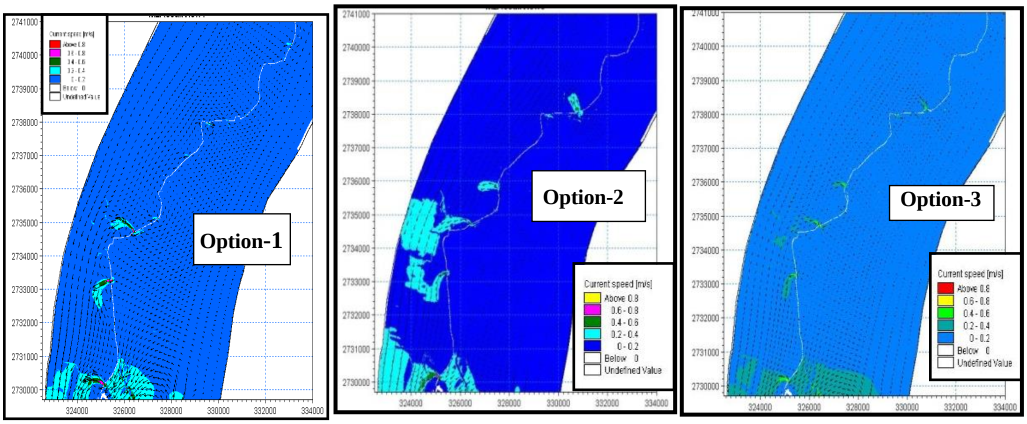

Fig. 4. Velocity field in different Options at and around structures of Derai-Sullah road for 50 year discharge

Since the road structures should be designed for 50-year discharge, the magnitude of flow through each structure corresponding to this discharge and also average and maximum through structure velocity under each option have been extracted from model simulation results. It is found that under Option-1 magnitude of flow as well as velocity through the structure is relatively high for most of the road structures compared to that under Option- 2 and Option -3. It happens due to complete closure of two existing large road gaps at Nayagaon and Kashipur. On the other hand, under Option-2 both discharge and flow velocity through the existing road structures reduce substantially for keeping the same road gaps open. It is revealed from the model simulation results for Option-2 that large flow occurs through these road gaps. Therefore, it will be wise to bridge these gaps against the public demand as it will help maintain natural flow conditions to some extent. Since these gaps are very large it will also not be feasible to construct very long bridge there. Therefore, two bridges of 103m and 76m clear opening and also five culverts in addition to existing ones have been considered under Option-3. It gives fairly reasonable results in terms of flow and velocity through the structures. Total flow through the considered road structures is also reasonable compared to Option-2 condition.

Table 1. Velocity and flow through different road structures under different options for 50 year discharge

SL. No.	Structure description	Option-1 Velocity (m/s) Avg/Max	Option-1 Discharge (m3/s)	Option-2 Velocity (m/s) Avg/Max	Option-2 Discharge (m3/s)	Option-3 Velocity (m/s) Avg/Max	Option-3 Discharge (m3/s)
1	Rajapur culvert (3 vent)	0.50/0.55	42	0.31/0.33	26	0.3/0.33	25
2	Rajapur culvert (2 vent)	0.52/0.56	29	0.32/0.33	18	0.32/0.33	18
3	Islampur culvert (1 vent)	0.48/0.48	13	0.28/0.28	8	0.3/0.30	8
4	Nayagaon culvert (1vent)	0.44/0.44	12	0.26/0.26	7	0.28/0.28	8
5	Nayagaon culvert (1vent)	0.44/0.44	13	0.24/0.24	7	0.33/0.33	10
6	Nayagaon culvert(2 vent)	0.45/0.48	25	0.26/0.28	14	0.36/0.39	20
7	Nayagaon bridge (103m)	-	-	-	-	0.52/0.57	188
8	Dhanpur culvert (3 vent)	0.40/0.42	37	0.32/0.33	30	0.4/0.43	37
9	Dhanpur culvert (1 vent)	0.30/0.30	8	0.15/0.15	4	0.31/0.31	8
10	Chandipur culvert(1 vent)	0.35/0.35	10	0.18/0.18	5	0.32/0.32	9
11	Chandipur culvert (2 vent)	-	-	-	-	0.53/0.56	31
12	Kashipur culvert (3 vent)	0.44/0.48	35	0.28/0.29	22	0.46/0.50	36
13	Kashipur culvert (1 vent)	0.38/0.38	10	0.22/0.22	6	0.38/0.38	10
14	Kashipur bridge (76m)	-	-	-	-	0.43/0.46	169
15	Darain culvert (3 vent)	0.49/0.56	43	0.35/0.38	31	0.51/0.56	45
16	Bholanagar bridge (94.16m)	0.60/0.64	244	0.42/0.46	170	0.62/0.70	251
17	Giridhar Culvert(3 vent)	0.54/0.54	15	0.37/0.37	10	0.55/0.59	53
18	Giridhar culvert (1 vent)	0.52/0.52	15	0.29/0.29	8	0.48/0.48	14
19	Giridhar culvert (1 vent)	-	-	-	-	0.43/0.43	12
20	Anandapur culvert (2vent)	-	-	-	-	0.51/0.55	27
21	Anandapur bridge(64m)	0.73/0.84	319	0.55/0.61	240	0.62/0.71	270
22	Sukline culvert (1 vent)	0.58/0.58	17	0.39/0.39	11	0.46/0.46	13
23	Angaruabari culvert (1vent)	0.54/0.54	15	0.38/0.38	11	0.44.0.44	12
24	Angaruabari culvert(2vent)	-	-	-	-	0.5/0.53	28
25	Sullah culvert (3 vent)	-	-	-	-	0.41/0.45	34
26	Sullah bridge(94.16m)	0.65/0.74	413	0.43/0.45	273	0.48/0.53	305

Water Levels along the Roads

The two-dimensional plots of the water levels at and around the road and associated road structures in the study area for different options have been furnished in **Fig. 5**. It is evident from the figures that the afflux caused by the road and road structures is not much high for the considered extreme hydrological condition. Afflux caused by the road varies from 2.0 cm to 4.0 cm. It is clear that water surface slope in the study area is very mild. The water level along the road from Dirai end to Sullah end varies from 7.74 cm to 7.70 cm for 50 year discharge whereas it varies from 7.44 cm to 7.40 cm for 20 year discharge.

Water Depth

The Dirai-Sullah road runs across the low-lying haors and beels that are unique in their hydro-ecological characteristics. In the monsoon season this low-lying haor area starts to get filled by floodwater coming from the trans boundary catchments through both
Surma and Champti-Darain systems. Flood (water) depths in the study area and at and around the road for different Option are revealed in **Fig. 6.**

Since the road and road structures are already in place the hydraulic analysis is made incorporating them in the model according to their placement, dimension and orientation as accurately as possible. The Dirai-Sullah road is situated in a low lying area that goes under water during flood season. Therefore, there is possibility that the road embankment could be subjected to wave run-up. At some locations the existing road runs more or less parallel to the rivers that flow through the haor area and the distance between the road and the river bank margin is very less.

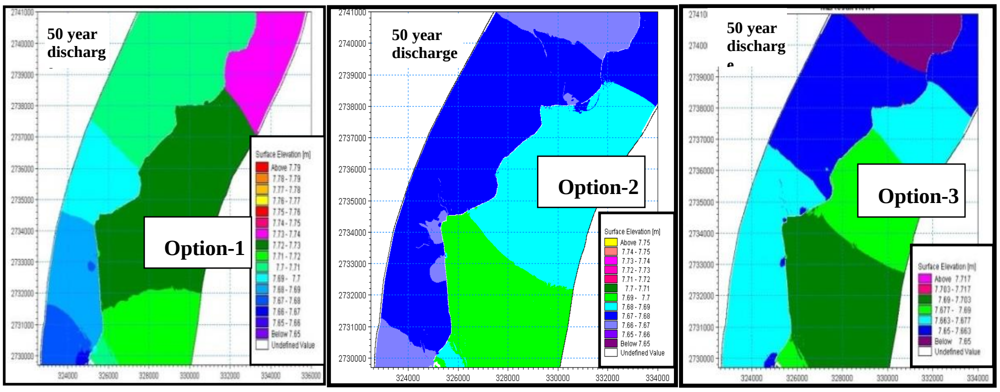

Fig. 5. Simulated two-dimensional plots of water level in the study area for 50 year discharge condition at and around structures of Derai-Sullah road

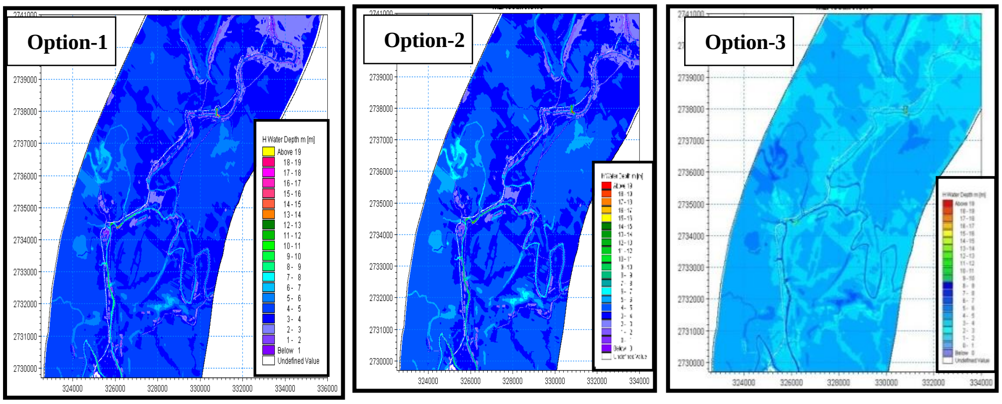

Fig. 5. Simulated two-dimensional plots of water Depth in the study area for 50 year discharge at and around structures of Derai-Sullah Road

Therefore, Option-2 and Option-3 have been devised by changing the existing road alignment locally at some locations where there is no or very less setback distance between the road and the river bank margin.

It is evident that due to blockage of two major flow routes under option-1, most of the flow finds its way southward. As a result, flow concentration occurs at and around the Sullah Upazila Headquarter causing an increase in the flow velocity there. It shows that about 1314m^3/s of flow passes through the 19 (nineteen) existing structures of which 732 m^3/s passes under the two bridges at Sullah and Anandapur. On the other hand, a total of about 1081 m^3/s of discharge passes through the 8 (eight) road structures (3 bridges and 5 culverts) in the downstream (southwest direction) of the blockage at Kashipur, which is about 82% of the total flow that passes through all the existing structures on the Dirai-Sullah road. It means first 11 (eleven) road structures starting from the Dirai end convey only about 18% of the total flow. In Option-2, total flow through the existing structures is 899m^3/s compared to 1314m^3/s under Option-1. However, flow through the two road gaps at Nayagaon and Kashipur is found to be 450m^3/s and 244m^3/s respectively. Therefore, total cross-flow under this condition is 1593m3/s, which is higher than that under Option-1 condition. As in Option-1, majority of flow (about 76%) occurs through the three bridges at Bholanagar, Anandapur and Sullah. It indicates the gaps at Nayagaon and Kashipur should be bridged instead of keeping closed. In Option-3 condition, the total flow across the Dirai-Sullah road is 1641 m^3/s of which 104m^3/s passes through 10(ten) 6m culverts that already exist. The total flow through the five 12m and six 18m culverts is122m^3/s and 223m^3/s respectively. On the other hand, the total discharge through the five bridges including the two newly suggested ones is 1183m^3/s. It means under Option-3, over 72% of the total flow passes through the bridges. It is marked that due to introduction of 7 (seven) newly proposed structures, the average flow velocity through majority of the existing structures decreases compared to Option-1condition. Noticeable decrease occurs in the flow through Anandapur bridge and Sullah bridge. However, a slight increase in the flow through the Bholanagar bridge is noticeable. It appears from the model results that Option-3 is the best among three considered options as it will allow for safe passage of an extreme flood discharge with relatively low flow velocity through the structures. The flood flow through the newly suggested bridges at Nayagaon and Kashipur is found to be 188m^3/s and 169m^3/s respectively. Also under Option-3 condition total cross-flow is higher than that under Option-1 and Option-2 conditions. level along the road varies from Also under Option-3 condition total cross-flow is higher than that under Option-1 and Option-2 conditions. Water 7.65cm to 7.70cm. The average water level is 7.675mPWD.

Slope Protection Works

The road runs through a low lying area. During extreme flood this low area experiences average flood depth of more than 3.0 m with low velocity. Along the existing road 26 road structures (culverts and bridges) have been suggested to allow for safe passage of flood water. The structures have been constructed mainly over the drainage routes that cross the road. No hydro-morphological study has been conducted to decide about hydrologic and hydraulic design parameters of most of these structures. Some of the road structure approaches have got damaged due to hydraulic actions and other reasons. Due to blockage of normal floodplain flow by constructing road, flow occurs parallel to the road embankment that causes damage to road embankment and approaches of the road structures. Therefore, slope of the approaches of road structures is vulnerable to damage by parallel current and thereby, needs protection against such hydraulic actions.

It is revealed from the study that the approach embankment slope protection works will be needed at all bridge approaches and also at 12 stretches of the proposed road. Some of the identified road stretches where slope protection works will be needed have been shown in **Fig. 6**. The hydraulic design data for the existing and newly suggested bridges and slope protection works have been furnished in **Table 2** and **Table 3** respectively.

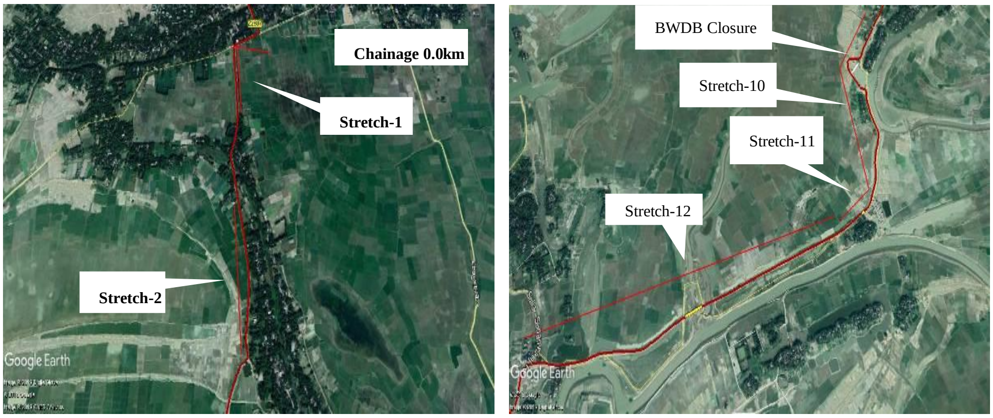

Fig. 6. Some location of vulnerable road Slope Protection Works

Table 2. The hydraulic design parameters of suggested and existing bridges

Road Structure description	Chainage (km)	Length (m)	Design discharge (m^3/s)	Design water level (mPWD)	Maximum velocity (m/s)	Pier Scour level (mPWD)	Abutment scour level (mPWD)
Nayagaon bridge (suggested)	5.40	103.0	188	7.68	0.57	-6.19	-7.6
Kashipur bridge (suggested)	10.02	76.0	169	7.68	0.46	-5.95	-7.43
Bholanagar bridge (existing)	11.94	94.16	251	7.68	0.70	-	-
Anandapur bridge (existing)	14.46	64.0	270	7.68	0.71	-	-
Sullah bridge (existing)	17.63	94.16	305	7.68	0.53	-	-

Table 3. Design parameters for slope protection works

Design flood level	: 7.68mPWD
Velocity	: 0.8 m/s
d_m of silt	: 0.06mm
Depth of flow	: 3m to 5m
Wind speed	: 30 m/s
Fetch length	: 7 km
Wind duration	: 2 hours
Wave height	: 1.4m
Wave period	: 3.6 seconds
Wave runup	: 1.12 m

Navigational Clearances

The rivers and drainage routes that the proposed road crosses do not fall under BIWTA navigational route classification. From June to November the study area remains under water and waterway communication becomes the only means for the people to go from one place to another and to transport goods. Generally people use motor driven boats for this purpose. Based on the information obtained from the local people regarding navigation condition a vertical clearance of 1.5m has been proposed for two newly suggested bridges at Nayagaon (Dhanpur) and Kashipur. The suggested length of these two bridges is 103m and 76m respectively.

Conclusion

Establishment of smooth road communication between Dirai and Sullah upazila headquarters for vehicular movement is complicated because of a number of physical and hydrological factors. The partially constructed road and slope protection works have already been subjected to damage to some extent at different locations due to hydraulic actions and other reasons. The existing road alignment lies on relatively high elevation area along the right side of the Champti-Darain river. This road alignment is suitable with some modifications particularly at places where the setback distance between the road and bank margin is very less. There are twenty existing road structures (bridges and culverts). However, approaches of most of these structures are either not yet constructed or have gotten damaged fully or partially. There are instances of road damage in the form of settlement of road pavement and erosion or washout of approach embankment materials. The road embankment slope failure at unprotected locations and full or partial damage of slope protection works at many protected locations are noticeable. The identified causes of approach and road damage are poor quality of locally available fill material, inadequate compaction, long standing high depth floodwater on both sides of the road, parallel current and wave action.

The flood level corresponding to 50 year discharge varies from 7.65mPWD to 7.70mPWD along the road. The water level slope is very mild. The average flood level is 7.675mPWD. On the other hand, the average flood level corresponding to 20 year discharge is 7.375mPWD. At present the existing top level of a number of road stretches is below the formation level of the road. The formation level of the road is 8.5mPWD. The afflux caused by the road and road structures is not high and varies from 1.0 cm to 4.0 cm for the considered hydrological condition. During flood season very low velocity is observed on the floodplain whereas relatively high velocity is observed only along the main river courses. Two new bridges and five new culverts should be introduced on the road at suggested locations in addition to the existing ones for smooth passage of a extreme flood discharge. These structures should be constructed as per suggested dimensions. Since there is strong public opposition for construction of a bridge as Kashipur it may be omitted and instead the clear opening of the proposed bridge at Nayagaon may be increased by 30m to compensate for this to some extent.

At some stretches of the road, there is no or very less setback distance between the road and the river right bank margin. It will be wise to retire the existing road embankment at those locations to avoid potential risk of road damage due to hydraulic actions. The road gaps at Nayagaon and Kashipur are important for maintaining river and floodplain connectivity. The road embankment may come under wave action at some locations and estimated wave runup is 1.12m. The model simulated flow velocity through the different existing and proposed road structures varies from 0.28 m/s to 0.71 m/s for recommended measures.

Road embankment slope protection works will be needed against the occurrence of parallel flow along the road embankment and wave action. Long standing high water depth on both sides of the road should also be considered as a factor of potential road damage. Approach road slope protection measures should be undertaken at suggested locations and road stretches vulnerable to parallel current, long standing high water depth and wave action.

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MANGANESE REMOVAL FROM DRINKING WATER USING ROUGHING FILTRATION

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Abstract

Manganese can be removed using the processes of oxidation, precipitation and filtration as in iron removal. Conventionally, a strong oxidant such as chlorine or potassium permanganate is used for oxidation of Mn(II) rather than oxygen alone.Naturally present DO and other constituents (e.g., bicarbonate) in water can promote removal of Mn in a filter media (by sorption and oxidation) without the addition of an oxidant. Two multistage filtration units (MSFU) have been constructed in Sirajgonj to investigate the effectiveness in removing manganese from groundwater adopting the technique of adsorption and co-precipitation of manganese onto the flocs of ferric hydroxide, making use of the naturally occurring iron of groundwater. Observed results indicate that manganese removal is a function of raw water manganese concentration. Higher the Mn concentration, greater is the removal performance. 80-89% manganese removal performance was achieved through this filtration process. Contribution of Down-flow roughing filter (DRF) alone in removing manganese was observed very significant (around 37%). Contribution of aeration, flocculation and sedimentation in removing manganese is around 33%. Contribution of Up-flow roughing filter (URF) in removing manganese is moderate (around15%).

Keywords: *Concentration, DRF, effect, filtration, manganese, performance, removal, URF*

Introduction

Manganese is one of the most abundant metals in Earth's crust, usually occurring with iron. It is a component of over 100 minerals but is not found naturally in its pure (elemental) form (ATSDR, 2000). Manganese is an element essential to the proper functioning of both humans and animals, as it is required for the functioning of many cellular enzymes (e.g. manganese superoxide dismutase, pyruvate carboxylase) and can serve to activate many others (e.g. kinases, decarboxylases, transferases, hydrolases) (IPCS, 2002). Manganese can exist in 11 oxidative states; the most environmentally and biologically important manganese compounds are those that contain Mn2+, Mn4+ or Mn7+ (USEPA, 1994).

At excessive concentrations, manganese can be detrimental to health. Evidence from occupational exposure indicates that manganese can affect neurological function. Miners and welders exposed through airborne contamination for long periods have eveloped neurological disorders such as Parkinson's disease (Takeda, 2003).Some links have been made between exposure to manganese and a form of motor neuron disease found in the Pacific region, known as Guamian amyotrophic lateral sclerosis (Foster, 1992). Iwami et al. (1994) found correlations between the concentrations of manganese in food and the prevalence of motor neuron disease in the Kii Peninsula of Japan. Cawte et al. (1987) also reported neurological symptoms in manganese ore miners from Australia. Occupational exposure to manganese has also been linked with liver, kidney and lung damage.

Groundwater is abundant in Bangladesh and the aquifers are highly productive. 90% of Bangladeshi depends on ground water for drinking purpose because much of surface water of Bangladesh is microbially unsafe to drink (Ahsan and Del Valls,2011). Unfortunately, the vast area of Bangladesh's groundwater is naturally contaminated with arsenic, iron and manganese concentrations above the World Health Organization (WHO) drinking water guideline and even the Bangladesh drinking water guideline (BGS and DPHE 2001; Smedley 2003; Anawar et al., 2003).Presence of excessive manganese in potable water may cause significant adverse health impacts. It may also causes problems related to aesthetics and may cause precipitation in the water distribution system. The World Health Organization (WHO) has a provisional health based guideline value of 0.4 mg/l for manganese in drinking water (WHO, 2004) forprotect against neurological damage. The WHO guideline value from consumer acceptability consideration is 0.10 mg/l (WHO, 993). Bangladesh Standard for manganese in drinking water is also 0.10 mg/1. At levels exceeding O.1 mg/, manganese in water supplies stains sanitary ware and laundry and causes undesirable taste in beverages. The presence of manganese in

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drinking water may lead to accumulation of deposits in the distribution system. Even at a concentration of 0.02 mg/I, manganese may form coating on distribution pipes, which may slough off as a black precipitate(Hoque,2006).

The national hydro-chemical survey (BGS and DPHE, 2001) have shown that in Bangladesh, large numbers of wells exceed permissible limits for iron (Fe) and manganese (Mn). This is true of shallow tubewells, and also to some extent for deep tubewells and ring-wells, which are common water supply options in arsenic-affected areas. The national hydro-chemical survey found that half of the wells surveyed exceeded the Bangladesh drinking water standard for iron (I mg/I) and three quarters exceeded the standard for Mn (0.1 mg/I). Both of these limits are based on aesthetic considerations; above these levels, people may be unwilling to drink the water, and turn instead to a better-tasting, but microbiologically less safe water sources. About 40% of wells were found to exceed the WHO health-based guide value (0.4 mg/I). Some of iron and manganese concentrations reported .in the national hydro-chemical survey (BGS and DPHE, 2001) are very high, over ten times the permissible limit. Iron and manganese concentration as high as 25 mg/l and 10 mg/l, respectively have been reported. Average iron concentration has been reported to be 3 mg/l (median 1 mg/l) and average manganese concentration 0.5 mg/l (median 0.3 mg/l) (BGS andWaterAid,2001).

Unlike the distribution of arsenic, which has a distinct regional pattern with highest contamination in the south, south-west, and north-eastern regions of Bangladesh, high concentrations of manganese are found in most areas, but relatively high concentrations are seen in the current Brahmaputra and Ganges floodplains. The distribution generally does not correspond to that of arsenic (BGS and WaterAid, 2001). This means that groundwater with acceptable concentration of arsenic may not have acceptable concentration of manganese (Hoque,2006).

Manganese can be removed using the same processes of oxidation, precipitation and filtration as in iron removal (Fair et al., 1968). Conventionally, a strong oxidant such as chlorine or potassium permanganate is used for oxidation of Mn(II) rather than oxygen alone (Hartmann, 2002) .However, some studies (e.g., ITN-BUET, 2011) reported that naturally present DO and other constituents (e.g., bicarbonate) in water could promote removal of Mn in a filter media (by sorption and oxidation) without the addition of an oxidant. This phenomenon needs to be investigated in more detail through laboratory experiments (Habib,2013). Mn(II) oxidation can lead to precipitation of Mn(III, IV) oxides which are in turn good adsorbents and oxidants (Hem, 1978) . A number of studies (Kanet a l., 2012; Buamah, 2009; Afsana, 2004) showed that Mn is removed effectively from groundwater by oxidation and adsorption processes. Media coated with synthetic Mn oxides have also been found to have good Mn removal efficiency (Merkle et al., 1997; NMSU, 1999; Dhiman and Chaudhuri, 2007; Maliyekkal et al., 2009). Manganese oxide coated filter media could therefore be potentially used for Mn removal from groundwater, although no report of its use in Bangladesh could be gathered (Habib,2013.Oxidation precipitation is by far the most widely used technique for manganese removal from water. A number of water quality parameters such as pH,eH ,iron, organic matter etc. can affect the efficiency of manganese removal from water (Seeling et al., 1992; Lemley et al., 1999). Manganese oxide coatings formed on filter media in filtration beds have been found to act as good adsorbent for Mn and also plays a role in its oxidation (Eley and Nicholson, 1993; Tasneem, 2010; ITN-BUET, 2011).

The iron problem has long been recognized in Bangladesh, and many technologies have been developed for iron removal at municipal, community and household levels. Municipal Iron Removal Plants (IRPs) were first installed in Bangladesh during the early 1980s. After the detection of arsenic in ground water, many municipal IRPs are now being designed and used for removal of both iron and arsenic. In the backdrop of the discovery of arsenic in many areas of the country, community treatment units designed for removal of both arsenic and iron are becoming popular. Many NGOs are now installing different types of such community-based iron/ arsenic removal plants. However, most of the plants have been constructed without following any technical design parameters (BRTC, 2006). So it is important to see whether Mn is removed significantly in the currently operational iron and/or Fe-As removal plants, which have been designed primarily for removal of iron and/or arsenic.

Objective of the study are (a) to investigate the effectiveness of roughing filtration in removing manganese from groundwater adopting the technique of adsorption and co-precipitation of manganese onto the flocs of ferric hydroxide, making use of the naturally occurring iron of groundwater, (b) Manganese Removal Performance of Different Treatment Unit Processes (c) Effect of initial manganese concentration on manganese removal performance.

Methodology

Selection of study area

Two study area Kodda and Chala in Sirajgonj district were selected on the basis of iron and manganese concentration in ground water. Department of Public Health Engineering (DPHE) were contacted and requested to extend their co-operation for this research work. The field sites were identified by direct co-operation of the Department of Public Health Engineering (DPHE) staffs. The water quality characteristics of the study area and plant location have been shown in Table 1.

Table 1. Water quality characteristics of the study area and plant location

Location of Filtration Units	pH	Alkalinity (mg/lSO as $CaCO_3$)	Iron (mg/l)	Manganese (mg/l)
Kodda, Sirajgonj	7.1	174	16	1.625
Chala, Sirajgonj	7.0	134	15	0.720

Selection of kinetics of manganese oxidation, precipitation & removal

Manganese is much more slowly oxidised through aeration than iron. In fact, the rate is negligible at pH levels below 9.0.Chemical oxidation of Mn requires a pH level above 8.5 and 1.0 mg of chlorine can oxidise 1.3 mg of Mn. Mn oxidation through chlorine requires 2-4 hours to react completely.Both hydrous $Fe(OH)_3$ & MnO_2, tend to sorb Fe++ & Mn++ ions.Removal of iron and manganese is generally hastened and made more efficient (swifter) by letting water trickle downward or rise upward through gravel or other relatively coarse heavy materials coated with hydrous oxides of Fe(III) and Mn(IV) precipitates by sorption.If Fe(II) > Mn(II) rather than Mn(II) alone, removal than becomes predominantly a matter of sorption of Mn++ on incipient ppt. of iron

Selection of the unit process for the MSFU

Multistage filtration units (MSFU) considered under the study comprised of three units :1st chamber (aerator plus down-flow flocculator), 2nd chamber (sedimentation plus up-flow roughing filter) and 3rd chamber (down-flow roughing filter).

Functions of individual unit

The functions of the individual units have been shown in the following table.

Table 2. The functions of the individual units of roughing filter

Unit	Functions
Aerator	Water entering the first chamber is distributed uniformly over the whole bed of course media through a porous thin ferro-cement plate placed on the top, resulting strip out of CO_2 and increase of pH value for the oxidation of soluble iron.
Down flow Flocculator	Oxidation and subsequent precipitation of iron oxy hydroxides occurs respectively on the top and within the interstices of coarse media which adsorbs manganese oxy anions. Sinusoidal flow across the coarse media enhance collisions for the flocculation of precipitated particles.
sedimentation chamber	Comparatively larger flocculated precipitates settle at the bottom of the 2^{nd} chamber
up-flow roughing filter	Maximum removal of precipitated particles occurs by sorption on to iron oxy hydroxides and mechanical straining during up-flow through the comparatively finer coarse media bed in the 2^{nd} chamber.
down-flow roughing filter	Final removal of precipitated particles occurs through sorption on iron flocs and other metal oxy-hydroxides during down-flow through the comparatively finer coarse media bed than the 2^{nd} chamber

Design Parameter of multistage filtration units

Following design parameters have been considered for multistage filtration units.

Table 3. Design parameters for multistage filtration units.

Unit	Parameter		
Down flow Flocculator	Face velocity =3.24-4.05 m/hr	Detention time =4.5 – 6 min	Flow =16 - 20 L/min
Sedimentation Chamber	Surface Over Flow Rate=9.7-12.9 m^3/m^2-day	Detention Time = 28 - 35 min	
Up-flow Roughing Filter	Face Velocity = 0.32 – 0.40 m/hr	Detention Time = 28 - 35 min	Flow = 12 – 15 L/min
down-flow roughing filter	Face Velocity = 0.22 – 0.27 m/hr	Detention Time = 66 - 82 min	Flow = 2 – 2.5 L/min

Sampling and analytical methods of testing

Water quality analysis of this study was conducted at the laboratory of Environmental Engineering Laboratory, Department of Civil Engineering, BUET, Dhaka.The pH and Iron contents of the water samples were determined in the field regularly. In this process iron concentrations were determined using HACH field kit and pH were determined by field pH meter. At each treatment plant location, raw and treated water samples were collected for subsequent analysis of iron, manganese and other selected water quality parameters in the laboratory. Samples were collected in pre-washed 500 ml plastic bottles and were acidified with 1 ml concentration Nitric acid, which were later used for analysis of dissolved manganese and iron. In the laboratory iron and manganese concentrations were determined using HACH spectrophotometer as well as Flame-AAS(Atomic Absorption Spectrophotometer).

Result and discussions

Performance analysis of the filtration units on the basis of collected field data and laboratory test results have been analyzed and presented in the following articles.

MSFU-1 (Kodda,Sirajgonj)

The variation of average manganese concentration with operation period in different treatment unit processes of the MSFU-1 have been explained in Fig. 1. The initial concentration of manganese in the effluent of URF and DRF were found to be 0.7 and 0.29 mg/L indicating removal efficiency of 57 % and 82% respectively. With the passage of time the manganese concentration in the effluent of URF and DRF decreased upto 0.58 mg/L and 0.11 mg/L indicating removal efficiency of 64 % and respectively 93%. This was because there were gradually adsorption of precipitated iron flocs on the coarse media surfaces and gradually deposition of the same in the interstices and these iron particles along with other metal oxy-hydroxides provided increased adsorption surfaces for the manganese ions to be adsorbed. However, after 3 to 4 weeks of run the manganese concentrations in the effluent of DRF again started to increase. This was because when the coarse media pores were clogged the increased pore velocities caused shearing / sloughing of precipitated iron particles which resulted less adsorption site available for arsenic ions and ultimately appeared with the effluent water.

From the beginning of the filter run a continuous increasing trend of arsenic concentration in the effluent of sedimentation chamber have been observed .Because with the passage of time gradually accumulated iron flocs at the bottom of sedimentation chamber along with adsorbed manganese ions were carried over and ultimately appeared in the effluent of this chamber.

This figure indicates that DRF process have significant effect on manganese removal. Manganese concentrations in the effluent of sedimentation chamber and URF chamber were much above the WHO health based guideline value for manganese (0.4 mg/L) to protect against neurological damage.

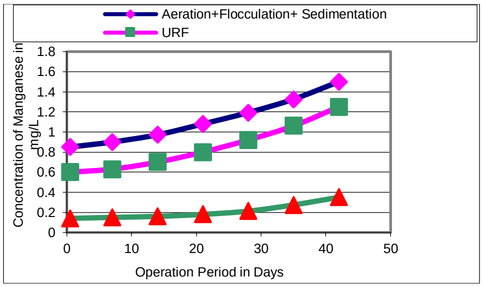

Fig. 1. Variation of average manganese concentration with operation period in different treatment unit process of Kodda

On the other hand,manganese concentration in the effluent of DRF was observed less than 0.4 mg/L. This was due to the fact that detention time of pre-DRF chamber did not meet the time requirement of manganese oxidation-because manganese is much more slowly oxidised through aeration than iron and not only that Mn oxidation through chlorine requires 2-4 hours to react completely.

MSFU-2 (Chala, Sirajgonj)

Fig.2 represents the variation of average manganese concentration with operation period in different treatment unit processes of the MSFU-2.The initial concentration of manganese in the effluent of URF and DRF were found to be 0.35 and 0.16 mg/L indicating removal efficiency of 52% and 77% respectively. With the passage of time manganese concentration in the effluent of URF and DRF decreased upto 0.288 mg/L and 0.1 mg/L indicating removal efficiency of 60% and respectively 86%. The figure 4.11.1 and 4.11.2 shows that manganese removal efficiency of MSFU-2 was not as effective as MSFU-1. One reason is manganese removal is a function of raw water manganese concentration .i.e. higher the Mn concentration, greater is the removal performance .Since tube well water manganese concentration of MSFU-2 is less than MSFU-1,so manganese removal efficiency of MSFU-2 was<MSFU-1. Another reason is total number of users of MSFU-2 > total number of users of MSFU-1.So detention time available for manganese oxidation in all the chambers of MSFU-2 was less than that of MSFU-1. This figure indicates that residual manganese concentration in the effluent of DRF successfully satisfied the WHO health based guideline value (0.4 mg/L) to protect against neurological damage. Due to less detention time and more use of the plant, pre DRF processes was unable to maintain the WHO health based guideline value for manganese (0.4 mg/L) in the effluent.

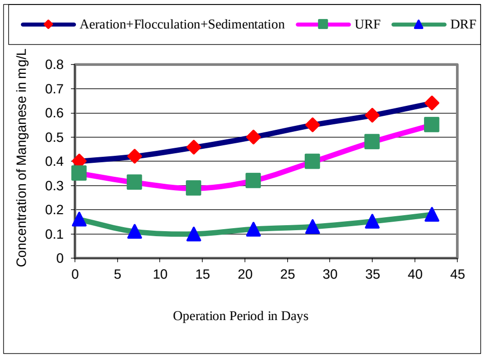

Fig.2. Variation of average manganese Concentration with operation period in different treatment unit process of Chala

Manganese Removal Performance of Different Treatment Unit Processes

Following Fig. 3 elaborates the average manganese removal performance of the different Treatment unit processes of the Multi-stage Filtration Units. Appreciable amount of manganese reduction have been occurred through Aeration cum Flocculation cum Sedimentation Processes (33%).

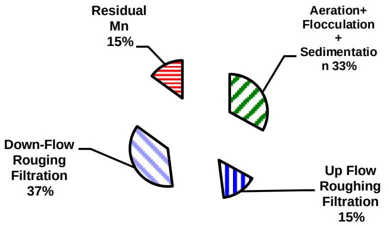

Fig.3. Manganese removal performance of different treatment unit processes (average of Kodda and Chala)

Effect of DRF process alone in removing manganese was found very significant (37%).Role of URF process in reducing manganese was observed moderate (15%).

Effect of initial manganese concentration on manganese removal performance

Effect of initial manganese concentration on manganese removal performance of the MSFUs of Kodda and Chala have been furnished in fig.4.

It indicates that in Kodda, where manganese concentration was 1.625 mg/L and iron concentration was 16 mg/L, then residual manganese concentration was detected around 11% (0.178 mg/L) in the treated water and in Chala, where manganese concentration was 0.72 mg/L and iron concentration was 15 mg/L, then residual manganese concentration was detected around 20% (0.144 mg/L) in the treated water. From this it can be concluded manganese removal performance was observed to be a function of raw water manganese concentration. Higher was the manganese concentration, greater was the removal performance.

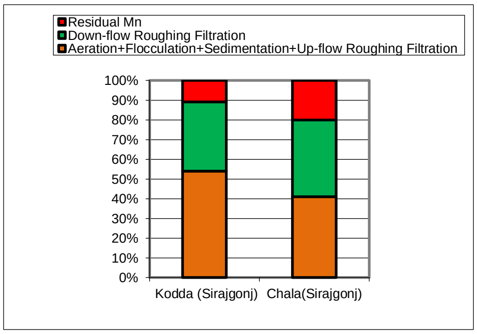

Fig. 4. Effect of initial manganese concentration on manganese removal performance of the MSFUs of Kodda & Chala

Conclusions

(1)Multistage filtration can be used effectively in removing manganese from groundwater.

(2) Around 33%,15% and 37% manganese removal have been occurred through pre-URF (aeration + flocculation + sedimentation),URF and DRF (post URF) processes respectively

(3) Manganese removal performance was observed to be a function of raw water manganese concentration. Higher was the manganese concentration, greater was the removal performance.

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A CASE STUDY ON PERFORMANCE OF CONCRETE BLOCK MAT FOR RIVER BANK PROTECTION USING SCALE MODELING

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Abstract

This paper presents the findings of the research on Concrete Block Mat (CBM) - an innovative idea produced by Manob Hitoishi Sangstha (MHS) for river bank protection over the river Arial Khan at Ramarpole near Mollarhat bazar of Kalkini upazila under Madaripur district using scale modelling. The river reach of about 1.0 km with an average width about 133m of Arial Khan is reproduced in this research. The research is conducted on an open-air mobile bed of RRI having undistorted scale 1:30 and model is designed to satisfy both flow & sediment transport condition at the same time. In this research, different application test runs are conducted with different test scenarios using low, medium and high flows. The research shows that the proposed technology (CBM) is not so effective as compared to the conventional method though its cost estimate seems to be comparatively less. The construction of concrete blocks, filter placement under water, block placement through the wire etc. are found very complex in the model but in nature it might be more complex. CBM might be very difficult to implement in the field and its construction is time-consuming. It needs special working technology to construct in the field. This concept can be applied in any small river as a pilot project to investigate its effectiveness as well as to identify its complexity in the field.

Keywords: *Arial Khan river, concrete block mat (CBM), complexity, effectiveness, river bank protection, scale modeling and technology.*

Introduction

The Arial Khan river is a distributary of the Padma river. The river maintains a meander channel throughout its course and it is erosional in nature. Due to severe river bank erosion, a number of settlements have already been destroyed and the process is going on. Bangladesh Water Development Board (BWDB) has taken some measures to save these areas using conventional method. Concrete block mats and placed concrete blocks with filter are the developments of conventional loose concrete block placement and loose concrete block dumping. This idea and technique have been developed by Manob Hitoishi Sangstha (MHS). Before model testing, it is expected that the effectiveness of Concrete Block Mat (CBM) will be more than the conventional method of river bank protection and expenditure will be less.

From approximate technical and financial analyses, it is found that river bank erosion control using concrete block mats and placed concrete blocks with filter are the best effective substitute and much cheaper than the conventional loose concrete blocks placement and dumping for the control of river erosion. The technical and financial aspects of newly proposed protective measure has been tested through laboratory research using scale modelling on bank erosion control of Arial Khan river at Madaripur District.

Making contact with the concerned water resource specialists of different organizations, it is concluded that the proposed new river bank protective measure using concrete block mats and placed concrete blocks with filter can be applied to the bank of river after model test at RRI. The test can be done in the laboratory for a location along the bank of river in low, medium and high flow condition.

The overall objective of the study is to evaluate and determine the performance of concrete block mats and placed concrete blocks with filter in river bank erosion control compared to conventional methods.

Methodology

The research is done on a mobile bed. The hydraulic similarity is established in the model to an undistorted scale. The model is constructed to an undistorted scale. The scale ratio is selected as 1:30. The model has been designed to fulfil both flow and sediment transport criterion simultaneously. It means the model velocity is higher than the critical flow velocity for the initiation of sediment motion. This is because for any velocity higher than the critical, the scour dimensions are only function of flow direction and structure geometry. The model will, therefore, reproduce the scour holes correctly.

An open air model bed of RRI has been selected for model development. It provides all kinds of facilities related to model study. Then layout of

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model is given by grid system. After setting reference grid points in the model, channel planform is given using these grid points and the bed & bank levels are fixed up by levelling instrument as per bathymetry using Rise & Fall method. This requires some cutting and filling of sand from the model bed. In this scale model, various types of instrument and facilities are needed such as, a sharp-crested weir for measuring flow, point gauge for measuring water level, 3-D current meter for measuring velocity, high resolution camera for taking video and photographic view of model, stopwatch for taking instant time and plastic colored balls (floats) for tracing flow path of flowing water. The discharge in the model is measured using sharp-crested weir at the inflow section using Rebock's formula. Model velocity is quantified by current meter. Water slope can be found by analyzing the water level measurements of different point gauges installed in the model. Flow lines of the stream can be identified by dropping colored balls starting from calibration section and catching them at the end of the model. A stopwatch can be used to calculate surface velocity of the flow. In the scale model, model data requires to be analyzed for interpretation of test results.

Model Setup

The river reach about 1.0 km length and average width about 133m is reproduced in the model. The model is run with different test scenarios using low, medium and high flows. The CBM protective work is from C/S53 to C/S57 covering a reach of about 168m along the R/B of the river. Model bed and bank are composed of fine sand having D_{50} about 0.16mm. Ebb discharge corresponding to LWL, 2.33-yr WL & 100-yr WL is taken into account to investigate the model. Two different discharge conditions used, one is Froudian discharge and the other is scour discharge. Froudian discharge provides flow pattern & velocity field as a whole and the scour discharge focuses on the scour simulation and sediment transport. Each test of the model continues until a dynamic equilibrium condition is reached and Froudian discharge is run for 12 hrs.

The model is setup in the outdoor model bed (60mX40m) of RRI. On the basis of topographic, bankline and bathymetric survey of March 2017, the model bed is constructed. After calibration of the model, application tests are conducted with the discharges corresponding to LWL, 2.33-year WL & 100-year WL. These data are used to measure flow velocity, scour depth and float tracking. The layout of the model is shown in **Fig. 1.**

Test Scenarios

In the model, 2 (two) calibration tests (T0-1 & T0-2) and 17 (seventeen) application test runs (T1-T17) were conducted. 6 (six) different designs have been tested in the model with various flow conditions changing velocity & water level as mentioned in **Table 1.** These designs have been applied in tests T1, T2, T5, T8, T11 & T12. Each test run is carried out using low, medium and high flow discharge.

Results and Discussion

Different designs of CBM are tested in this research. These are shown in test T1, T2, T5, T8, T11 & T12. Among the designs tested in the scale model, the design provided in test T12 performs relatively better than other tests. In this design, the lower bank is protected by using loosely placed concrete block mats on filter and upper bank is protected by using closely placed concrete block mats with some gaps on filter using low flow. Here the gaps in the upper bank protection on filter are kept for plantation. The model bed is prepared according to the bathymetric survey of March 2017. Scour discharge is run until equilibrium condition is reached and Froudian discharge is run for 12 hrs. Using the same design tested in test T12, test T13 and T14 have been conducted in the model using medium flow and high flow respectively. The details of the optimized design of proposed concrete block mats (CBM) are as follows:

a. River bank level: 3.0mPWD
b. Low water level: 0.24mPWD
c. Block type: Holed concrete block
d. Block size used in the model: 13.33mmX13.33mmX3.33mm (40cmX40cmX10cm in proto)
e. Length of river reach to be protected: 5.6m (168m in proto)
f. Length of upper bank protection: 0.22m (6.6m in proto)
g. Length of lower bank protection: 1.15m (34.5m in proto)
h. Length of RCC pipe: 12 inch (360 inch in proto)

i. Diameter of RCC pipe: 0.5 inch (15 inch in proto)
j. Width of filter: 12 inch (360 inch in proto)
k. Total nos. of CC blocks in each 12 inch (360 inch in proto) wide strip at top & bottom layer in lower bank: 58x5+58x5=580
l. Here each 12 inch (360 inch in proto) wide strip contains 5 column of CC blocks at the top layer & 5 column of CC blocks at the bottom layer. Each column containing 58 blocks. These strips are overlapped by 50 % over each other. The top 5 column of CC blocks have been placed over the spaces among CC block columns at the bottom layer. Here 2 layers of CC blocks & 2 layers of filter in each strip.
m. Length of u/s termination 0.75 m (22.5m in proto) & d/s termination 0.50 m (15m in proto)
n. The design of CBM tested in test T12, T13 & T14 is found to work better in spite of its other complexity. The optimised design is shown in Fig. 4, 5 & 6 (a&b).

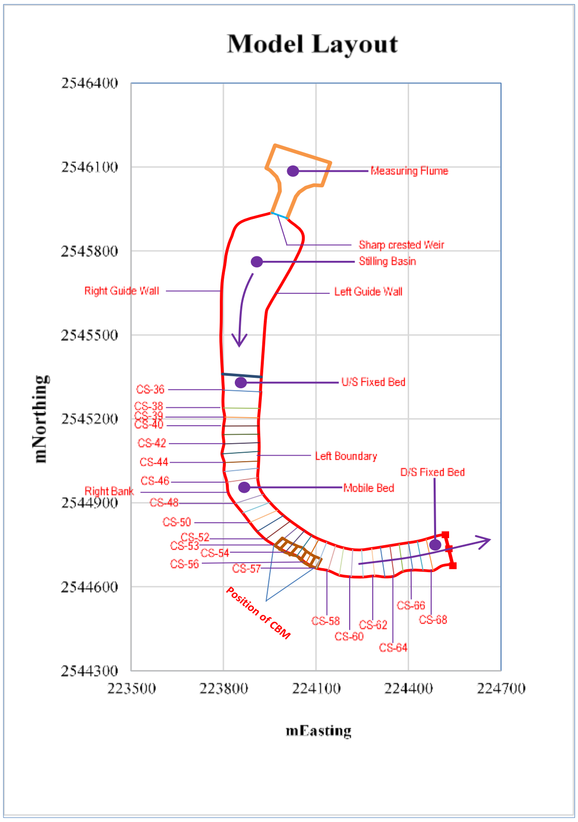

Fig.1. Layout of the Concrete Block Mats (CBM) Model

Table 1. Test Scenarios of the model

Test No.	Test Scenarios & WL/Q Conditions
Calibration Test (T0-1)	○ Test with existing conditions & 2.33-year RP water level ○ WL= 1.23 mPWD ○ $Q_{sectional}$ =1309 cumec (C/S39)
Calibration Test (T0-2)	○ Test with existing conditions & using field data ○ WL= 0.73 mPWD ○ $Q_{sectional}$ = 340 cumec (C/S36)
1st application test (T1)	○ Design supplied by MHS ○ Lower bank protection works applying strip type concrete block mats on filter ○ Upper bank protection works applying wide type concrete block mats on filter ○ Low flow WL= 0.24 mPWD ○ $Q_{sectional}$ =808 cumec
2nd application test(T2)	○ Modification of design based on test T1 ○ Low flow WL= 0.24 mPWD ○ $Q_{sectional}$ =808 cumec
3rd application test(T3)	○ Same design as in test T2 ○ Medium flow WL= 1.23 mPWD ○ $Q_{sectional}$ =1381 cumec (C/S36)
4th application test(T4)	○ Same design as in test T2 ○ High flow WL=2.93 mPWD ○ $Q_{sectional}$ = 1967 cumec
5th application test (T5)	○ New design supplied by MHS ○ Lower bank protection works applying loosely placed concrete block mats on filter ○ Upper bank protection works applying closely placed concrete block mats with some gaps on filter ○ Low flow WL= 0.24 mPWD ○ $Q_{sectional}$ =808 cumec
6th application test (T6)	○ Same design as in test T5 ○ Medium flow WL= 1.23 mPWD ○ $Q_{sectional}$ =1381 cumec
7th application test (T7)	○ Same design as in test T5 ○ High flow WL=2.93 mPWD ○ $Q_{sectional}$ = 1967 cumec
8th application test (T8)	○ Modification of new design based on test T5 ○ Low flow WL= 0.24 mPWD ○ $Q_{sectional}$ =808 cumec
9th application test (T9)	○ Same design as in test T8 ○ Medium flow WL= 1.23 mPWD ○ $Q_{sectional}$ =1381 cumec
10th application test (T10)	○ Same design as in test T8 ○ High flow WL=2.93 mPWD ○ $Q_{sectional}$ = 1967 cumec
11th application test (T11)	○ Different design supplied by MHS ○ Low flow WL= 0.24 mPWD ○ $Q_{sectional}$ =808 cumec
12th application test (T12)	○ Final design supplied by MHS ○ Low flow WL= 0.24 mPWD

Test No.	Test Scenarios & WL/Q Conditions
	○ $Q_{sectional}$ =808 cumec
13th application test (T13)	○ Same design as in test T12 ○ Medium flow WL= 1.23 mPWD ○ $Q_{sectional}$ =1381 cumec
14th application test (T14)	○ Same design as in test T12 ○ High flow WL= 2.93 mPWD ○ $Q_{sectional}$ =1967 cumec
15th application test (T15)	○ Same design as in test T12 with introduction of oblique flow ○ Low flow WL=0.24 mPWD ○ $Q_{sectional}$ = 808 cumec
16th application test (T16)	○ Same design as in test T12 with introduction of oblique flow ○ Medium flow WL= 1.23 mPWD ○ $Q_{sectional}$ =1381 cumec
17th application test (T17)	○ Same design as in test T12 with introduction of oblique flow ○ High flow WL= 2.93 mPWD ○ $Q_{sectional}$ =1967 cumec

The optimized design of CBM is subjected to angle of flow attack. Here a char is artificially reproduced in the model which makes an angle of 140-degree oblique flow attack with the incoming flow as per recommendations of Chief Engineer (Design), BWDB. Under oblique flow attack, test T15, T16 & T17 have been conducted in the model using low, medium & high flow condition respectively following the same optimized design. Scour and velocity were measured in the vicinity of CBM and oblique char. Maximum local scour around CBM structure & oblique char is respectively 7.65m (-22.27mPWD) & 7.8m (-22.45mPWD) in test T17 under oblique flow condition. Maximum velocity measured around the top of CBM (along the right bank) is 2.36 m/s, 2.83m/s & 3.64m/s respectively in test T15, T16 & T17 with the introduction of oblique char.

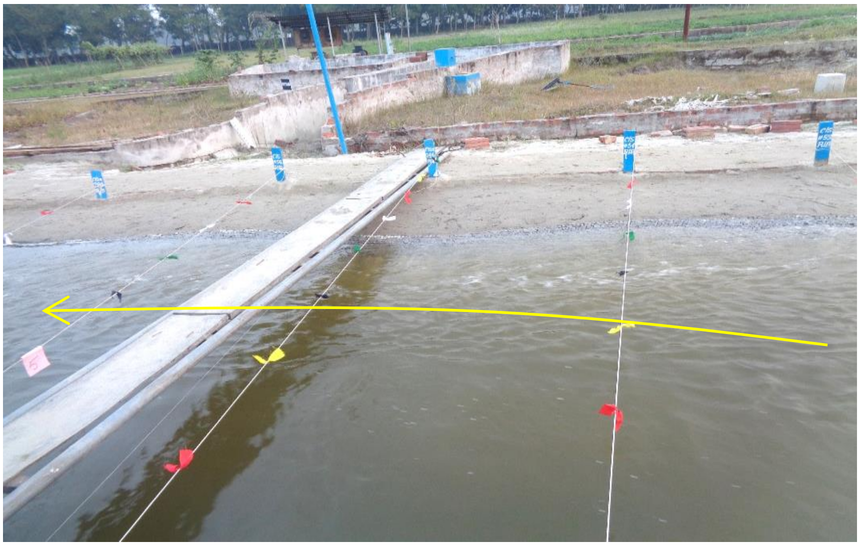

Fig.2. Model is under running condition with CBM (T14)

Fig.3. Scour pattern around CBM after run (T14) in the model

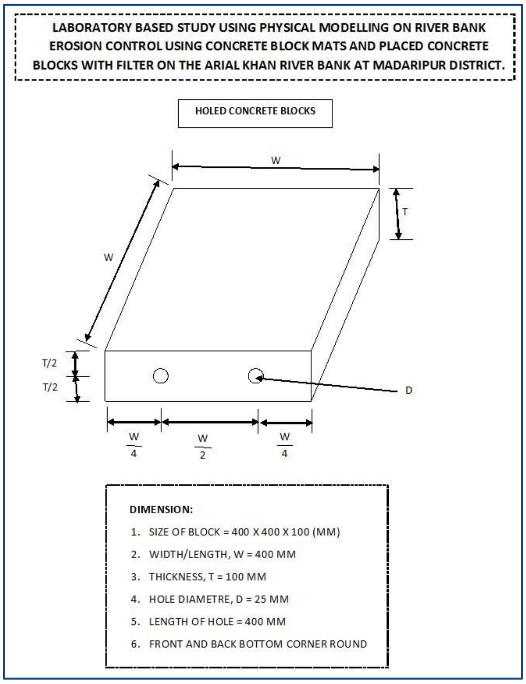

Fig.4. Typical holed block of CBM

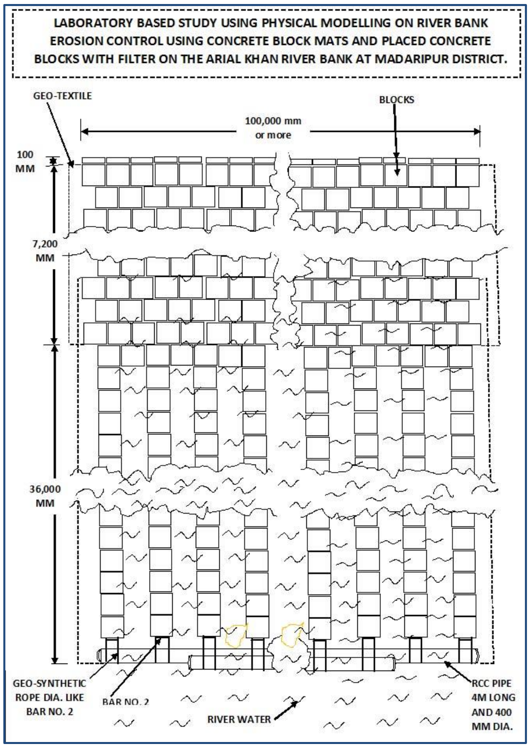

Fig.5. Typical plan of erosion control measure by CBM at Arial Khan River bank

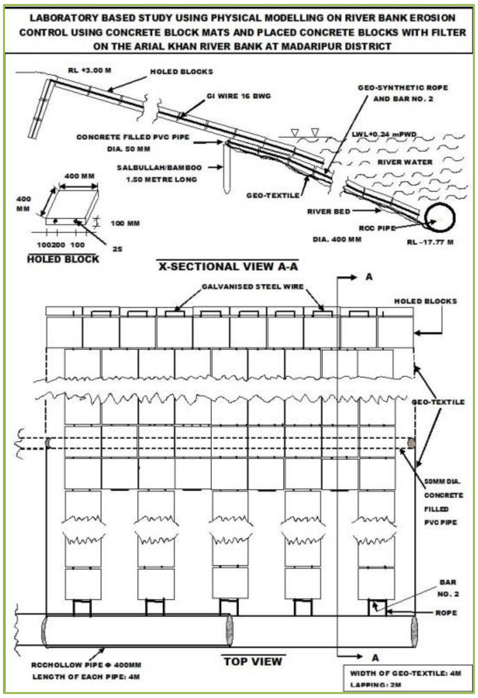

Fig.6 (a). Typical section (A-A) of erosion control measure by CBM at Arial Khan River bank

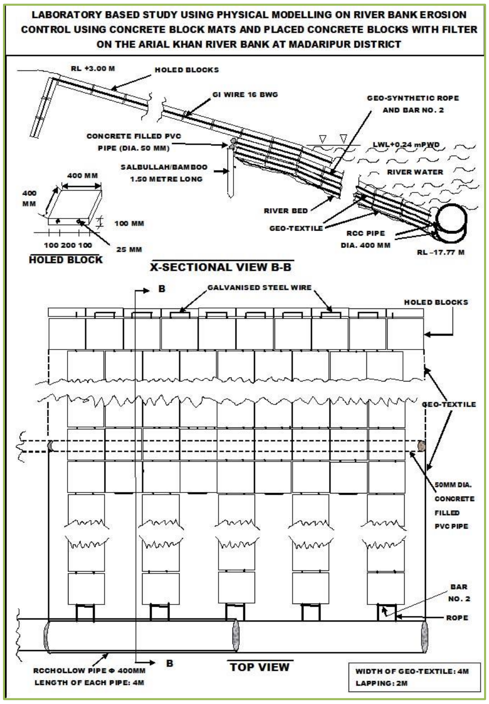

Fig. 6 (b). Typical section (B-B) of erosion control measure by CBM at Arial Khan River bank

Cost Comparison

The concerned field office of BWDB at Madaripur was requested to extend their cooperation to analyze the cost of protection measure applying loose concrete blocks and proposed concrete block mats for a same location. The location is selected at Ramarpole, Mollarhat under Kalkini upazila of Madaripur district along the bank of Arial Khan River. Scale modeling was done referring to this location. From the above comparison it is found that the protection measure applying new measure of concrete block mats is about 30% less than the cost of protection measure applying conventional measure of loose concrete blocks.

Comparative Statement between Conventional & Proposed Concrete Block Method

A comparative statement made by MHS between conventional method and proposed concrete block method with respect to Technical Performance, Workability and Sustainability for river bank protection is given below:

Table 2. Comparison of conventional & proposed new method

	Conventional Method	Proposed Concrete Block Method
Technical performance	a) In conventional method, loose concrete blocks are used at upper bank and lower bank for river bank erosion control. So, the placed position of individual blocks can change easily due to soil erosion. b) Block can move from one position to other due to free connection with other blocks when underlying soil erode. c) In lower bank, huge blocks are accumulated by dump which attract sedimentation, blocks can move to the river bed. So, river depth can decrease.	a) In proposed new method, concrete block mats can be used at upper bank and lower bank for river bank erosion control. So, the placed position of individual blocks can't change easily due to erosion. b) Block can't move from one position to other due to inter connection with other blocks when underlying soil erode. But, direction of block can change due to flexible inter connection. c) In lower bank, two layer concrete block mats can be used which don't attract sedimentation, blocks can't move to the river bed. So, river depth can increase.
Workability	a) Easy to construct in the field. It is practiced from British period. b) Workers are already acquainted with the construction procedure due to practice for a long time. c) For construction no diver, swimmer and barge required.	a) Not easy to construct in the field but possible. b) Workers are not acquainted with the construction procedure. So, training will be required. c) For construction diver, swimmer and barge may be required.
Sustainability	a) Sustainable if huge expenditure for maintenance is ensured. b) Less durable than new method.	a) Sustainable with negligible expenditure for maintenance. b) More durable than the conventional method.
Cost comparison	a) Cost for 314m length protection by Conventional Loose Concrete Blocks is Taka 5,05,19,318.00	a) Cost for 314m length protection by Proposed Concrete Block Mats is Taka 3,58,65,859.00

Conclusion

- The effectiveness of the proposed CBM is not so attractive compared to the traditional method as a whole, though the cost estimate supplied by the Senior Design Engineer appears to be relatively less.
- The construction of concrete blocks, filter placement under water, block placement through the wire etc. are found very complex in the model. But in nature it might be more and more complex.
- The construction of proposed bank protective structure (CBM) might be very difficult to implement in the field. The construction of proposed new method of bank protection (CBM) is time-consuming.
- CBM needs special working technology to construct in the field.
- The optimized design tested in test T12 is found to work better compared to other tests.
- Proper monitoring & supervision of CBM is necessary during and after construction phase if implemented in the field.

Recommendation

This concept can be applied in any shorter reach of a small river as a pilot project to verify its effectiveness as well as to identify its various complexity in the field condition.

Acknowledgement

The Authors acknowledge BWDB for providing financial support and MHS for technical assistance to carry out this study. The authors are extremely indebted to the Executive Engineer, Madaripur O&M Division, BWDB for providing insight and expertise that greatly assisted the research. The Author's deepest gratitude is also extended towards a number of technicians of RRI, Faridpur.

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SPATIAL AND TEMPORAL DISTRIBUTION OF HEAVY METALS IN THE BURIGANGA RIVER

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Abstract

River pollution is identified as one of the top environmental issues during the couple of decades in Asian region. The Buriganga is considered the most important and polluted river in Dhaka. Disposal of untreated industrial and domestic waste, encroachment and dumping of solid waste have degraded the overall quality of the river Buriganga. The present study investigated the spatial distributions of heavy metals in water at sixteen different sites spread over whole stretch of the Buriganga River (27km). The concentrations of seven metals Cr, Cd, Pb, Ni, Fe, Zn and Cu were analyzed using an Atomic Absorption Spectrophotometer (Thermo-Scientific, 3000 series) in the RRI laboratory. The concentrations were compared with several standard Guideline values provided by different organizations like WHO, DoE, FAO and CCME. The visualization of the spatial pattern of individual metal throughout the Buriganga was primed using ArcGIS 10.3 software. The result showed that the surface water of the whole stretch of the Buriganga was severely polluted by these heavy metals in dry season except Cu and Zn. The statistical analysis showed that a wide variation of concentrations among Cr, Cd, Pb, Ni, Fe, Zn and Cu but slightly differed among the locations. Anthropogenic activities are mainly responsible for elevated levels of the measured metals in river water. The lower concentration of heavy metal was found in post rainy season compared to dry season. Even though the concentration has decreased in post rainy season some severe toxic heavy metals like Cr and Cd concentrations are far above than the safe recommended values. Prevention of metals entering in to the rivers should be enforced to save ecosystem of the watershed environment of the Buriganga.

Key words: *Heavy metal, Pollution, Buriganga River, ArcGIS, Spatial distribution, Seasonal variation, Environment, Industrial waste.*

Introduction

Unintended rapid urbanization and industrial growth have triggered serious concerns in environment. Heavy metal contamination in aquatic environments has received huge concern due to toxicity of metals, abundance and persistence in the environment and subsequent accumulation in aquatic habitats. Rivers are a dominant pathway for metals transport (Hasan, *et al*., 2014) and undergo a global ecological cycle (Afrin, *et al*., 2014). Heavy metals are naturally occurring metallic elements that have relatively high atomic weight and density compared to water (Tchounwou, *et. al.,* 2012) and usually non-biodegradable (Mahfuza, *et al*., 2012). It can concentrate along the food chain, producing toxic effect at points after far removed from the source of pollution (Tilzer and Khondker, 1993).

Metal pollution has harmful effect on biological systems and therefore, exposure to heavy metals has linked to several human diseases such as malformation, kidney damage, cancer, abortion, effect on intelligence and behavior, and even death in some cases of exposure to very high concentrations (Ghrefat and Yusuf, 2006). The toxicity depends on several factors including the dose, route of exposure, and chemical species, as well as the age, gender, genetics, and nutritional status of exposed individuals. Because of high degree of toxicity, cadmium (Cd), chromium (Cr), lead (Pb), nickel (Ni), arsenic (As), mercury (Hg), cupper (Cu), zink (Zn) and iron (Fe) rank among the priority metals that are of public health significance. These metallic elements are considered systemic toxicants that are known to induce multiple organ damage, even at lower levels of exposure (Pehlivan, *et al.,* 2009). The World Health Organization (WHO) as well as the Food and Agriculture Organization (FAO) of the United Nations state that monitoring of toxic heavy metals like Cr, Cd, Pb and Ni are mandatory and others are suggestive in aquatic system.

The Buriganga is subject to severe pollution and considered as one of the worst polluted rivers in the world. The dyeing factories and tanneries are the main polluters of the Buriganga. Department of Environment (DoE) reports that the tanneries collectively dump 22,000 liters of toxic waste including cancer-causing chromium into Buriganga every day (Barton, 2011). Waste from the industries is usually connected to the sewerage system that directly follows to the river

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without any treatment. Moreover, the river has become a dumping ground of all kinds of solid, liquid and other chemical waste. The river is losing natural flow due to river encroachment and dumping of solid waste. Therefore, the river Buriganga is increasingly being polluted everyday with heavy metals containing toxic wastes through city's thousands of industrial units and sewerage lines (Islam *et al*., 2006).

A number of studies have been reported on the pollution of the Buriganga River. Some of the previously studies described only the physicochemical properties and biochemistry of the river water (Ali *et Al*., 2008; Bhuiyan *et al*., 2011; Sikder *et al*., 2012) and few others have dealt with the distribution of heavy metals (Ahmad, *et al*., 2010; Islam *et al*., 2015). However, the study of heavy metals pollution of entire reach of the Buriganga hasn't been carried out so far. Besides, the study of individual metal distribution in surface water of the Buriganga in GIS platform has not yet been reported.

In view of the above circumstances, a comprehensive study has been done on the distribution of Cr, Cd, Pb, Ni, Fe, Zn and Cu at sixteen different sites spread over whole stretch (27 km) of the Buriganga River with the following objectives:

i) to determine the spatial distributions of heavy metals in surface water of the Buriganga River;
ii) to investigate the seasonal variation of heavy metal concentrations; and
iii) to assess heavy metal concentration in an inclusive and integrated way using ArcGIS

Methodology

Study area and sampling site

The Buriganga River was selected as a study area. The Buriganga is a tide-influenced river flowing west and then south of Dhaka City. It is only 27 km long and its average width and depths are 400 and 10 m, respectively (Islam *et al*., 2015). Due to rapid and unplanned urbanization and industrialization the Dhaka city's surrounding rivers, including Buriganga have gradually experienced undue and unbearable pressure to their existence. To investigate the spatial pollution pattern, sixteen different sites spread over the whole stretch of the Buriganga river were selected in this study and shown in Fig. 1 and Table 1.

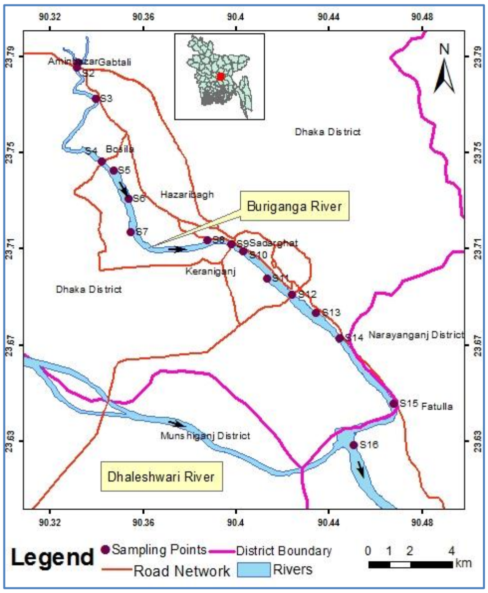


Fig. 1. Map of study area with sampling site

Sample collection procedure

Water samples were collected in 2015 in dry and post rainy seasons. The month of March is considered to be representative of dry season where as the October as treated as post rainy season assuming the river water become stable in this month after rainy season. In dry season, water samples were collected from 16 sites. However, to observe the seasonal variation of metals, water samples were collected only from 4 sites namely, Amin Bazar, Hazarribagh, Saderghat and Fatulla in post rainy season. The water samples were collected from 10 to 15 cm below the water surface using acid washed 1 litre HDP plastic bottle. Prior to sample collection, the bottles were rinsed with the river water to avoid contamination in the bottle. At each sampling site, a composite sample was collected by taking 3 numbers of samples at 1 m interval. The collected samples were immediately acidified with HNO_3 and the bottles were carefully closed to avoid contact with air. The water samples were transferred to the laboratory as early as possible and were stored at 4°C in a refrigerator.

Table 1. Locations and description of sampling sites along the Buriganga River (total reach 27 km).

Site	Location	Coordinates	Description (Reason for choosing location)
S1	Ishakha Badh	23°47'10.65"N 90°20'8.45"E	It is the confluence of Turag, Karnatoli and Buriganga river.
S2	Amin Bazar Bridge	23°47'1.69"N 90°20'8.28"E	It is the commencing point of the Buriganga river with the joint flow of Turag and Karnatoli Rivers. This point contains various quays of different types of transport.
S3	Adabor Sluice Gate	23°46'18.78"N 90°20'32.69"E	Buriganga river and Kallyanpur Canal get to meet here through a sluice gate.
S4	Bosila	23°44'38.61"N 90°20'46.27"E	At this point many drains are connected to the Buriganga river carrying municipal waste. There is tanneries waste disposal canals linked to the river.
S5	Hazaribagh Canal	23°44'24.52"N 90°21'5.00"E	A big canal carrying waste water of tanneries of Hazaribagh area runs into the Buriganga river at this site.
S6	Gazirghat	23°43'41.24"N 90°21'28.03"E	Some Canals carrying municipal wastes and untreated water of tanneries meets the river at this point.
S7	Kholamora boat terminal	23°42'51.54"N 90°21'31.39"E	A rivulet fall into the Buriganga river at this point. There is also many engine driven boats come and go.
S8	Sowarighat	23°42'38.27"N 90°23'30.26"E	A big canal carrying municipal and industrial waste of Kamrangirchar and Shahidnagar area gets joint to this point. There is a boat terminal present there.
S9	Babubazar Bridge	23°42'33.11"N 90°24'7.70"E	A storm water drain carrying municipal waste from old Dhaka area is connected to the Buriganga river at this point.
S10	Sadarghat	23°42'21.59"N 90°24'26.24"E	Sadarghat Launch terminal is the largest river port in Bangladesh. Oil and lube spillage happens during refueling of vessels and cargo handling. These vessels dump waste, including burnt oil, into the water.
S11	Mirerbag	23°41'40.06"N 90°25'2.82"E	There is a launch terminal and several dockyards at this point which is the reason to choose this point.
S12	Postogola	23°41'16.05"N 90°25'41.75"E	Some canals carrying municipal and industrial waste of Postogola area meet at this point with the Buriganga river.
S13	Monsikhola	23°41'0.63"N 90°26'2.48"E	It is the point where local brick traders run their business through loading and unloading bricks to ships of various sizes.
S14	Pagla	23°39'46.80"N 90°27'14.04"E	One of the biggest sewage treatment plants of Dhaka is situated about 1km away from this point and it lies in the middle of the quay used to transport bricks.
S15	Fatullah	23°38'26.15"N 90°28'21.33"E	Fatullah Bazar is one of the significant market places of Narayanganj district and contains a busy launch terminal. This point lies in the launch terminal.
S16	Dharmaganj	23°37'39.75"N 90°27'14.87"E	A branch of Dhaleswari river meets Buriganga river in this point and continues to flow further to meet Shitalakhya and Meghna river with several dockyards and brick fields around.

Chemical analysis of water samples

All reagents using in the laboratory were analytical grade. Deionized (DI) water was used for the preparation of all solutions. All glassware used in this study were cleaned by soaking in dilute acid for at least 24 h and rinsed abundantly in deionized water and dried before use.

For heavy metal analysis, water samples were prepared and analyzed according to Sharma and Tyagi, 2013. Briefly: Transferred 100 ml of well mixed acid preserved sample into a beaker and added 2 ml of concentrated HNO_3 + 5 ml of concentrated HCl. The beaker was placed on a hot plate at 90 to 95 ^{0}C and reduced the volume up to 10-20 ml. Then the beaker was removed and allowed to cool. The beaker was washed with deionized water (3 times) and filtered through a Whatman filter paper no. 42. Then the sample was poured into 100 ml volumetric flask and made it up 100 ml mix thoroughly. The sample was poured to HDP plastic bottle and kept in refrigerator for Heavy metals analysis.

The standard solution of the elements Cr, Pb, Cd, Ni, Cu, Fe and Zn were prepared by pouring the required amount of the solution from the stock solution, manufactured by Fisher Scientific Company, USA. The standard solution was prepared before every analysis of the current work. The samples were analyzed by using air acetylene flame with combination, as well as, single element hollow cathode lamps into an atomic absorption spectrophotometer (Thermo-Scientific, 3000 series). During quantifying metals, at least three concentrations were prepared of standard solution of a particular metal. Blank solution was aspirated and adjusted to zero. Each standard solution was aspirated into flame and prepared a calibration curve for absorbance versus concentration of standard solution. The appropriate dilution factor was used for the samples having higher concentration of metal ions. Average values of three replicates were taken for each measurement.

Statistical analyses

Statistical analysis has been done using GENSTAT 12th Edition (VSN International, Hemel Hempstead, England) for heavy metals.

Mapping procedures

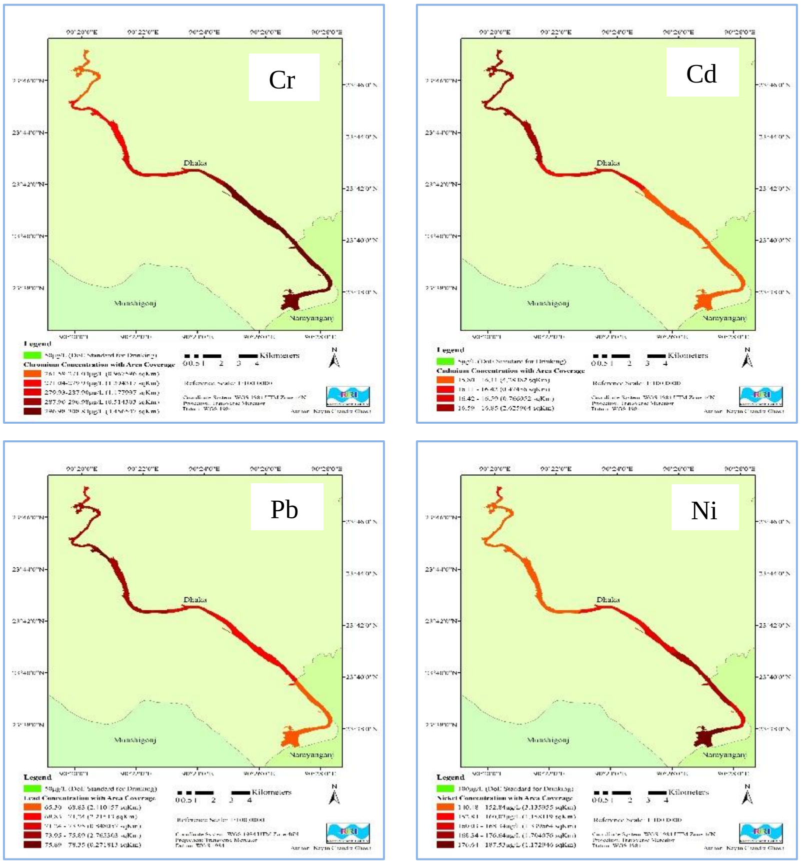
Distribution of heavy metal concentrations to the whole river reach was done by kriging ordinary interpolation method using ArcGIS 10.3. The interpolated data were reclassified into 5 classes with natural breaks (jenks) for convenience. The color of the class was choosen depending on the DoE standard with green and red for below and above respectively. The intensity of the color with lower to higher represents dark to light for green and light to dark for red respectively.

Results and discussions

Spatial distribution of metals in the Buriganga River water in dry season

The spatial pollution pattern by seven heavy metals at sixteen different sites spread over the whole stretch of the Buriganga river in dry season are visualized in Fig. 2 and Fig. 3. The whole stretch of the Buriganga was severely contaminated by four toxic metals Cr, Cd, Pb and Ni in dry season (Fig. 2). The concentration of Cu and Zn were acceptable level whereas Fe concentration was fluctuated in dry season at different sites of the Buriganga stretch (Fig. 3).

Among the 16 sites, the maximum Cr concentration was 346.76 μgL^{-1} at Postogola which is approximately seven times and the minimum was 223.82 μgL^{-1} in Adabor Sluice Gate which is approximately four times greater than standard value (50 µg/ L) provided by WHO & DoE (Table 2 and Table 3). The concentration of Cr was 236.06 and 289.90 μgL^{-1} at Ishakha Badh and Amin Bazar Bridge respectively, which are the commencing point of the Buriganga River indicates the Buriganga initiates with the Cr polluted water flow. The concentration of Cr was slowly increased throughout the flow of water of the river (Fig. 2). The Cr concentration was not significantly differed among the locations indicating whole stretch of the river is Cr polluted. According to CCME (2007), required Cr concentration is 0.02 μgL^{-1} to protect fish and 2.0 μgL^{-1} to protect aquatic life including zooplankton and phytoplankton (Table 3). Therefore, according to CCME (2007), aquatic life awkward in this severe Cr polluted river water. For the reasons of this high level of Cr that the Hazaribagh Tannery industries discharging their solid wastes and liquid effluent containing rotten flesh, fat, blood and skin, toxic chemicals, dissolved lime, chromium sulfate and alkali, hydrogen sulfide, heavy metals, suspended solids, etc., in most cases without any treatment directly to the river Buriganga (Zahid, 2004) at different places such as Basila, Hazaribagh, Kamrangir char etc. Moreover, Turag and Bongshi are situated up stream of the Buriganga and these rivers also carries considerable amount of Cr which flowing through the Buriganga. In this study, the excess amount of Cr found in the Buriganga is due to upstream Cr containing water flow along with Cr containing tannery and other industries wastes connecting to the Buriganga. Islam, *et al.* (2015) showed that Cr concentration in Buriganga 110 μgL^{-1} in summer which is lower than this study. However, Ahmad, *et al*. (2010) revealed that Cr concentration were 645.26, 605.87, 613.25 μgL^{-1} at Balughat, Shawaryghat, Foridabad, respectively, in the Buriganga in Pre monsoon which were greater than this study. Difference in concentration was possibly due to the difference in collection season, time, places, amount of wastage discharge and measurement method in different studies.



Note: Red and green colour indicate above and below DoE standard level respectively for water. Intensity of color is proportional to concentrations.

Fig. 2. Concentration of toxic heavy metals Cr, Cd, Pb, and Ni in μgL^{-1} at different locations of the whole reach of the Buriganga in dry season.

The Cd concentration of first location (Ishakha Badh) was 15.48 μgL^{-1} which indicates oncoming flow of Briganga severely Cd polluted (Table 2). The highest Cd concentration was 22.25 at Sadarghat and the lowest was 11.67 μgL^{-1} at Munshikhola that are far above the standard value as the standard value of Cd provided by WHO is 3 μgL^{-1} and Provided by DoE is 5 μgL^{-1}. However, different studies showed that variation in Cd concentration in Buriganga. For example Ahmed, *et al*. (2010) found that Cd concentration varied from 9.21 to 10.03 μgL^{-1} and Islam, *et al*. (2015) found 10 μgL^{-1} in Buriganga which are far below than this study. The Cd pollution mainly attributed by upstream flow and different Cd containing wastes and dockyard besides the Buriganga.

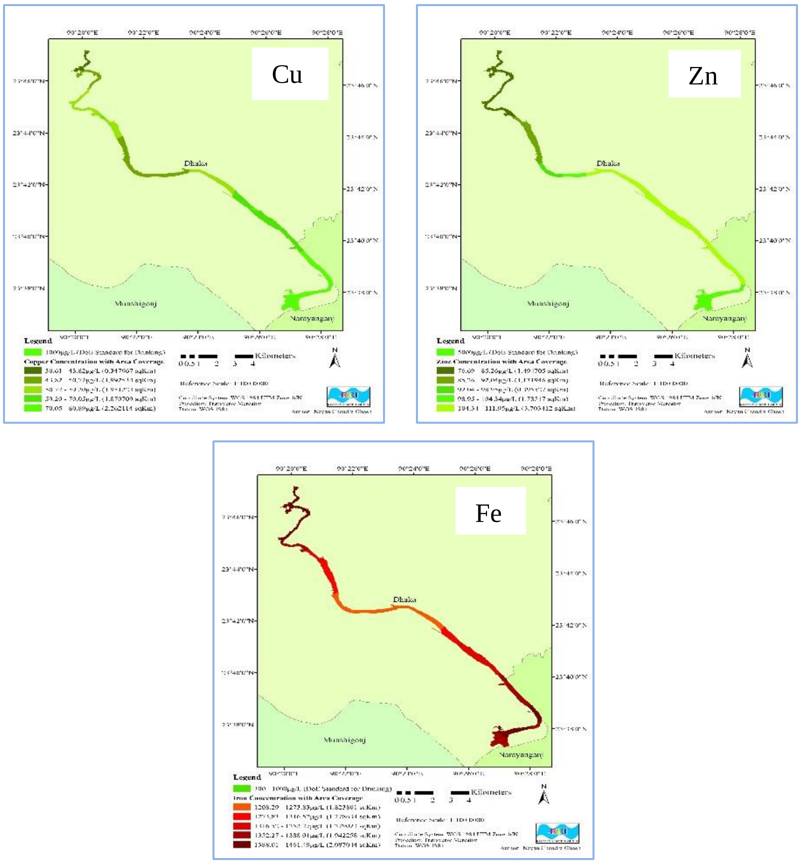
Bosila was the highest Pb contaminated site among the sites. The concentration of Pb fluctuated from 52.24 to 97.30 μgL^{-1} through the whole pathway of the Buriganaga (Ishakha Badh to Dharmaganj) which is approximately five to nine fold greater than WHO (2011) standard limit and one to two fold greater value provided by DoE (ECR, 1997). At the Ishakha Badh Pb concentration was 75.21 µgL^{-1} which represents upstream of the Buriganga contaminated by Pb containing many different industrial, agricultural and domestic wastes. Similar findings were observed Ahmed, *et al.* (2010) in some places like Swaryghat, Gazirghat, Balughat in Buriganga in Pre-Monsoon.

Table 2. Level of heavy metals in µgL^{-1} in Buriganga River water at different locations in dry season

Site	Location	Cr	Cd	Pb	Ni	Fe	Zn	Cu
S1	Ishakha Badh	236.06	15.48	75.21	170.15	1601.27	58.75	58.79
S2	Amin Bazar	289.90	19.82	53.94	158.91	1706.23	70.33	54.30
S3	Adabor Sluice Gate	223.82	16.35	81.12	129.87	1461.91	73.15	54.07
S4	Bosila	314.36	18.77	97.30	158.40	1499.36	89.04	78.38
S5	Hazaribagh	282.46	13.42	84.23	135.69	1022.34	104.01	46.03
S6	Gazirghat	256.82	16.66	60.84	130.84	1507.68	61.93	44.57
S7	Kholamora Boat Ghat	285.50	14.74	83.10	146.74	1058.89	66.86	48.45
S8	Swaryghat	296.28	13.54	74.50	172.89	1219.73	141.69	68.18
S9	Babubazar Bridge	270.82	13.61	52.24	157.03	995.55	94.06	55.06
S10	Sadarghat	332.53	22.25	63.74	153.60	1166.57	98.98	54.22
S11	Mirerbag, Balighat	269.25	14.79	85.85	161.07	1216.96	111.77	49.73
S12	Postogola	346.76	18.83	74.01	155.75	1909.19	78.84	89.89
S13	Munshikhola	300.41	11.67	79.09	175.21	1273.56	151.48	62.18
S14	Pagla	324.14	21.83	68.02	182.14	1188.59	118.41	62.62
S15	Fatulla	262.72	13.06	42.87	135.40	1612.55	118.39	86.32
S1 6	Dharmaganj	334.61	16.87	79.16	208.15	1568.41	76.61	77.39

Table 3. Guideline value for heavy metals in µgL^{-1}set by different organizations for Drinking water, Irrigation water and Aquatic life Purposes

Parameter	Drinking water			Irrigation water	Aquatic Life
	WHO (2011)	DoE (ECR,1997)	CCME (2007)	FAO (1994)	CCME (2007)
Cr	50	50	50	100	0.02 µgL^{-1}, To protect fish 2.0 µgL^{-1}, To protect aquatic life including zooplankton and phytoplankton
Cd	3	5	5	10	0.2 µgL^{-1} for Hardness 0–60 mg/l ($CaCO_3$) 0.8 µgL^{-1} for Hardness 60–120 mg/l ($CaCO_3$) 1.3 µgL^{-1} for Hardness 120–180 mg/l ($CaCO_3$) 1.8 µgL^{-1} for Hardness > 180 mg/l ($CaCO_3$)
Cu	2000	1000	1000	200	2.0 µgL^{-1} for Hardness 0–120 mg/l ($CaCO_3$) 3.0 µgL^{-1} for Hardness 120–180 mg/l ($CaCO_3$) 4.0 µgL^{-1} for Hardness > 180 mg/l ($CaCO_3$)
Fe	300	300–1000	300	5000	300 µgL^{-1}
Pb	10	50	50	5000	1.0 µgL^{-1} for Hardness 0–60 mg/l ($CaCO_3$) 2.0 µgL^{-1} for Hardness 60–120 mg/l ($CaCO_3$) 4.0 µgL^{-1} for Hardness 120–180 mg/l ($CaCO_3$) 7.0 µgL^{-1} for Hardness > 180 mg/l ($CaCO_3$)
Ni	70	100	-	200	25 µgL^{-1} for Hardness 0–60 mg/l ($CaCO_3$) 65 µgL^{-1} for Hardness 60–120 mg/l ($CaCO_3$) 110 µgL^{-1} for Hardness 120–180 mg/l ($CaCO_3$) 150 µgL^{-1} for Hardness > 180 mg/l ($CaCO_3$)
Zn	-	5000	5000	2000	300 µgL^{-1}



Note: Red and green colour indicate above and below DoE standard level respectively for water. Intensity of color is proportional to concentrations.

Fig. 3. Level of Cu, Zn and Fe in µgL^{-1} at different locations of the whole reach of the Buriganga in dry season.

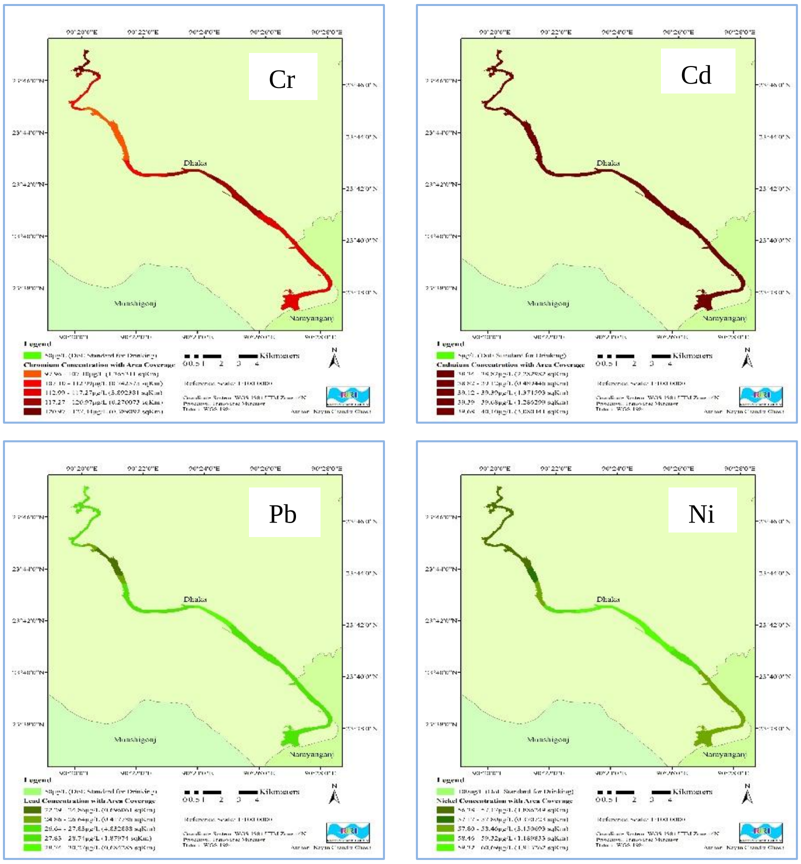
Concentration of Ni varied from 129.87 to 208.15 µgL^{-1} which are exceeded the recommended value provided by WHO (2011) and DoE (ECR, 1997). The highest level of Ni was found in Dharmaganj and it is likely because of several dockyards are present there. The higher level of Ni was also found in Ishakha Badh, Swaary ghat, Pagla and Dharmaganj indicating Ni containing industries such as alloys, stainless steel, batteries etc. are located around the places and discharging their wastes directly to the river. However, Ahmed, *et al*. (2010) observed far lower Ni concentration than this study in the Buriganga.

The surface water of the Buriganga was free from Cu and Zn pollution. Concentration of Cu varied from 44.57 to 89.89 µgL^{-1} and Zn varied from 58.75 to 151.48 µgL^{-1} which were lower that the recommended value provided by WHO (2011) and DoE (1997). Concentration of Cu at Swaryghat was 68.18 µgL^{-1} whereas Ahmed, *et al.* (2010) observed that Cu concentration was 132.18 µgL^{-1} at Swaryghat in Pre monsoon.

The concentration of Fe was fluctuated at different locations and varied from 995.55 to 1909.19 µgL^{-1}. The standard value provided by DoE (ECR, 1997) is 300 to 1000 µgL^{-1} for drinking water and 300 µgL^{-1} for aquatic life (CCME, 2007) which is lower than the values found in this study indicating iron pollution occurs. However, Fe concentration was much lower than this study observed by Sikder, *et al.* 2012. In this study, the higher level of Fe in Postogola, Fatulla and Dharmaganj is attributed by water vehicle and dockyard. Similarly, the greater Fe level in Ishakha Badh, Amin Bazar and Basila is also likely as Fe discharging metal industries such as the iron pipes, stailless still and water vehicle are available surroundings of these locations.

The concentration of Cr, Cd, Pb, Ni, Zn, Cu, and Fe showed a wide variation of concentration among the metals but slightly differed among the locations of the river (Table 2). The concentration of metals was significantly differed among them ($P < 0.001$) and was found in order of Fe> Cr > Ni > Zn > Pb > Cu > Cd. Among the investigated metals, Cr, Cd, Pb and Ni which are very toxic, exceeded the standard levels provided by WHO (2011), DoE (ECR, 1997) and CCME (2007) in all locations of the river. This findings may be related to the adsorption of the heavy metals by metal oxides or hydroxides. Major sources of these elements in river water include industrial wastes, tanneries, manufacturing processes related to chemicals and metals, contamination of water in natural geologic deposits, discharges of municipal waste, domestic wastes and atmospheric deposition.

Seasonal variation of metals

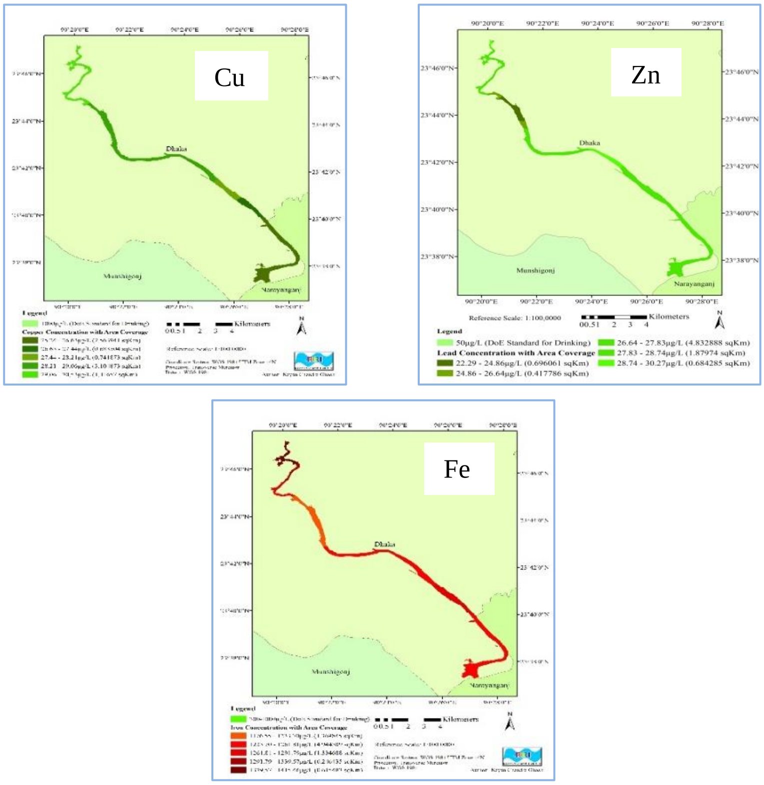


Note: Red and green indicate above and below DoE standard level respectively for water. Intensity of color is proportional to concentrations.

Fig. 4. Concentration of toxic metals Cr, Cd, Pb, and Ni in μgL^{-1} at different locations of the whole reach of the Buriganga in post rainy season.

Concentrations of heavy metals were greater in dry season compared to post rainy season (Table 2 and Table 4). The lower concentration of metals in post rainy season was because of dilution of water due to influx of rain water and flood water from surrounding areas. The dilution during higher water flow washed away much of the pollutants and decreased the concentration.

The reason could be that, during the monsoon season, polluted sediment particles may be suspended in the bottom sediment layer, which could lead to lower concentrations of heavy metals. Most of the suspended materials, which were not complex and precipitated with soil, organic matter and other compounds, were flushed out through the canal into the adjoining vast flood zone. However, the concentration of Cr, Cd and Fe were far above than the standard level provided by WHO and DoE in post rainy season. The level of Cr varied from 262.72 to 332.53 μgL^{-1} in dry season whereas 97.49 to 127.70μgL^{-1} in post rainy season. Concentration of Cr in dry season was significantly different ($P < 0.05$) than that of post rainy season. Even though Cr concentration was much lower in post rainy season than the dry season, it was approximately double than the standard level provided by WHO and DoE. The concentration of Cr at Amin Bazar, Hazaribagh, Sadarghat and Fatulla was 289.90, 282.46, 332.53 and 262.72 μgL^{-1}, respectively in dry season whereas 127.70, 97.49, 121.22 and 116.60 μgL^{-1} in post rainy season at same places. The levels of Pb, Ni, Cu and Zn were below the than the standard level provided by WHO and DoE (Fig. 4, Fig. 5 and Table 3). It is remarkable that the level of Cd is greater in post rainy season compared to the dry season. The lower concentration of the Cd in dry season also observed Islam et.al. (2015). Deviations of the results could be attributed to site-specific activities, source of waste and the flow of the river.



Note: Red and green indicate above and below DoE standard respectively for water.

Fig. 5. Level of Cu, Zn and Fe in μgL^{-1} at different locations of the Buriganga in post rainy season.

Table 4. Concentration of metals in μgL^{-1} in the water of Buriganga River in post rainy season

Season	Location	Cr	Cd	Pb	Ni	Fe	Zn	Cu
Post rainy	Amin Bazar	127.70	39.28	28.94	56.37	1418.59	38.12	34.29
	Hazaribagh	97.49	39.75	21.97	56.72	1174.02	41.99	24.75
	Sadarghat	121.22	40.12	30.47	60.75	1249.41	43.60	33.23
	Fatulla	116.60	38.33	28.67	57.87	1239.97	34.97	21.19

In post rainy season, two toxic metals Cr and Cd were found in significant amount throughout the river in this study (Fig. 4). As heavy metals are non-degradable and not decomposable it is likely to be present with the flow of water in post rainy season. During July to November rivers are full of water in Dhaka and generally the water seems good in terms of odor and color. Therefore, it is very disquieting that the river water is polluted by Cr and Cd during this period too and these two toxic metals still prevailing into the aquatic environment.

Conclusions

The present investigation demonstrates that the whole stretch of the Buriganga River is harshly contaminated by toxic heavy metals. At all 16 sites, the surface water of the Buriganga is severely polluted by metals except Cu and Zn in dry season. At the commencing point of the Buriganga, the concentration of Cr, Cd, Pb, Ni and Fe are far above level recommended by DoE (ECR, 1997) and WHO (2011) indicating the river pollution is largely dependent on the upstream river flow. Among these metals Cr, Cd, Pb and Ni toxicity are severe in the water possibly due to the discharge of high amount of these metals concentrated wastewater from industries and from surface runoff. Moreover, the level of heavy metal is lower in the post rainy season compared to the dry season perhaps due to the dilution of water and river flow. However, even though the concentration has decreased in the post rainy season some noxious heavy metals like Cr and Cd are two to five times greater than the safe recommended values. As heavy metals are non-biodegradable and not decomposable the high level of metals occurs at points are far from the source of pollution with the flow of water.

Bangladesh government has taken initiatives to reduce pollution and to increase navigation facility in dry season in the Buriganga through different projects. Flow of water would increase in dry season and the tannery industries are moving from Hazaribagh to Saver through these projects. However, this study shows that heavy metal pollution continues in post rainy season too with the increase of water flow. Moreover, the metals are easily moving with flow from upstream to downstream throughout the river and would move in to the other rivers as all rivers are interlinked in Dhaka. Therefore, it is mandatory to stop disposal of toxic metal containing wastes to rivers to protect Dhaka from pollutions otherwise any project wouldn't be successful. The enforcement of laws is very urgent to protect riverine ecosystem of Dhaka and should be achieved at any cost otherwise the situation in near future might be further worsened with the continuing development of industries.

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NUMERICAL MODELLING TO ADDRESS RIVER INSTABILITY PROBLEMS AT AN UNDER-CONSTRUCTION BRIDGE SIDE: A CASE STUDY

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Abstract

A hydro-morphological study using numerical modelling techniques was carried out at River Research Institute (RRI) for completion of Wazed Miah Bridge on Karatoya river at 27th km of Sadullapur (Madargonj)-Pirgonj-Nawabgonj Road. The study results show that there is lateral instability problem at the under-construction bridge location and there is a possibility of outflanking of the bridge by the river. Chute cut-off may happen in the bend immediately upstream of the bridge causing large morphological developments at the bridge location. The right bank upstream of the bridge may experience rapid erosion after chute cut-off in the upstream bend which could be a threat for the existence of the right approach embankment. The design discharge and design flood level for bridge and river training works are 2081 m^3/s, 26.97 mPWD and 2368 m^3/s, 27.21 mPWD respectively. From long-term consideration, bank revetment will be needed on both banks at the bridge location to arrest bank erosion. From consideration of river instability situations at the bridge location and long-term perspective, the minimum length of the bridge (abutment to abutment distance) should be 290m which could be adopted by increasing length of the rightmost span whereas the present bridge length is 278.8m.

Keywords: *Bathymetry, bridge, discharge, hydro-morphology, instability, numerical modelling, simulation and water level.*

Introduction

Sadullahpur-Pirgonj-Nawabgonj Road is an important zila road which connects Sadullahpur upazila of Gaibandha district and Pirgonj upazila of Rangpur district with Nawabgonj upazila of Dinajpur district. This zila road is also connected with Dhaka - Utholia - Paturia-Natakhola - Bogra - Rangpur - Bangabandhu National Highway N5. However, this important zila road link is interrupted by non-existence of a roadway bridge over the Karatoya river at 27th km (Katchdaha ghat). In order to facilitate smooth inter-district road communication, Roads and Highways Department (RHD) undertook a project for construction of a bridge over the Karatoya river. Construction of pre-stressed girder bridge was started in 1999 and up to 2011 about 80 percent construction of bridge substructure (piers and abutment) has been completed.

The bridge as designed is 278.885m in length with 7 (seven) spans. The length of six out of seven spans is 42.68m each. The length of the remaining rightmost span is 18.3m. The detailed design and drawing of the bridge have been prepared with due consideration of soil investigation report, navigation clearance and other relevant matters. The bridge site has been selected at an apparently straight reach and over a period of 8 (eight) years since the commencement of bridge construction there has been no significant change in the position of main stream at bridge location. However, since 2009, considerable left bank erosion has taken place in the vicinity of the bridge to Gobindaganj. The Karatoya river at the Wazed Miah bridge location has started to shift towards the Pirgonj end and moved considerably during the last few years. By 2011, the river has shifted beyond the planned location of left abutment. The river pattern is meandering and the river follows the valley slope. There are scars of abandoned former meander loops. It appears that chute cutoff occurs at a relatively low value of cutoff ratio. The average sinuosity of the river in the study reach (18 km upstream and 12 km downstream of the bridge) is about 1.26. The average sinuosity of the river from Badarganj to bridge site is 1.41 and that from bridge site to Gobindaganj is 1.33. It means sinuosity of the river in the study reach is less than the river as a whole from Badarganj.It is, therefore, clear that the apparent stability in the bridge reach as is envisaged during the planning process has already gotten affected by bend migration. Under this circumstance, revision of the original planning of the bridge is needed to come up with concrete decisions for successful completion of bridge construction with appropriate bank protection measures, if necessary.

The study aims to evaluate the hydraulic design parameters and to provide protective measures for

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mitigating the river bank erosion and river bed scour around the piers including the river training works for the proposed bridge.

Setup of 2D Model

The two-dimensional model covers around 26.5 km stretch of the Jamuneswari-Karatoya. The upstream limit is around 15.5 km upstream of the Wazed Miah Bridge and the downstream limit is around 11km downstream of the bridge. The model setup consists of the following steps in chronological order below:

Generation of computational curvilinear grid

In view of different objectives to be met 3 (three) computational grids having different resolutions are generated to study different aspects of the bridge project within the stipulated time. These grids are used to study flow hydrodynamics and river morphology. The curvilinear computational grid for hydrodynamic calibration of the model has a dimension of 507m×21 m. It means the length and width of the study reach are represented in the model with 507 and 21 grid points respectively. The grid for morphological calibration of the model has a dimension of 750m×60m, which is fine enough with respect to hydrodynamic grid for reproduction of morphological changes in the river with acceptable accuracy. The hydrodynamic and morphological calibration are done without bridge and proposed river training works. After hydrodynamic and morphological calibration of the model a third grid (1200m×90m) has been generated considering incorporation of bridge and proposed structures. For planning and design of necessary river training works, the model with third computational grid has been applied.

Generation of boundary conditions

Preparation of bathymetry

After completion of the bathymetric survey the data is processed, the initial bathymetry is then prepared using standard MIKE21C bathymetry preparation module. Suitable interpolation procedure is followed to generate bathymetry information at locations where bed level information is unknown. The generated bathymetry is then checked for consistency. The initial bathymetry corresponding to the grids for hydrodynamic, morphological and hydraulic structures simulation is shown in **Fig. 1**. The rated discharge time series at Badargonj have been directly applied as upstream boundaries of the model. The influence of inflow to and outflow from the river in between Badargonj and model upstream boundary on discharge time series at the model upstream boundary is thought to be not much. The water level time series at the model downstream boundary have been determined by slope analysis. Slope information is obtained from recorded water levels at Badargonj and Siraj, bed slope in the model domain and model generated water level slopes for different discharges. Attempts have been made to calibrate the model using water level data at Siraj. However, no discharge data is available for the years in which recorded water levels at Siraj appear to be acceptable. It is decided to calibrate the model for 2010 event. Since the recorded water levels at Siraj during this event are found incorrect, the water level time series at Siraj have been determined from recorded water levels at Badargonj by slope analysis.

Establishment of initial condition

The initial condition of the model is initial surface elevation. It is selected judiciously so that the model attains steady state condition quickly.

Initial assessment of model input parameters

The input parameters of the model are described as follows:

Time Step: There are a number of time steps that have to be selected properly. The time steps include hydrodynamic time step, morphological time step and advection-dispersion time step.

Time steps are estimated based on standard formula. Little adjustment in the estimated time steps if required is done based on previous experience.

Bed Resistance: Bed resistance can be specified either as a constant value or as a map. It will be decided after observing the degree of variation in initial bed configuration. Chezy resistance factor is used to specify bed resistance.

Eddy Viscosity: Eddy viscosity is estimated using the relevant formula. It can be specified as a map or constant value considering its requiring. Flooding and Drying: Flooding and drying depths are specified in such a way that it causes minimum loss of computational points but prevent occurrence of instability during simulation.

Helical Flow: Before solving the advection-dispersion equation for the concentration of the suspended sediment, the advection-dispersion equation for the helical flow must be solved. The standard formulation is used for the helical flow.

Sediment Grain Size: The sediment grain size is set from the sediment sample survey. In case of variation in sediment size along the model domain a grain size map can be specified. Otherwise, a global value may be used.

Sediment Transport Predictor: The Van Rijn model is used for both bed and suspended load. There is sediment transport and sediment concentration data. Equilibrium concentration concept is, therefore, adopted.

Model Calibration

The model has been calibrated hydrodynamically first using discharge and water level boundaries for 2010 event. The event 2010 is selected because it almost corresponds to the recorded 2.33-year or average discharge and it has happened in the recent past. The hydrodynamic calibration is made in base condition (without bridge). Siraj is the water level gauge station within the model domain and water level time series obtained at Siraj is used for calibration of the model hydro-dynamically. During hydrodynamic calibration default values of flooding and drying depth are specified. It is anticipated that it will cause minimum loss of computational points. Different values of eddy viscosity have been tested and finally a map is prepared. Bed resistance in the model has been specified by Chezy number. A map for Chezy number is specified first in order to have a depth map. Afterwards based on that depth map, local variations in the Chezy number have been specified using a map of resistance number. Model generated water levels at Siraj have been compared with obtained ones at the same location in order to confirm that the model is hydrodynamically calibrated. It can be seen from **Fig. 2** that reasonable agreement is achieved between hydrodynamically simulated and obtained water levels at Siraj for monsoon 2010 data. After hydrodynamic calibration, the model is also morphologically calibrated in base condition and for the same event (2010) as is done for hydrodynamic calibration. Model generated water levels at Siraj are compared with obtained data to ensure that the model is morphologically calibrated. **Fig. 2** shows that reasonable agreement is achieved between morphologically simulated and obtained water levels at Siraj for monsoon 2010.

Assessment of River Training Work (RTW) Options

Since only revetment type structure has been selected for stabilizing the river bank at the bridge location options have been devised on the basis of dimensions of the structures. The main objectives of the option simulations are to arrive at a fair decision regarding optimum dimensions of the structures. Although at present the left bank at the bridge location is experiencing erosion problem revetment type protection measures have been considered for both bank at the location in consideration of long-term safety of the bridge. The devised RTW options are shown in **Fig. 3.** There arise constraints in introducing RTWs in the model to their planned dimensions due to varying sizes of grid cells in the model. The introduced dimensions of the RTWs under three different options appear in **Table 1** below

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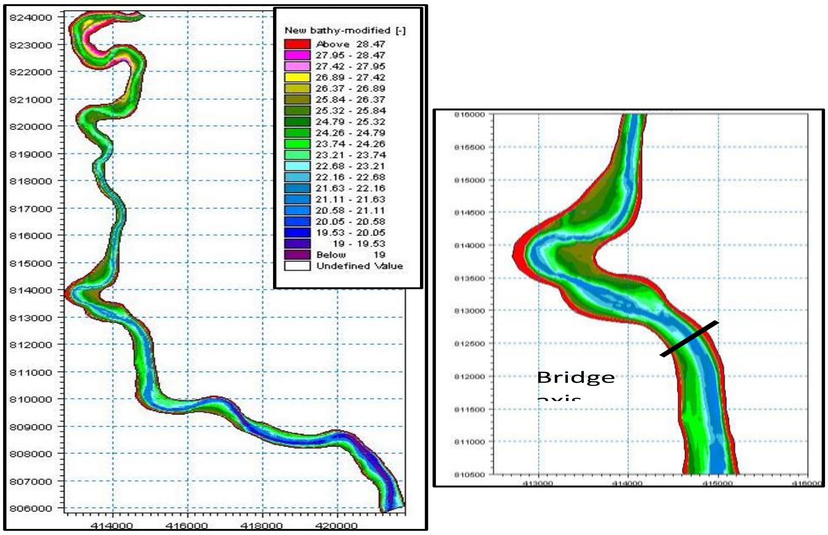

Fig. 1. Initial bathymetry of the model

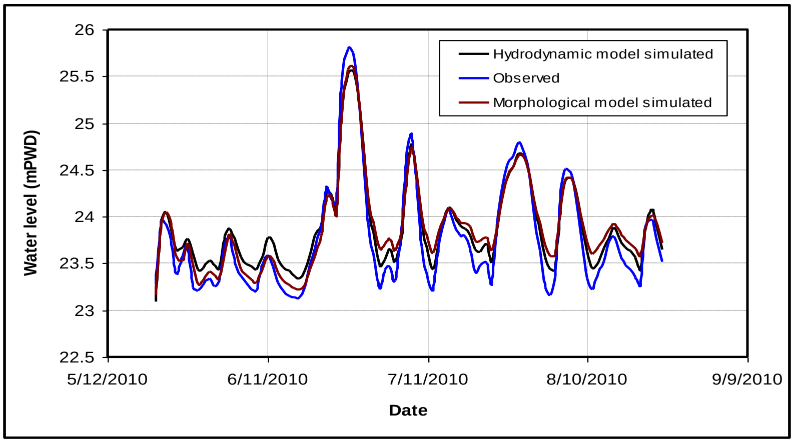

Fig. 2. Comparison between model simulated and observed water levels at Siraj for the year 2010.

Table 1. Dimensions of the RTWs (revetment) under three different options

Options / Structure	Total u/s distance	U/S straight distance	Total d/s distance	D/S straight distance
1/Left bank	80	61	30	23
1/Right bank	46	33	18	13
2/Left bank	101	75	36	29
2/Right bank	63	44	20	14
3/Left bank	160	111	43	34
3/Right bank	87	64	27	19

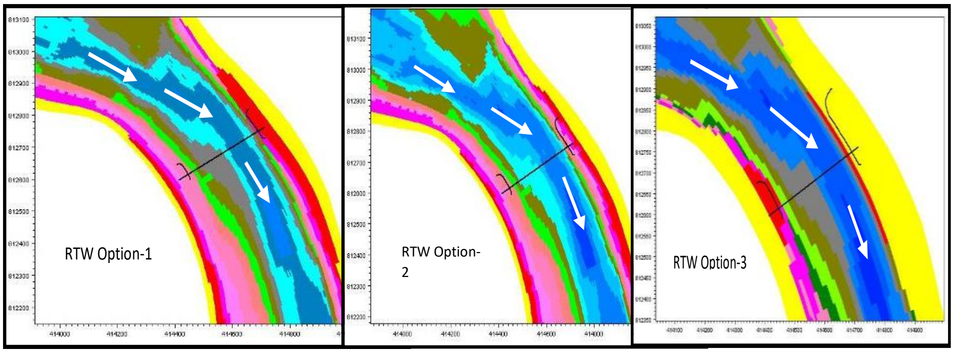

Fig. 3. Structural arrangement of river training works under Option-1, 2 & 3.

It is to be noted here that the u/s and d/s distances in **Table 1** indicate distances upstream and downstream from the bridge axis respectively. In all options the angle of sweep of curved upstream head is 90^0 and curved tail is 60^0.Option simulations with developed model have been conducted both hydrodynamically and morphologically. Since design discharge for river training works is considered as 100 year discharge the simulations have been made for 100-year discharge and 100-year event respectively. During option simulations the bridge piers have been introduced in the model to observe their effects on flow hydrodynamics and river morphology. Bank erosion model has been included in the morphological simulations to observe the effects of RTWs on bank erosion upstream and downstream of the bridge.

Results and discussion

It is observed from the model results that a chute cut-off **(Fig. 4)** may occur at the river bend immediately upstream of the under-construction bridge and it will have large bearing on the future morphological developments at the bridge location in terms of bank erosion. In short-term the present eroding left bank at the bridge location and right bank in the upstream of the bridge may experience accelerated erosion. In long-term the unabated right bank erosion in the upstream of the bridge may threaten the existence of right approach embankment.

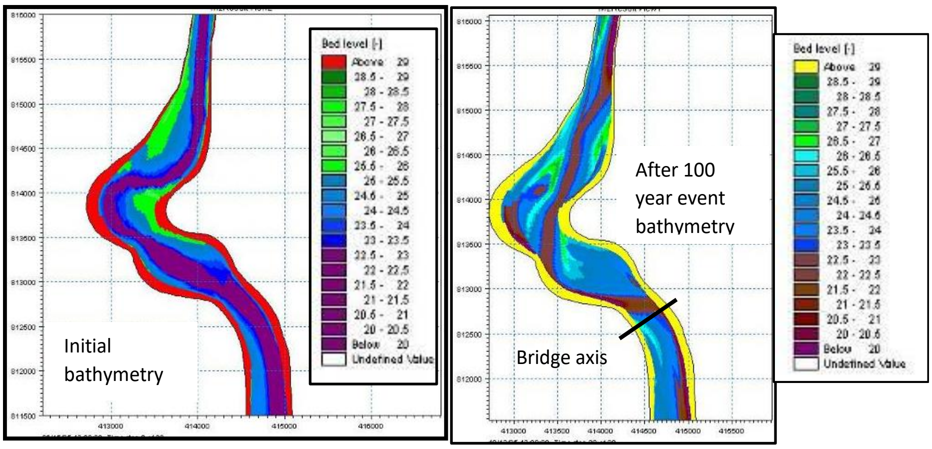

Fig. 4. Bathymetric planform development in the vicinity of the bridge under 100 years event.

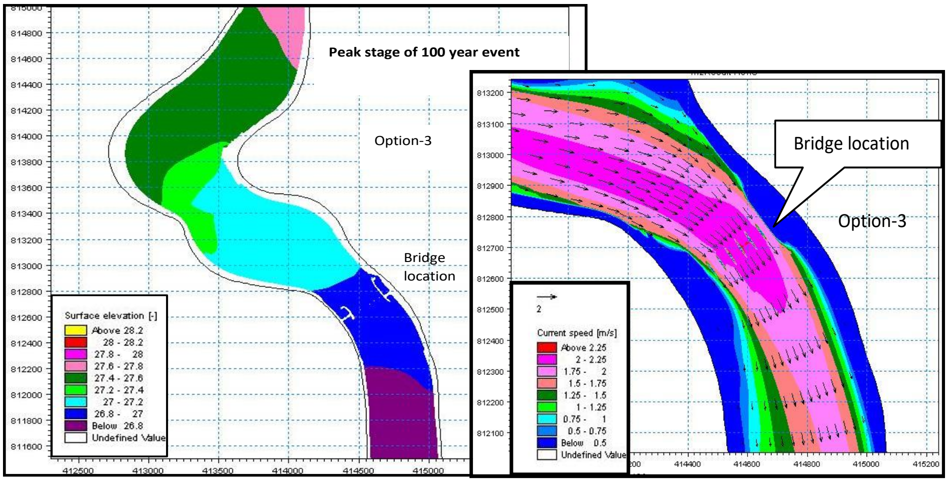

Fig. 5. Water level and velocity field at the vicinity of bridge (Opt-3) for 100 years discharge

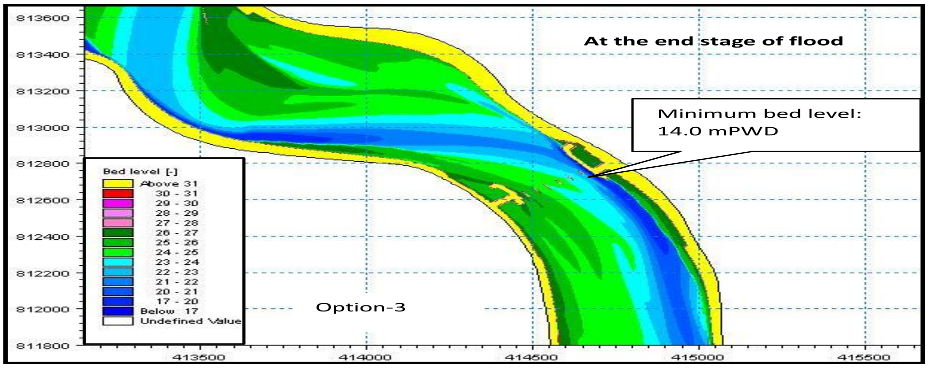

Fig. 6. Morphological conditions at the bridge location (Opt-3) after 100s year event

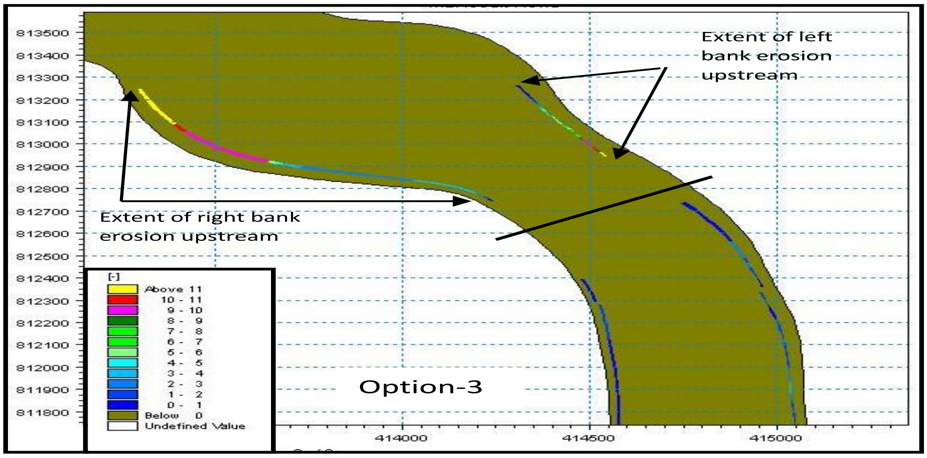

Fig. 7. Bank erosion u/s and d/s of bridge (Opt-3) for 100 years event

Given the present physical conditions of the river at the bridge location only revetment type structures will be appropriate for containing the eroding bank within the bridge limit. River responses to three different dimensions of bank revetment along both banks of the river at the bridge location have been investigated for an extreme event (100 year). The model investigation results show that there could be large scale morphological changes at the bridge location in the upcoming years. The immediacy of such developments, however, depends on the magnitude of flood events that may occur in the coming years. The river may abandon the bend in the immediate upstream of the bridge location by chute cut-off. It will result in rapid migration of the bend at the bridge location. In order to ensure safety of the under construction bridge the migrating left bank should be contained by adopting river training works.

Among the three tested options, Option-1 and Option-2 appear not to be technically feasible as the dimension of the left bank revetment is not enough to provide armour to entire length of eroding bank. Planned left bank revetment under Option-3, on the other hand, could stop caving of bank upstream and downstream of the same beyond certain limit and thus, would reduce the risk of failure of the structure against an extreme event. It is also revealed from the study that there is no risk of right bank erosion at the bridge location in short-term. However, after a chute cut-off in the upstream bend rapid right bank erosion may take place in the upstream of the bridge. The developments there should, therefore, be closely monitored. In order to ensure long-term safety of the bridge against such likely developments bank revetment is also needed along the right bank at the bridge location. Model simulation results indicate that the dimension, orientation and placement of revetment as in Option-3 may be appropriate to contain the river under the bridge.

The flow field at the bridge location under Optoion-3 for 100-year discharge is shown in **Fig. 5** shows the water level in the vicinity of the bridge at peak discharge of 100-year event. The likely morphological developments at the bridge location under Option-3 for 100-year event have been shown in **Fig. 6.** The extents of bank erosion upstream and downstream of the left and right bank revetment under Option-3 are shown in **Fig. 7.**

Conclusion

- Analysis of satellite images and recent cross-section data and model results show that there is lateral stability problem at the under construction Wazed Miah bridge location.
- Left bank erosion at the bridge location could vary from 0m to 16m in a year.
- Model simulated minimum bed level along the left bank revetment is 14.0 mPWD and it occurs near the downstream termination of the same.
- Chute cut-off may occur in the bend immediately upstream of the bridge. After chute cut-off the right bank may experience rapid erosion.
- The river appears to be vertically stable at the bridge location. The foundation levels of the bridge substructure are sufficiently below the expected maximum scour level.
- The bridge is, therefore, safe against bridge scour.
- The vertical and horizontal clearance of the bridge appear to be appropriate. The width of waterway for the bridge could vary from 207m to 290m depending on various factors.
- The planned length of the bridge including the width of the piers (278.885m) is, therefore, somewhat below the upper limit of the width of waterway for the bridge.
- The design discharge for bridge and river training works are 2081 m^3/s and 2368 m^3/s respectively.
- The design flood level for bridge and river training works are 26.97 mPWD and 27.21 mPWD respectively.
- Bank revetment is the appropriate type of protection measure for the eroding bank.
- Both the left and right bank revetment should be provided with suitable upstream and downstream terminations as per standard practices and the dimension, placement and orientation of the suggested measures may be refined by model application.

Recommendation

The river training measures suggested under this study should be adopted as per design. Stabilization of the eroding left bank should be done on priority basis. In consideration of river instability situations at the bridge location and from long-term perspective the minimum length of the bridge (abutment to abutment distance) should be 290m. The present length of the bridge is 278.8m. The minimum length of the bridge could be adopted by increasing length of the rightmost span. The developments in the river channel upstream of the bridge particularly at the bend locations should be monitored very closely. In case of any delay in implementation of the suggested river training measures, temporary measures should be considered to prevent the bridge from being outflanked by bank erosion

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Acknowledgement

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IRRIGATION REQUIREMENTS ASSESSMENT IN A SELECTED IRRIGATION UNIT

A. Akter *

Abstract

Better understanding on water management in any irrigation project plays vital role in incremental crop yield. The Karnafuli Irrigation Project (KIP) in the south-eastern region of Bangladesh is a flood control, drainage and irrigation project. The project is well known for its success in achieving irrigation, except few crop damages due to flooding and water logging. Among two units of this project, CROPWAT 8.0 has applied on the Halda unit by using 15 years (1993 to 2007) climate data, considering two scenarios, i.e., rainfed and irrigation. The aim of this study is to estimate total irrigation water requirements (IWR) using CROPWAT. For total IWR estimation, CROPWAT could reasonably represent the soil moist deficit under rainfed and irrigation scenarios. A water balance calculation was carried out to estimate the surplus water, and hence CROPWAT is found to be a useful tool in estimating irrigation requirement against flooding condition. This study is being expected to contribute to the decision process on optimal irrigation supply.

Keywords: *Irrigation Water Requirements (IWR), CROPWAT, Halda, Evapotranspiration (ETo), Karnafuli Irrigation Project (KIP).*

Introduction

The Food and Agriculture Organization (FAO) introduced CROPWAT as a numerical model to calculate the crop water requirements and irrigation water requirements (IWR) from climate, crop and soil data. This model also allows for developing irrigation schedules for different management conditions and the calculations of water supply scheme for varying crop patterns. The CROPWAT model comprises of (i) Penman-Montieth method for calculating evapotranspiration (ETo) (Smith, 1992), and (ii) the calculation of crop water requirements and IWR requirements are mainly based on FAO irrigation and drainage papers, are (a) Guidelines for computing crop water requirements and (b) Yield response to water (Doorenbos and Kassam, 1979.).

CROPWAT have been using by researchers for estimating the evapotranspiration ETo (Shakoor et al., 2006; Najafi, 2007), crop water requirements (Döll and Siebert, 2002; Shakoor et al., 2006; Cazanescu et al., 2009), water deficit (Severini and Cortignani, 2008) and soil moisture deficit (Roy et al., 2009). CROPWAT seemed promising in predicting the crop water requirements under water stress (Cavero et al., 2000; Marica et al., 2001;Hassanli et al., 2008; Nazeer, 2009) and climate change (Moussa and Amadou, 2006; Nazeer, 2009; Roy et al., 2009). Roy et al. (2009) estimated the soil moisture deficit using CROPWAT in selected parts of Bangladesh over the projected climate scenarios for the years 2030 and 2075 along with the crop water requirement assessment and yield variations. So, there is a knowledge gap on applicability of this model to estimate the excess water causing water logging or flood in irrigated area.

Study site

The project area under Halda unit of KIP is 15386 ha and the irrigable area is 12550 ha, located in 22°25'-22°35' latitude and 91°45'-91°60' longitude (Figure 1). Miah (1986) observed this area comprises of three main seasons, namely, monsoon (June-October), dry season (November-February) and pre-monsoon (March-May). About 80% of the total annual rainfall recorded during monsoon, dry season comprises of lower rain whereas the pre-monsoon comes with occasional heavy rain storms (Miah, 1986). Top soil textures of this area range from loam to clay, with silty clay loam and silty clay are the most common (JCHW, 1968).

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The KIP is supervised by three agencies, the Bangladesh Water development Board (BWDB), Department of Agricultural Extension (DAE) and Bangladesh Rural Development Board (BRDB). The BWDB is responsible for the hydraulic and river training structure operation and maintenance, DAE works for people motivation and participation for High Yield Variety (HYV) crop cultivation and BRDB works for capacity build up through formation of farmers' cooperatives (KSS) and micro credit.

The KIP project implementation period was 1975-76 to 1982-83. Before starting the project a feasibility study was carried out in 1974 (IECO, 1974) and since 1982, Kaptai Operation and Maintenance Division, BWDB observing the achieved irrigated area over the target (Figure 2). Qasem (1984) reported during dry season around 800 low lift pumps were employed for High Yield Variety (HYV) rice cultivation using surface water. Miah (1986) described that per 0.05 cumec pump covers around 17 ha command area. Around 1214 ha of higher land was irrigated by ground water using shallow/ deep tubewells during dry season and about 10117 ha of low lying land near the Halda river tributaries used to irrigated by surface water (Miah, 1986).

Due to flood and water logging two major crops (i.e., Aman and Boro rice) under this project area often damaged (Miah and Mohit, 1996). So, proper management on the crop water requirements and irrigation supply need to be better understood. The study reported here addresses this through CROPWAT estimated surplus irrigation water. The available survey data in KIP shows in 1992 about 90% of the target irrigated area have achieved, so for calculation it is assumed that the 100% of target area (i.e. 12550 ha) had achieved in 1993 and onward.

Theory

Water balance studies play a vital role in IWR calculation as well as in the modern hydrology. The inflow to the field comprises of the total precipitation and irrigation, where water moves from the field through ETo, seepage, percolation and surface runoff. A generalized water balance equation for an agricultural field can be expressed as follows:

$$S(t+\Delta t) = S(t) + P(t,t+\Delta t) - ET_o(t,t+\Delta t) - R(t,t+\Delta t) - I(t,t+\Delta t) - S_1(t,t+\Delta t) + IR\,(t,t+\Delta t) \quad \ldots(1)$$

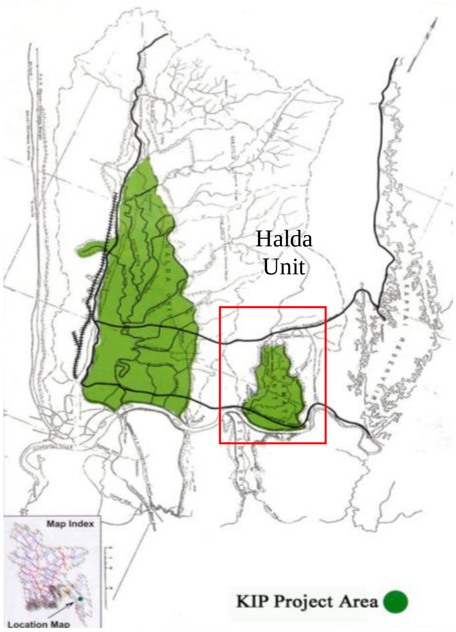

Figure 1. Halda unit in KIP project

Where,

$S(t+\Delta t)$	=	Soil moisture content at time $t+\Delta t$, mm
S(t)	=	Soil moisture content at time t, mm
$P(t+\Delta t)$	=	Precipitation of all forms between time t and $t+\Delta t$, mm
$ET_o(t+\Delta t)$	=	Reference evapotranspiration between time t and $t+\Delta t$, mm
$R(t+\Delta t)$	=	Runoff between time t and $t+\Delta t$, mm
$I(t+\Delta t)$	=	Percolation loss to groundwater between time t and $t+\Delta t$, mm

$S_1(t+\Delta t)$	=	Seepage loss between time t and $t+\Delta t$, mm
$IR(t+\Delta t)$	=	Irrigation supplied between time t and $t+\Delta t$, mm.

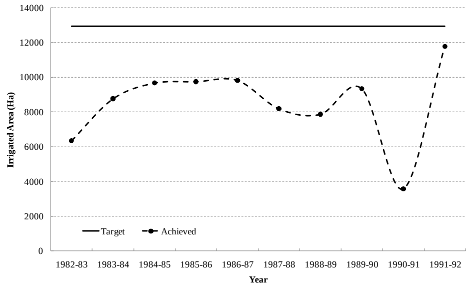

Figure 2. The target and achievement of irrigation area in Halda unit, KIP (Kaptai O &M Division, BWDB)

Precipitation provides part of the required water for crops, need to satisfy their transpiration requirements. The soil acts as buffer to store parts of the precipitated water and supply to the crops under water stress. In humid condition, the whole mechanisms ensure satisfactory growth under rainfed agriculture. In dry condition irrigation needs to meet the deficiency due to evaporation and precipitation. The irrigation water requirements were calculated into the CROPWAT model by the difference between the crop evapotranspiration under standard conditions (ETc) and the effective rainfall contributions over the same time step, i.e.,

$$\text{IWR} = A\sum_{i=1}^{365}\left(ET_O \times K_c - P_{eff}\right) \qquad \ldots(2)$$

Where,

IWR	=	Irrigation water requirement (m^3/year)
ET_o	=	Reference evapotranspiration (mm/day)
K_c	=	Crop coefficient
P_{eff}	=	Effective precipitation (mm/day)
A	=	Irrigated area (percentage of total area)

Net irrigation requirements are therefore defined as the volume of water needed to compensate for the deficiency between the potential evapotranspiration and the effective precipitation over the crop growing period (Faurès et al, 2002). So, the IWR estimation was done using Equation 2 i.e. the standard CROPWAT calculation method. However, to evaluate the CROPWAT performance, the water balance has computed for irrigated lands in the study area using Equation 1.

Evapotranspiration (ETO) is defined as the consumptive use for a particular crop during transpiration and the evaporation either from the plant itself or from the adjacent soils (Gangopadhyaya et al., 1966, Garg, 1998). So, the precipitation, soil moisture condition, specific plan water requirement and the physical nature of the land cover or the studied watershed hydrology are the local factors, influence the evapotranspiration (Dunn and Makay, 1995). Generally, the irrigation requirements of the crops are calculated from the difference between the consumptive use and the effective precipitation (Garg, 1998). The well known methods for estimating ETo are the Penman (Penman, 1948), Penman-Monteith (Monteith, 1965 and 1981), FAO Penman–Monteith (FAO-PM) method, Pan Evaporation, Kimberly-Penman (Jensen et al., 1990), Priestley–Taylor (Priestley and Taylor, 1972), Hargreaves (Salazar et al., 1984), Hargreaves class A pan evaporation method (Garg, 1998), Samani-Hargreaves (Samani and Hargreaves, 1985) and Blaney–Criddle (Allen et al., 1998). Intensive studies carried out by different researchers for a suitable ETo estimating method in irrigation project context and the Penmen-Monteith model was reported to be the best suited model (Lee et al., 2004). ETo in CROPWAT is computed by FAO Penman-Monteith Model (Allen et al., 1998). The FAO Penman-Monteith method to estimate ETo is:

$$ET_o = \frac{0.408\,\Delta(R_n - G) + \gamma\dfrac{900}{T+273}\,u_2(e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)} \qquad (3)$$

Where,

R_n	=	Net radiation at the crop surface (MJ/m^2 day)
G	=	Soil heat flux density (MJ/m^2 day)
T	=	Air temperature at 2 m height (°C)
u_2	=	Wind speed at 2 m height (m/s^1)
e_s	=	Saturation vapour pressure (kPa)
e_a	=	Actual vapour pressure (kPa)
e_s - e_a	=	Saturation vapour pressure deficit (kPa)
γ	=	Psychometric constant (kPa/°C)
Δ	=	Slope vapour pressure temperature curve (kPa/°C) (ASCE, 2002)

$$= \frac{2504\, exp\, exp\left(\frac{17.27\, T}{T + 237.3}\right)}{(T + 237.3)^2}$$

Materials and Methods

Details on the collected climate data are given in Table 1. Generally, for a normal year the recorded precipitation in the studied station is 2900 mm and during 1993-2007 the driest most year was 1994 with 2258 mm precipitation. In CROPWAT, to account for the losses due to runoff or percolation, the effective precipitation (P_{eff}) can be estimated by four methods, namely Fixed percentage, Dependable rain, Empirical formula, and USDA Soil Conservation Service (FAO, 2010). The USDA Soil Conservation Service (SCS) described crops can use almost 60 to 80% of the total monthly precipitation (P_{month}) upto 250 mm and beyond this amount of precipitation the crop consumption eventually become less (USDA, 1993).Thus the effective rainfall (P_{eff}) is:

$$P_{eff} = \frac{P_{month} \times (125 - 0.2 \times P_{month})}{125} \quad (6)$$

for , $P_{month} <= 250\ mm$

$$P_{eff} = 0.1 \times P_{month} \quad (7)$$

for, $P_{month} > 250\ mm$

In last decade, during June to August the recorded amount of precipitation in the study area often become two to three times more than the USDA prescribed amount. So, to understand the general performance of the existing irrigation project both of the USDA and the fixed percentage (80% efficiency) rainfall/precipitation method were tested. As this study aimed to investigate optimal irrigation requirements against the water logging and flooding, so the fixed rainfall has chosen as higher possible value. One limitation in rainfall data entry in CROPWAT 8.0 is the present setup can only accept three digits plus a decimal for a given month. In July 1997, July 1998 and August 1998 the recorded rainfall data were 1033 mm, 1291 mm and 1194 mm respectively; the input were given as 999.9 mm in CROPWAT. So, the underestimation was 3% for 1997 and 22% for 1998.

The details on crop and cropping pattern input data tabulated in Table 2 and the percentage of cultivated area noted down in Table 3. Miah and Mohit (1996) observed that the available irrigation water had positive influence on the crop selection and cropping pattern in the KIP project area and the copping intensity in the whole project had increased from 181% (in 1981) to 200% (in 1990). The crop selection goes for transplanted Aman and Boro HYV instead of the local variety Aus, Aman and Boro (Miah and Mohit 1996). So, during calculation the available crop types in 1992 has considered throughout the study area reported in this paper. *Soil data* for the study area is tabulated in Table 4.

Irrigation water requirements (IWR)

United States Department of Agriculture (USDA) soil conservation method was used for estimating the actual irrigation requirements for different crops. Two scenarios were tested, i.e., (A) Rainfed and (B) Irrigation. The main differences in these two scenarios were in the growth stage of plant under same land preparing schedule. In scenario A there was no irrigation, where as the scenario B comprise of (i) For rice: irrigation timing at fixed water depth i.e. 5 mm and the refill application at fixed water depth would be 100 mm, and (ii) For Rabi crop: irrigation timing at 100% critical depletion and refill application at soil moisture content to 100% field capacity. The irrigation efficiency was considered as 70%.

Flow data

Fifteen years (1993-2007) monthly flow data were collected for the station Punchpukuria

(Longitude: 91° 46' E, Latitude: 22° 40' N) from BWDB.

Results and discussion

ETo Calculation

Except the minimum and maximum temperature there were discontinuous data series for other parameters in the meteorological station, so the relevant option in CROPWAT had used for estimating the missing parameters and then the ETo. Figure 3 represents the range of monthly ETo variation in last 15 years. The total ETo had calculated from the average monthly values (Table 5). Due to lack of details data on the Aus in literatures, the CROPWAT estimated value could not compare. However, Boro and Aman rice and the Rabi crops seemed well estimated using CROPWAT setup.

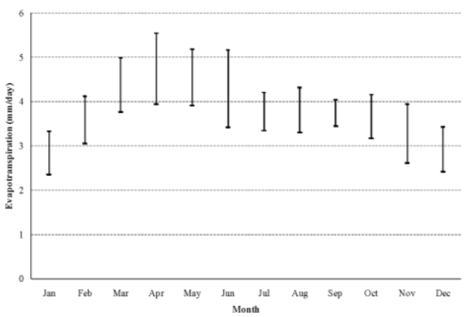

Figure 3. Average monthly CROPWAT estimated ETo in Chittagong station (1993-2007).

Irrigation requirements

The CROPWAT prediction for the *scenario A* showed cropwise yield reduction over last 15 years without irrigation Figure 4. Low rain year, like 2006, needed irrigation for all four category crops. But normally Boro and Rabi need irrigation for enhancing the growth. According to Mahmood (1997) Aman is rain-fed rice, supplement irrigation is required for Aus in initial stage and Boro needs irrigation. Similar conclusion can be made for the scenario A. 40 to 100 mm soil most deficits during harvest period observed under this scenario (Figure 5).

In *scenario B*, Boro and Rabi need some additional irrigation, however with the provided amount of irrigation the CROPWAT prediction having soil moist deficit under 20 mm (Figure 5) and there was no yield reduction. The USDA soil conservation method showed the average effective rainfall for the studied period (1993-2007) for Aus, Aman, Boro and Rabi were 56%, 75%, 54% and 79% respectively. It is assumed that if a fixed rainfall of 80% is available then all of the four crops would be benefited. So, for water balance calculation this setup was continued for both of the USDA soil conservation services and fixed rainfall method.

The water balance had calculated using Equation 1 , in this calculation the total irrigated land had taken as 12950 ha (the target irrigated area) (Data Source: Kaptai O &M Division, BWDB). The CROPWAT estimated effective precipitation 'Peff' excludes the loss due to seepage and percolation from total precipitation, so the 2nd, 5rd and 6th term in Equation 1 represented by Peff. Then, CROPWAT estimated ETo had used. According to Alauddin and Quiggin, (2008) environmental changes might happen when freshwater diversion reaches 25 – 30% of historic seasonal low flows. Feld (1995) noted many of Bangladeshi rivers already exceed these levels and three highly affected rivers are Buriganga near Dhaka, Sitalakhya near Narayanganj and Karnafuli near Chittagong. So, for present study the irrigation supply (IR) was calculated by applying 12.5% as surface water efficiency of input of river flow (station Punchpukuria) in Halda unit. Garg (1998) demonstrated the runoff (R) can be calculated as (Runoff = Runoff coefficient × precipitation), where, the runoff coefficient for the clay and silty loam flat cultivated area is 0.30.

Two conditions, (i) 80% fixed rainfall and (ii) USDA soil conservation services, had evaluated under consideration of 70% irrigation efficiency, 100 mm fixed irrigation for rice, and irrigation for Rabi crop providing soil moisture content refill up to 100% field capacity.

So, from the comparison between the two methods it can be concluded that the selection of 12.5% extraction from river can be further reduce based on more details data from the irrigated land itself.

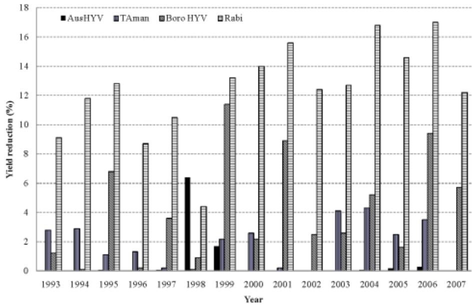

Figure 4. CROPWAT predicted crop yield reduction (Scenario A)

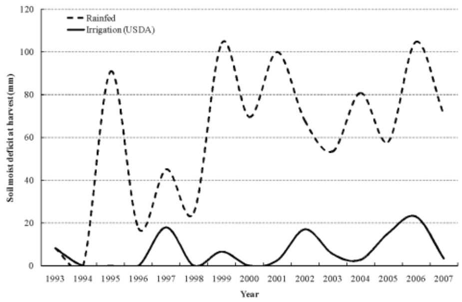

Figure 5. Comparative soil moist deficit under two scenarios

Table 1. Details on climate and precipitation data

Parameters	Duration	Details
Minimum temperature (°C)	1993-2007	Station: Chittagong Source: Bangladesh Meteorological Department Frequency: Monthly data
Maximum temperature (°C)		
Humidity (%)	1995-2001	
Wind (km/day)	1995-2001	
Sunshine (hour)	1997-2001	
Precipitation (mm)	1993-2007	

Table 2. Details on crop input data for CROPWAT

Crop[c]		Growth Stage					Data Source
		Land preparation	Initial	Development	Mid	Late	
Length (Days)	Boro	30	15	65	45	25	Mahmood, 1997 Mondal et al., 2010
	Aman	30	20	50	30	15	Bangladesh Aman rice v.1, www.irri.org/irrc/ssnm
	Aus	30	30	30	60	30	Allen et al. (1998) for rice
	Rabi crop	30	35	50	25	30	Allen et al. (1998) for potato.
Kc wet	Boro	-	1.10	1.20	0.9	0.60	Based on wind speed and relative humidity, the initial, development and late stage value was taken from Allen et al (1998) for rice. Mid stage calculated from the average of the development and late stage.
	Aman	-	1.15	1.20	1.05	0.90	
	Aus	-	1.10	1.20	0.97	0.75	
	Rabi crop	-	-	1.15	0.95	0.75	
Rooting depth (m)	Rice	-	0.15	-		0.5	Yoshida (1981) for rice
	Rabi crop	-	0.4	-		0.75	Battilani et al. (2008)

Puddling depth (m)	0.12	-		-		Saunders and Hettel (1993)
Critical deple-tion	Boro	0.27		0.18	0.28	Based on average evapo-transpiration for relevant months and the crop coefficient, crop evapotranspitarion (ETc) was calculated. Then the critical depletion factor with respect to the ETc had taken from Allen et al (1998).
	Aman	0.2		0.21	0.29	
	T. Aman	0.23		0.21	0.29	
	Aus	0.17		0.16	0.28	
	Rabi crop	0.35		0.27	0.32	
Yield response factor	Boro	1.1	1.75	2.4	0.29	Mondal et al. (2010)
	Aman	1.1	1.1	2.4	0.33	Prasad et al. (2006)
	Aus	1.4	1.4	3.0	0.4	Doorenbos and Kassam (1979)
	Rabi crop	0.45	0.8	0.7	0.2	Allen et al. (1998)
Crop height (m)	Boro			1.1		Pathak et al., 1999
	Aman			1.5		Zubaer et al., 2007
	Aus			1		www.knowledgebank.irri.org/uplandRice/farmersGuideUplandRice.pdf
	Rabi crop			0.6		Allen et al., 1998

[c]CROPWAT model input planting date collected from cropping calendar of Halda unit adjacent upazila (smallest administrative unit of Bangladesh) (BBS, 1985): Aus (Local) - 15April, Aus (HYV) -15 May, Aman (Local) – 15 July, Transplanted Aman (HYV) – 15 August, Boro (Local) -15 January, Boro (HYV) – 15 February, Rabi crop – 15 November.

Table 3. Crop wise cultivated area, Halda unit (Without project, 1974 and with project 1986, Evaluation Studies of KIP by SARM, 1986 and For 1992, Household Survey by DPE, 1992)

Crops		Without Project (1974)		With Project (1986)		With Project (1992)	
		Area (Ha)[d]	Yield (Ton/Ha)	Area (Ha)[d]	Yield (Ton/Ha)	Area (Ha)[d]	Yield (Ton/Ha)
Rice	Aus (Local)	4452 (14%)	1.29	-	-	-	-
	Aus (HYV)	2832 (9%)	2.58	8175 (28%)	2.60	4655 (15%)	2.76
	Aman (Local)	8094 (26%)	1.94	-	-	-	-
	T.Aman (HYV)	6070 (19%)	2.95	10842 (38%)	3.07	13825 (44%)	3.08
	Boro (Local)	809 (3%)	2.30	-	-	-	-
	Boro (HYV)	7285 (23%)	4.24	8652 (30%)	3.57	11780 (38%)	4.15
Rabi	Pulses/ Oilseeds	1376 (4%)	0.74	382 (1%)	1.04	480 (2%)	1.22
	Vegetables	405 (2%)	6.99	672 (3%)	6.94	670 (1%)	7.02

[d] percentage of the total cultivated area had represented in the parenthesis

Table 4. Soil input data

Parameters	Value	Sources
Total Available soil moisture (mm/m)	140	Silty clay composition (sand 20%, silt 30%, clay 50%) was derived for lower Atrai Basin in Bangladesh (Alam et al, 2007). Using these values the water content at field capacity and wilting point had found as 0.43 m^3 m^{-3} and 0.29 m^3 m^{-3} respectively.
Maximum rain infiltration rate (mm/day)	115	For the silty loam texture, the basic infiltration rate of 4.8 mm/h (Ali et al., 2007)
Maximum rooting depth (m)	0.75	Battilani et al. (2008)
Initial Soil moisture deplete (% of TAM)	40	Chowdhury (2009) used 5% and 20% of the total available for drought risk assessment numerical model in major rivers in Bangladesh. Parker (1992) found the amount not more than 40% of the total available water.
Drainable porosity	10	Using hydraulic properties calculator for silty loam http://weather.nmsu.edu
Critical depletion for puddle cracking	0.6	Allen et al. (1998) suggested values are between 0.4 and 0.6, whereas the lower values are for sensitive crops with limited rooting systems under higher evaporation rates, and higher values for deep and densely rooting crops and low evaporation conditions.
Water availability at planting (mm)	31	Samson et al. (2004)
Maximum water depth (mm)	101	Samson et al. (2004)

Table 5. Comparison between two methods for evapotranspiration estimation

Crop		Crop growing duration[e]	CROPWAT Total ET_o (mm)	Total ET_o (mm) from previous studies
Rice	Aus HYV	Apr to Sep (15/04 to 11/09)	650.43	---
	T Aman	Aug to Dec (15/08 to 7/12)	415.7	476 (Bhuiyan and Islam, 1991)
	Boro HYV	Jan to Jun (15/02 to 13/07)	652.86	741 (BRRI, 1985) 760 (Bhuiyan and Islam, 1991) 710 (Mahmood, 1997)
Rabi crop		Nov to Apr (15/11 to 3/04)	472.66	432 (Khan et al. 1981)

[e] details on crop growth with data sources has tabulated in Table 3

Table 6. Actual irrigation (mm) requirement[f] for different crops under scenario **A** using CROPWAT

Year	Aus (HYV)	T Aman	Boro (HYV)	Rabi	Summary
1993	-781.6	-573.6	-438.5	**190.3**	
1994	-687.7	-293.8	-272.5	**247.1**	
1995	-580.4	-654.9	-91.6	**321**	
1996	-715.7	-757	-354	**158.1**	Irrigation is required only for Rabi crop for 1993 to 2007.
1997	-601.9	-703.2	-122.6	**236.4**	
1998	-604.3	-555	-180.9	**100.4**	
1999	-573.5	-672.6	**43.8**	**362**	
2000	-725.5	-708.4	-172.6	**346.6**	
2001	-478.6	-450.4	-74.1	**413.8**	Additional irrigation is needed for Boro rice for the year 1999 and 2005.
2002	-703.2	-574.7	-174.5	**365.2**	
2003	-515.2	-300.9	-171	**290.5**	
2004	-514.9	-449.2	-269.3	**378**	
2005	-464.7	-683.4	**23.4**	**327.2**	
2006	-374.6	-317.9	-77.3	**431.9**	
2007	-658.2	-959.6	-241.6	**319.1**	

[f]negative sign represents excess water

Conclusion

The Halda unit once suffered with severe flash flood and in dry season it failed to achieve crop production. The Karnafuli Irrigation Project (KIP) (implementation period 1975-76 to 1982-83) brought significance change in this area with dual purpose pumping facility, i.e. use for irrigation in dry season and usage for drainage when it requires. With 15 years climate data, this study aimed to assess the CROPWAT model's applicability in excess water estimation. Most of the previous study using CROPWAT focused on irrigation requirements in water scarcity situation. Although the flooding affinity in Halda unit had minimized by the existing project, as this area prone to flooding and water logging improved prediction of excess water would be strengthen the project management.

In this study, CROPWAT showed reasonable values in ET_o computation. To predict the irrigation requirement, rainfall plays crucial role, in this case, the two years 1997 and 1998 were having reasonable rainfall throughout the year, so the under estimation didn't make noticeable difference. However, for enhancing use of this model there is an urgent need to improve the monthly rainfall data entry facility in CROPWAT. With the limited soil property data, the prediction gives sensible values however for intensive study more details are needed. On the other hand DAE already had done progress with the crop selection sector. So, the enriched knowledge on soil fertility and crop management integration with this prediction process for irrigation requirement can offer the optimal decision**.**

Acknowledgement

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ASSESSMENT OF BANK LINE SHIFTING OF SURMA RIVER USING GIS AND REMOTE SENSING APPROACH

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Abstract

River and Drainage System the rivers of Bangladesh are very extensive and distinguish both the physiography of the country and the life of the people. Bangladesh is called a land of rivers as it has about 700 rivers including tributaries. This study has been conducted to assess the bank line shifting of Surma River using Remote Sensing and GIS approach in light of the recent outbreak of erosion in many rivers in Bangladesh. Landsat Satellite images of 1994, 2006, and 2018 have been collected based on study area. ERDAS IMAGINE 2014 and ARC GIS 10.3.1 have been used to analyze the satellite images. A number of 23 places between Dowarabazar and Golapgonj of Sylhet district have been selected for study area to easily perceive the order of erosion of the river in those corresponding places. Kalaruka, Dowarabazar, Dakshin Kushighat have been found as the most vulnerable places and Haripur, Chatak, Mollapara have been found as the least vulnerable places due to the river bank erosion within our fixed time frame. Also, most erosion has been found at the bends of the river, the fact, which has been also emphasized in this study.

Keywords: Surma River, Remote Sensing, GIS, Landsat Satellite, River Bank Erosion.

Introduction

The Surma-Meghna system' the Meghna is the longest (669 km) river in Bangladesh. It drains one of the heaviest rainfall areas (e.g., about 1,000 cm at Cherapunji in Meghalaya) of the world. The river originates in the hills of Shillong and Meghalaya of India. The main source is the Barak River, which has a considerable catchment area in the ridge and valley terrain of the Naga-Manipur hills bordering Myanmar. The Barak-Meghna has a length of 950 km of which 340 km lie within Bangladesh. On reaching the border with Bangladesh at Amalshid in Sylhet district, the Barak bifurcates to form the steep and highly flashy rivers surma and kushiyara [Rahman, 2010]. River bank erosion is a natural disaster and takes place round the year. Impacts of river bank erosion are multifarious: social, health, education and sometimes political. Numerous studies was done in the past regarding the bank line shifting which generalizes the term "River bank erosion". Like almost all of the rivers in Bangladesh, Surma River has also got no escape from erosion. In this study, this assessment will be done. In this regard, morphological condition of the river from 1994 to 2018 will be observed with the Landsat Satellite images. This will be conducted in three divisions. First, from 1994 to 2006, then 2006 to 2018 and finally entire time period of 1994 to 2018 will be analyzed. ERDAS IMAGINE 2014 will be used mostly throughout the study. Our study area will be selected from the satellite image and after completing some steps in the ERDAS IMAGINE, ARC GIS 10.3.1 will be used ultimately to measure the shifting of the bank line. The erosion of the Surma and Kushiara along the Sylhet border is pushing the Bangladesh border inward, already resulting in the loss of thousands of acres of land to India in last few years. Official sources said more than 3000 acres of Bangladesh territory have already gone to India due to the erosion of the two rivers only. Locals, however, estimate that the loss is no less than 4,000 acres [Rahman, 2010]. Hence, it's very important for the community living on the bank and in the surrounding areas to assess their vulnerability to river erosion and take sustainable steps to mitigate the hazard of river bank erosion. The main objectives of this study is to find out the bankline shifting of the Surma River and analyze the current position of the Bankline and compare with previous year's data. Also have an idea of the risks of the people living adjacent the river and suggest further studies and actions to deal with the problem.

Study Area

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The Surma-Meghna system the Meghna is the longest (669 km) river in Bangladesh. It drains one of the heaviest rainfall areas (e.g., about 1,000 cm at Cherapunji in Meghalaya) of the world. The river originates in the hills of Shillong and Meghalaya of India. The main source is the Barak River, which has a considerable catchment area in the ridge and valley terrain of the Naga-Manipur hills bordering Myanmar. The Barak-Meghna has a length of 950 km of which 340 km lie within Bangladesh. On reaching the border with Bangladesh at Amalshid in Sylhet district, the Barak bifurcates to form the steep and highly flashy rivers Surma and Kushiyara [Islam, 2017]. The Surma, flowing on the north of the Sylhet basin, receives tributaries from the Khasia and Jaintia hills of Shillong. Some of the important tributaries of these two rivers are Luba, Kulia, shari-goyain, Chalti-nadi, Chengar-khal, piyain, Bogapani, Jadhukata, Someshwari and kangsa. The Surma meets the Meghna at Kuliarchar upazila of Kishoreganj district [Mishuk & Islam, 2014].

Methodology

Data Used

To detect the morphological changes of the river, three Landsat satellite images were used:

1. December 13, 1994
2. December 14, 2006 and
3. November 13, 2018.

Software Used

The following software's were used throughout our study:

1. ERDAS IMAGINE 2014
2. ARC GIS 10.3.1

Methodology

Methodology of the work has been shown in the Figure 1

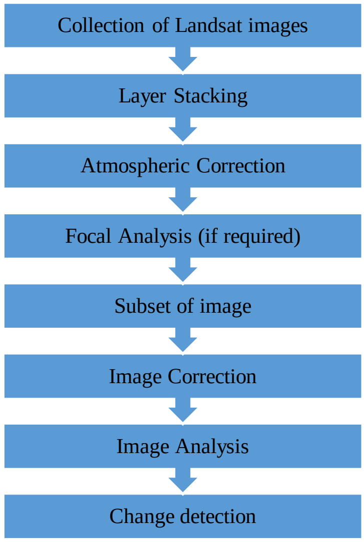

Fig 1: Process flow diagram

Collection of Images

Landsat Images were collected from this website (https://earthexplorer.usgs.gov). Path and Row was assigned for our study area and cloud cover was kept minimum. Landsat 8 image was collected for 2018, Landsat 5 image for 2006 and 1994.

Result and Discussion

The following figures depict the corresponding shifting of bank line from year 1994 to 2018. In this regard, shape file of only river land of year 1994, 2006, 2018 were used. In figure: 24 shape file of year 1994 assigned as red and 2006 as green. In figure: 26 shape file of year 2006 as sky blue and 2018 as brown. In case of graphs, the diagram shows degree of bank line shifting toward right and left. Values above zero indicates shifting of banks toward right and values below zero indicates shifting toward left [Joseph, 2005]. From the analysis Kalaruka has suffered most severe bank erosion. In Chatak area there is no erosion.

Shifting of Bank line from 1994 to 2006

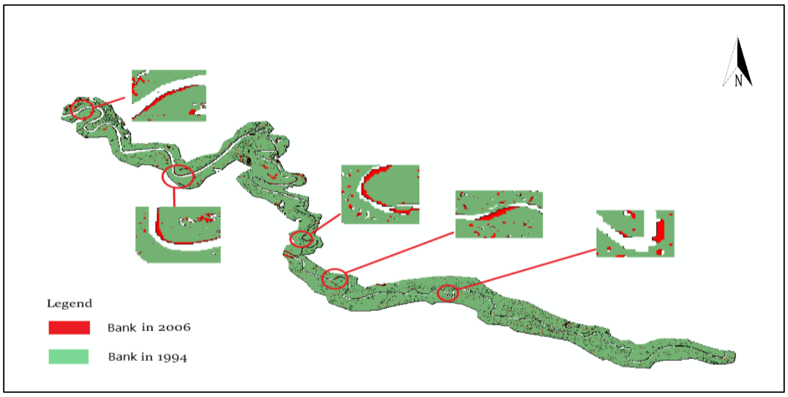

Fig. 2: Severely bank line shifted places (1994-2006)

Merging of two shape file of 1994 and 2006 was done in fig. 2 places were selected to measure the shifting. But of all places, most severely shifted places were identified in this figure. In the following table, names of different location, shifting of right and left bank and direction of shifting are shown. For example at Dowarabazar left bank of the river has shifted 84.85 meters leftward and right bank has shifted 42.42 meters rightward.

Table 1: Data of Bank line shifting in (1996-2006)

Location	Left Bank shift (m)	Direction	Right Bank shift(m)	Direction
	84.85	Right	42.42	Right
Dowarabazar 2	47.43	left	60	left
Haripur 1	81	Right	75	Right
Haripur 2	47.43	Right	41.97	Left
Betura 1	61.17	Right	30.01	Right
Betura 2	63.63	Left	84.85	Left
Chatak 1	No shifting		No shifting	
Chatak 2	60	Right	30	Right
Chatak 3	61.84	left	45	Left
Chatak 4	60	Right	75	Right
Kalaruka 1	54.08	left	67.08	left
Kalaruka 2	30	left	30	left
Kalaruka 3	60.57	left	96.04	left
Kalaruka 4	66.41	Right	30	Right
Kalaruka 5	117.15	Right	127.27	Right
Kalaruka 6	61.85	left	32.39	Right
Lamakazi 1	60.78	left	60.32	left
Lamakazi 2	30.07	left	75	Right
Gopall	90.014	left	30	Right
Mollapara	60	left	63.63	Right
Dakshin Kushighat	30.91	left	63.63	Right
Maijbag	30	left	77.81	Right
Hajipur	37.18	left	42.42	Right

In the figure below the downward gradient means shifting to left and upward gradient means shifting toward right. The numbers indicates the location.

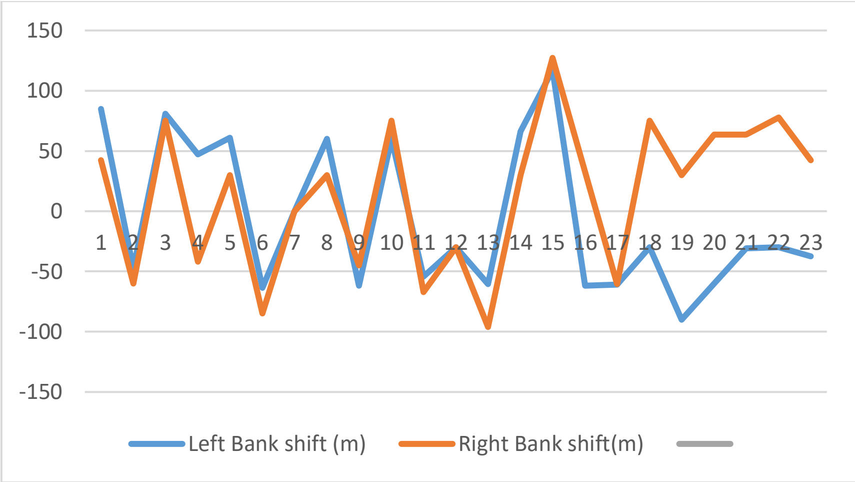

Fig. 3: Bank line shifting of the River (1994-2006)

Shifting of Bank line from 2006 to 2018

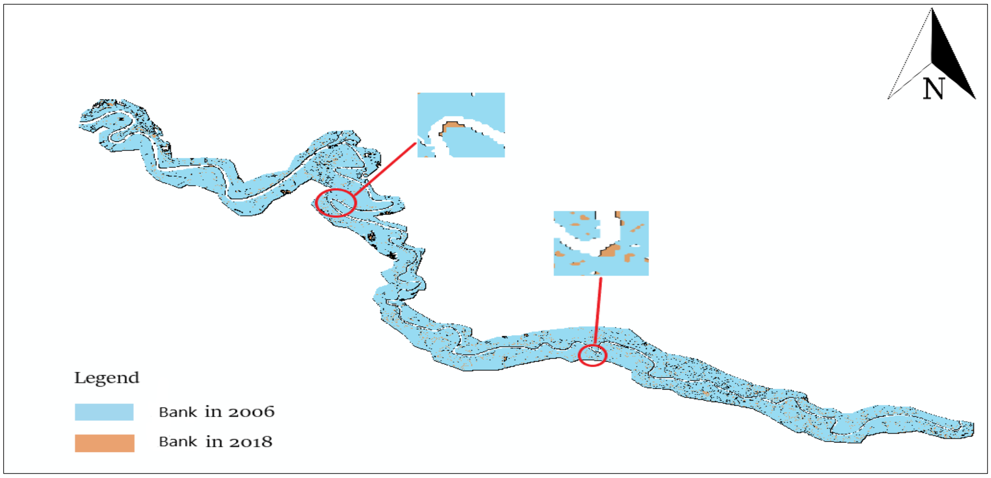

Fig. 4: Severely bank line shifted places (2006-2018)

Table 2: Bankline shifting in (2006-2018)

Location	Left Bank shift(m)	Direction	Right Bank shift (m)	Direction
Dowarabazar 1	31.66	Right	57	Right
Dowarabazar 2	47.43	Right	75	Left
Haripur 1	33.54	Right	30	Left
Haripur 2	30	Left	60.33	Left
Betura 1	60	Right	65	Right
Betura 2	53.87	Left	42.42	Left
Chatak 1	No shifting		54.07	Right
Chatak 2	30.02	Right	30	Right
Chatak 3	60	Left	30.07	Left
Chatak 4	30	Right	60.01	Left
Kalaruka 1	107.39	Left	69.06	Left
Kalaruka 2	30	Right	23.31	Right
Kalaruka 3	30	Right	60	Right
Kalaruka 4	63.63	Right	30	Right
Kalaruka 5	42.42	Right	33.17	Right
Kalaruka 6	30.04	Right	30.018	Left
Lamakazi 1	30.04	Right	45.93	Right
Lamakazi 2	21.21	Right	30	Left
Gopall	84.94	Right	73.76	Right
Mollapara	31.97	Right	31.54	Left
Dakshin Kushighat	26.46	Right	78.79	Right
Maijbag	28.09	Right	30	Left
Hajipur	31.82	Right	30	Left

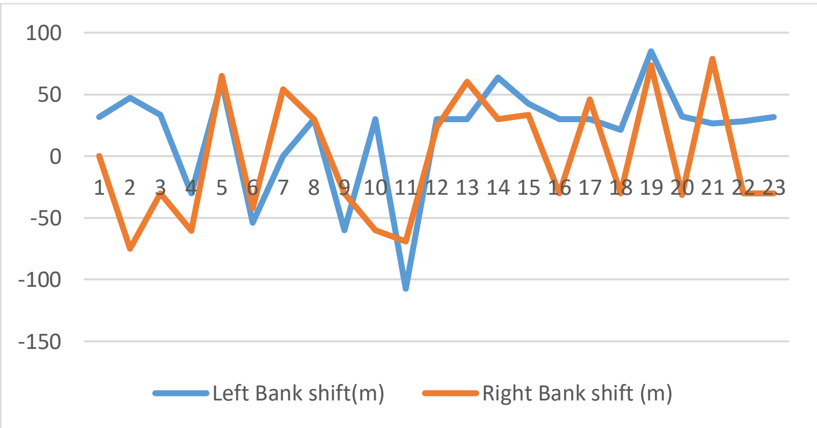

Fig. 5: Bank line shifting of the river (2006-2018)

Error! Bookmark not defined.The table shows total aggradation and degradation for entire time period of our selected timeframe 1994 – 2018.

Table 3: Data of Bank line shifting in (1996-2006)

Location	Left Bank shift (m	Direction	Degradation/Aggradatio	Right Bank shift(m	Direction	Degradation/Aggradatio
Dowarabazar 1	87.09	Right	Aggradation	114.59	Right	Degradation
Dowarabazar 2	75	Right	Aggradation	87.46	Right	Degradation
Haripur 1	42.42	Right	Aggradation	30	Left	Aggradation
Haripur 2	63.07	Left	Degradation	60	Left	Aggradation
Betura 1	90	Right	Aggradation	73.77	Right	Degradation
Betura 2	75	Left	Degradation	108.17	Left	Aggradation
Chatak 1	No shifting			41.83	Right	Degradation
Chatak 2	60	Right	Aggradation	60	Right	Degradation
Chatak 3	90	Left	Degradation	60	Left	Aggradation
Chatak 4	114.23	Right	Aggradation	75	Left	Aggradation
Kalaruka 1	127.27	Left	Degradation	116.19	Left	Aggradation
Kalaruka 2	42.37	Left	Degradation	30	Left	Aggradation
Kalaruka 3	78.51	Left	Degradation	64.02	Left	Aggradation
Kalaruka 4	63.63	Right	Aggradation	83.61	Right	Degradation
Kalaruka 5	100.62	Left	Degradation	128.16	Right	Degradation
Kalaruka 6	53.5	Left	Degradation	30	Right	Degradation
Lamakazi 1	90	Left	Degradation	60	Left	Aggradation
Lamakazi 2	30	Left	Degradation	30	Left	Aggradation
Gopall	75	Right	Aggradation	60	Right	Degradation
Mollapara	No shifting			41.45	Right	Degradation
Dakshin Kushigha	No shifting			114.1	Right	Degradation
Maijbag	No shifting			30	Right	Degradation
Hajipur	42.42	Left	Degradation	42	Right	Degradation

Discussion

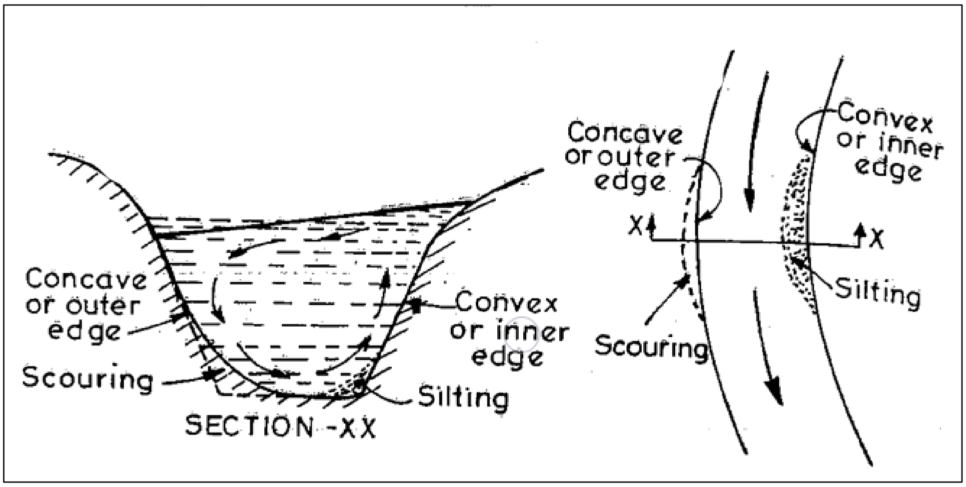

Fig. 6: Scouring and siting mechanism of river

When the flow moves round the bend, a centrifugal force is exerted upon the water, which results in the formation of traverse slope of water surface from the convex edge to the concave edge, creating greater pressure near the convex side. To keep its own level, water tends to move from the convex side towards the concave. However, the topmost water surface movement is prevented by the centrifugal force. Moreover, towards the bottom, the velocities are much less than towards the top and enough centrifugal force is not available to counteract the tendency of water at the top to move inwards. Hence, the water dives in, from the top at the concave end and moves at the bottom toward the convex end. These rotary currents cause the erosion of concave edge and deposition on convex side. When once a bend is formed, the flow tends to make the curvature larger and larger.

Analyzing the above theory, maximum shifting is supposed to be occur at the bending of the river. And we also found that. In 1994-2006 maximum shifting was found at kalaruka 1 that was situated at the bend of the river. Same evidence was also found for 2006-2018.

Conclusion

- In light of the bank line shifting data that have been shown aforesaid, in 1994-2006 maximum shifting was found at kalaruka 5, in 2006-2018 maximum shifting was found at kalaruka 1.
- In 1994-2006 no shifting was found at chatak 1 and also in 2006-2018 there was no shifting at chatak 1 left bank despite of having some in right bank.
- It is evident from the data that most shifting occurs at the point where the river changes its direction which is also known as meandering characteristics. For example, maximum shifting was found for the entire time period of 1994-2018 at kalaruka 5 point where the river changes its direction.

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A STUDY ON GEOTECHNICAL CHARACTERIZATION OF RED SOIL OF DHAKA CITY OF BANGLADESH

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Abstract

Generally red soils characters are different from other soils. In Bangladesh, a very few red soils are characterized in geotechnical point of view. However, it is necessary to massive study on it. From this point of view, a study has been undertaken to characterize the red soils from their properties which assist the engineer for design of structure of foundation on red soil and same strata of red soil in other area. From this study the iron concentration have been found about 27.8% for red soils with 38% clay whereas soils with 4% clay contain iron concentration of about 12.9%. That is soils containing high percentage of clay have more iron and manganese concentration in compare with soils containing low percentage of clay. The pH values have been varied from 5.6 to7.026. It indicates the soils are strongly acidic and sometime it is neutral. The natural moisture content varies from (8.90-28.99)% whereas plasticity ranges from (28-32)% and shrinkage limit varies from 7% to 22%.The consistency ranges from stiff to very stiff and the red soils have contained particles sand (12-25)%, silt (39-49)% and clay (26-39)%. The dry density of these soils varies from (14-16) kN/m^3 and the specific gravity of the cohesive soil varies from (2.675-2.684) and of the non-cohesive soil varies from (2.642-2.653). The study finds that the natural moisture content is not adequate to fulfill engineering needs. Plasticity as well as stiffness of red soil should also be considered to draw engineering attention for construction works.

Keywords: Dhaka city**,** Red soils, geotechnical characterization, iron content, alkali content.

Introduction

Geological Formation of Bangladesh

Brammer (1996) classified Bangladesh as three main geological formations of area. Tertiary sediments in the northern and eastern hills; the Madhupur clay of the Madhupur and Barind tracts in the center and west; and recent alluvium underlying the floodplain and estuarine areas which occupy the remainder of the country. The Madhupur and Barind tracts which together occupy about 8 percent of the country are underlain by the Madhupur Clay. The same formation may occur also on the so-called Akhaura Terrace and on the summit of the Lalmai hills. Unweathered Madhupur Clay is remarkably homogeneous in appearance throughout its extent, both vertically and laterally. It comprises a layer of unconsolidated clay about 10m thick near Dhaka, but it apparently becomes thin towards the east and is much thicker in the west of the Barind tract.

Nature of Red Soil

Siyanbola Samuel Malomo, (1977) expressed that red soils are tropically weathered soils with a high concentration of sesquixides of iron and/or alumina. They have correspondingly low content of alkalis and alkaline earths. They exist in wide ranges of chemical composition. Silica content varies from low to medium and exists usually as kaolinite, whenever it is found in substantial amounts.

Mishra & Suresh (2017) explained that red soils are formed due to weathering of igneous rocks. They are deficient in nutrient sand humus and have low water holding capacity. Red soils form the second largest soil group in India. The red color is mainly due to the presence of iron oxides.

These soils are found with low rainfall and they are not capable of retaining moisture. Red soil possesses lower strength compared to other soils due to its porous and friable structure.
They also worked to improve the engineering and strength properties of these soils by adding some additives to these soils. They use plastic products such as polythene bags, bottles, chairs, toys etc. which creates much environmental problems and increases day by day. They observed that the effect of addition of various percentages of waste plastic bag strips enhance the properties of red soil. Hence the use of waste plastics is a means of soil stabilizer and an economical utilization since there is scarcity of good soil for different engineering application.

pH

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Soils throughout the world exhibit different pH values. Because of the variations in climatic conditions, geology, land use, fluctuations in groundwater tables, etc. pH is an important factor for the chemical properties of the soil and can impact cation-and anion-exchange capacity by altering the charge on the soil colloids. Although some soils may exhibit electropositive charge, electronegative charge predominates in most soils. (https://www.sciencedirect.com)

Saroja & Visakhapatnamp (2017) studied on the investigation in the red soils of Visakhapatnam region as the region is significant with red soil deposits. They have been verified red soils collapsibility and effective utilization in geotechnical applications based on these values. They conclude that soil compacted at low dry densities and low water content exhibited high degree of collapsibility. They also conclude that saturation destroys the bond between sand particles by dissolved clay particles and salts of oxides leads to increases the collapsible behavior. They recommend that structures located on this soil deposit need a special attention to understand the behaviour of the soil to avoid distress.

Mohanalakshmi et al (2016) has studied on the effect of wollastonite ($Casio_3$) on geotechnical properties of soil. They observed that addition of wollastonite improves the properties of the red soil. It increases of maximum dry density (MDD) by the addition of wollastonite enhances the strength of the red soil. It also improves unconfined compressive strength results in the reduction of difficulties in foundation work. They also found that the addition of wollastonite to the red soil lead to the reduction of optimum moisture content (OMC) and increases of MDD (maximum dry density).

John et al (2017) studied the soil properties in presence of iron as iron is one among the oldest heavy metals existing on earth and the presence of iron in soil can also result in the alteration of soil properties. Their study deals with the presence of iron and how iron can alter the liquid limit, plastic limit and hydraulic conductivities of two soils; one of low plasticity (CL) and another of high plasticity (CH). They observed that an increase in liquid limit of 41% was observed for CL clay and a decrease by 50% for CH clay. The plastic limit decreased by 35% for CH clay and increased by almost 85% for CL clay. They also observed that permeability decreased by two orders, i.e., from 10^{-7} to 10^{-9} cm/sec on the addition of the contaminant, then as the concentration of contaminant increased, a slight increase in permeability was observed, but still lower than that of virgin soil.

Bangladesh is occupied by red soils of about 8% of the country. Red soil is formed due to weathering of igneous rocks with a high concentration of iron. They have correspondingly low content of alkalis and alkaline earths. Red soils possess lower strength and different properties compare to other soils due to its porous and friable structure. Their character is different from other soil. That's why an attempt has been made to undertake the study. In order to characterize the red soils of Dhaka city of Bangladesh, data have been collected from soil testing report of River Research Institute (RRI).

Though Bangladesh is occupied by red soil of small area of the country but their characters are different from any other soils in engineering practice however, research and defining character of red soils in engineering aspect is limited. Under such circumstances, an attempt has been made to characterize the red soils of Dhaka city of Bangladesh from their properties which assist the design engineer for other structure of construction on red soil and same soil strata in the other area.

Objectives

The specific objectives of the study have been described below-

- to determine geotechnical properties of red soil in the study area
- to determine iron and some other chemical properties of red soils in the study area.
- to characterize the red soil as geotechnical point of view.
- to draw engineering attention on geotechnical properties of red soils for construction works.

Methodology

The soil testing parameters are collected from the soil testing report of geotechnical research directorate (GRD) of River Research Institute (RRI).The sample was sent by SDS Engineering and Construction through boring and field investigation from the site Kurmitola, Dhaka Cantonment, Dhaka which has been shown in Fig-1. The sub-soil has been explored up to 15m and the samples were collected at an interval of 1.5m depth. The boring log of the hole has been presented here recording ground water table, reduced level, SPT value, colour as per depth which provide field investigation synopsis.

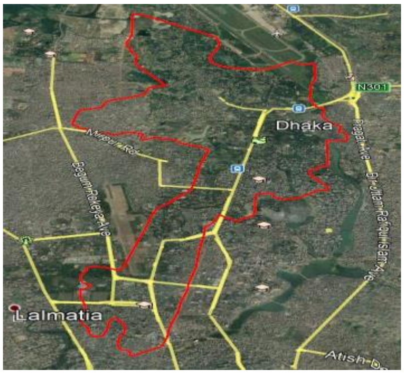

Fig. 1: Dhaka Cantonment area (red line).

Table 1. Showing soil profile of field investigation and ground water level in accordance with depth and its reduced level

Sample ID	Depth (m)	Ground Water Table (m)	Soil Description	Colour	Reduced Level (m)
D-1	1.5				
D-2	3.0				
D-3	4.5		Medium to stiff cohesive soil	Red	
D-4	6.0				
D-5	7.5	2.2-6.90			10.006-10.298
D-6	9.0				
D-7	10.5				
D-8	12.0		Medium Dense Non-cohesive soil	Red	
D-9	13.5				
D-10	15.0				

Laboratory Investigation

Laboratory investigation is performed in RRI Soil Mechanics Laboratory and the tests are conducted in the same. In order to investigate geotechnical properties of the soils all the samples are visually examined and the representative soil samples are tested. One of the laboratory tests has been shown in the Fig-2. The respective results are shown in Table- 5. In this study, iron content is especially determined as red soil is responsible for its colour in accordance with depth and noticeable physical and index properties. The photograph of the selected four soils is shown in Fig-3. The chemical test has been conducted for selected red soils in the Chemical Laboratory and their respective results are shown in the Table-6 with their respective particle size.

Fig. 2. Plasticity (Liquid limit and Shrinkage limit) measurement test

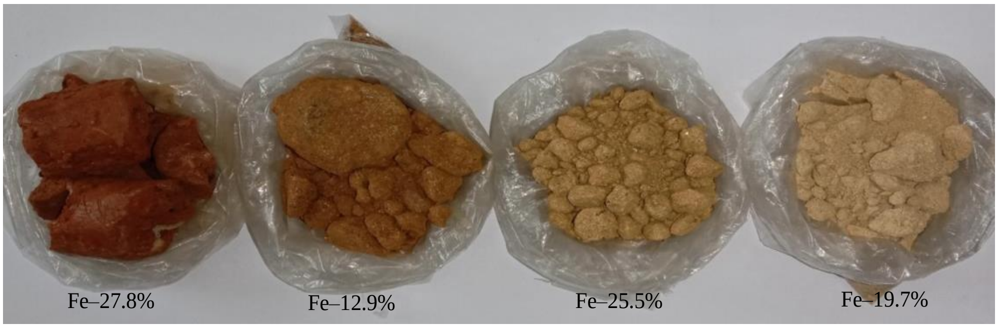

Fig. 3. A view of the selected typical red soils are shown up in accordance with iron content

Table 2. Relation of Consistency of Clay,Number of Blows N on Sampling spoon and Unconfined Compressive Strength,q_u in tons per sq ft

Consistency	Very soft	Soft	Medium Stiff	Stiff	Very stiff	Hard
Unconfined Compressive Strength,q_u (TSF)	0-0.25	0.25-0.50	0.50-1.00	1.00-2.00	2.00-4.00	>4.00
Compressive Strength(kN/m^2)	0-23.94	23.94-47.88	47.88-95.76	95.76-191.52	191.52-383.04	>383.04
Standard Penetration Resistance-'N'	0-2	2-4	4-8	8-16	16-32	>32

Table 3. Density Index (I_D) of Sand

Number of blows	Density Index(ID)
0-4	Very loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very dense

(**Source:** Terzaghi & Peck, 1948)

Table 4. Soil reaction is measured by laboratory or field tests and expressed on a logarithmic scale referred to as pH. Degrees of acidity and alkalinity corresponding with particular pH levels are given below-

Extremely acid	Below pH 4.5	Neutral	pH 6.6-7.6
Very strongly acid	pH 4.5-5.0	Mildly alkaline	pH 7.4-7.8
Strongly acid	pH 5.1-5.5	Moderately alkaline	pH 7.9-8.4
Medium acid	pH 5.6-6.0	Strongly alkaline	pH 8.5-9.0
Slightly acid	pH 6.1-6.5	Very Strongly alkaline	pH>9.0

(**Source:** Brammer, 1996)

Results and Discussion

Field Investigation Result

The data have been collected from the field investigation record. The graph is plotted as depth versus SPT-value. In graph depth is plotted as abscissa and SPT-value as ordinate.

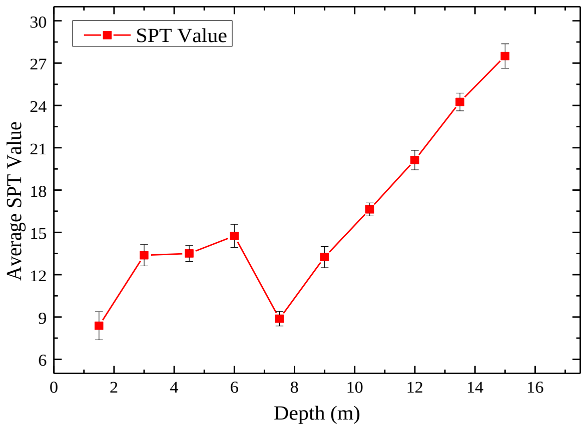

Fig 4. Graph showing the SPT value in accordance of depth

From the graphical presentation of field investigation result it has been observed that the SPT-value has been increased with the increases of depth. However, SPT-value decreases suddenly with the increases of depth at certain layer of soils. After that SPT value smoothly increases with the increases of depth.

Laboratory Investigation Result

Table 5. Showing the laboratory test results of physical and index parameters of cohesive and non-cohesive soils

Name of the parameter	Location Kurmitola, Dhaka Cantonment, Dhaka Cohesive Soil	 Non-cohesive Soil
Depth, in m	0-7.50	9-15
Colour	Red	Red
SPT value	4-19	11-29
Natural Moisture Content, NMC in (%)	8.90-28.99	
Liquid Limit, LL in (%)	56-62	
Plastic Limit,PL in(%)	28-30	
Plasticity Index,PI in (%)	28-32	
Shrinkage Limit, SL in (%)	7.049-21.56	
Organic Content in (%)	5.77	
Wet Unit Weight,γ_w in kN/m^3	18.51	
Dry Unit Weight γ_d in kN/m^3	15.13	
Specific Gravity, Gs	2.675-2.684	2.642-2.653
Compression index, Cc	0.147	
Hydraulic Conductivity, k in cms^{-1}	7.65×10^{-4}	9×10^{-2}
Unconfined compressive Strength, q_u (kN/m^2)	198.70 at 8% strain	
Cohesion, c (kN/m^2)	90.4	
Angle of internal friction ϕ (degree)	19	
Sand (%)	12-25	56-64
Silt (%)	39-49	28-30
Clay (%)	26-39	7-16

(Source: Report No. Soil- 11 (2018-19)

Table 6. Showing the results of laboratory test of iron, manganese, chloride, sulphate content and pH value in accordance of particle size

Location	Depth (m)	Colour	Fe (%)	Mn (%)	pH	Cl^- (mgkg^{-1})	SO_4^{2-} (mgkg^{-1})	Percentage of soil		
								Clay (%)	Silt (%)	Sand (%)
Kurmitola, Dhaka	4.5	Red Cohesive	27.8	0.564	5.66-7.08	110	115	38	44	18
	7.5		25.4	0.421				26	40	34
	9	Red Non-Cohesive	12.9	0.276				4	26	70
	13.5		19.7	0.378				8	30	62

(Source: Report No. Soil-11 (2018-19)

In graphical presentation, a plot of clay percentage versus NMC has been shown in Fig. 5(a).The graphs have been plotted altogether with clay percentage versus iron and manganese concentration in Fig.5 (b). Similar graphs have been plotted separately for manganese and iron which have been shown in Fig. 5(c) and Fig.5 (d) respectively.

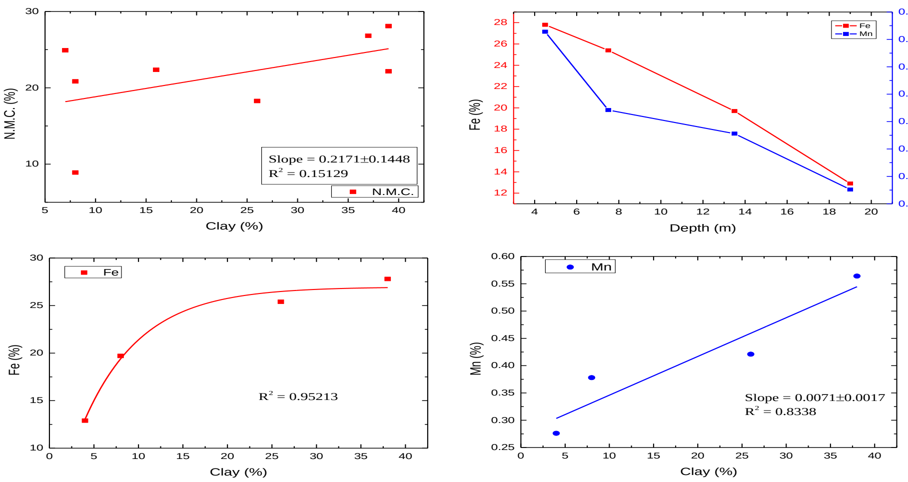

Fig. 5. Graph showing (clock wise) (a) the clay percentage with NMC (b) the depth vs. Fe and Mn concentration (c) the clay percentage vs. Mn and (d) the clay percentage vs. Fe concentration

From the result of Table and graphical presentation it has been found that the colour of the soil is red which contain more iron and as well as more percentage of clay. The percentage of iron increases with the increases of clay percentage. The manganese increases with the increases of iron. The consistency in terms of plasticity is high plastic as well as shrinkage limit varies. The stiffness has been varied from medium stiff to very stiff.

In the layer of 4.5m depth, it has been found that iron contains 27.8ppm where clay contains 38%,silt 44% and sand 18%. Manganese contains 0.564ppm, Chloride 110mg/kg and Sulphate 115mg/kg in which pH varies from 5.66-7.085.The soil contains organic matter in the percentage of 5.77%. The character of the soil is red cohesive and high plastic in terms of plasticity and stiff to very stiff in terms of consistency. The moisture content varies from (22-23) %.The dry density varies from (14-16) kN/m^3.The specific gravity of the soil is 2.684.

In the layer of 7.5m depth, it has been found that iron contains 25.4ppm where clay contains 26%,silt 40% and sand 34%. Manganese contains 0.421ppm. The character of the soil is red to brown cohesive and high plastic in terms of plasticity and the sample is medium stiff to stiff in terms of consistency. The moisture content varies from (26-28)%. The specific gravity of the soil is 2.683.

In the layer of 9m depth, it has been found that iron contains 12.9ppm where soil contains clay 4%,silt 26% and sand 70%. The layer contains 0.564ppm of Manganese. The character of the soil is red to brown non-cohesive. The soil contains high percentage of sand and it has no plasticity with medium dense in terms of density index. Colour of soils showing the results of iron content. The specific gravity of the soil is 2.654.

In the layer of 13.5m depth, it has been found that iron contains 19.7ppm where the soil contains clay 8%, silt 30% and sand 62%.The character of the soil is light brown non-cohesive. It has no plasticity as soil contains high percentage of sand and medium dense in terms of density index. Here colour showing the result of content of iron. The layer contains 0.378ppm of Manganese. The specific gravity of the soil is 2.646.

From the graphical presentation of iron and manganese concentration in depth it has been observed that iron and manganese concentration is lessen with increases of depth. The exception has been occurred in case of 9m depth. In this layer, the iron and manganese concentration has been lessened where the soil contains high percentage of sand.

Conclusion

In this study, a number of red soils are tested in the laboratory for determining engineering properties of soil as well as chemical properties. Representative four soils are selected for determining iron and manganese content and chemical tests. However, chloride and sulphate are tested for one layer soil only. From this study it is observed soils containing high percentage of clay have more iron and manganese concentration in compare with soils containing low percentage of clay. The pH of the soils has varied from acid level to neutral level. Natural moisture content has not increased as their plasticity. The stiffness has varied from medium stiff to very stiff. The very stiff soils are brittle if there is no sufficient natural moisture content.

Recommendation

Red soils are the composition of physical and chemical properties. For its characterization, a vast analysis of geotechnical and chemical knowhow are very important. In this study, a tiny attempt has been undertaken for knowing red soils of Dhaka city of Bangladesh. Though it will be very crucial to characterize the red soils with a very few test and analysis. However, it is a familiarization with red soils. Here it is recommended that red soils properties are not suitable to foundation of construction. Therefore, it is very essential to analyze all the physical and chemical properties of red soils of Bangladesh minutely and hence it can be managed the red soils as per need.

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EVALUATION OF URBAN DRAINAGE NETWORK PERFORMANCE UNDER DIFFERENT CLIMATIC AND LAND USE CONDITIONS USING HEC-HMS

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ABSTRACT

Unplanned urbanization along with city's changed landscape causes urban drainage changes and also fall short maintaining its capacity. This study therefore aims to evaluate the performance of drainage pattern under different land use and climatic conditions. Mahesh khal is taken as a study area, a major drainage canal connected with Karnaphuli River. This study analyses land use pattern of the study area with the data collected through field investigation and also gathered from the secondary sources using GIS tool. Chattogram city holds monthly average rainfall of around 243 mm and therefore totalling 2919 mm in a year which is about to increase 5% - 6% by the year2030. Out of total 8.59 square kilometres of total land areas 5.12 square kilometres areas of land occupied as vegetation and open areas which was about 60% of the total area in 1988 but unfortunately within the 30 years of time span the areas lost its 28% of the vegetation and open. Moreover the peak discharge found for 2, 5, 10, 25, 50 and 100 years return period were 19.8, 29.4, 35.8, 44.1, 50.4 and 56.5 m^3s^{-1}. The study also found that the peak discharge decreases with the increase of Roughness values because the canal with high roughness values indicates high weeds which will give more resistance than a clear canal. The results also revealed that peak discharge increases with the increase of Curve Number (CN) value and percent (%) impervious which indicates that landuse changes has great impact on urban drainage runoff as CN value and percent (%) impervious are related with landuse patterns of the urban areas.

Keywords: *CN, HEC-HMS 4.2, ArcGIS 10.4, Runoff, impervious, rainfall etc.*

Introduction

Hydrologic cycle is greatly affected with the growth of urbanization in many ways such as increases percent impervious areas(Lee & Chung, 2007, Schuelet, 2000), surface runoff, decreases vegetation and open space, infiltration of runoff into soils and base flow, withdrawing water (Chung, Park, & Lee, 2011),water quality replacing indigenous vegetation with irrigated ornamental vegetation etc. (Guan, Sillanpää, & Koivusalo, 2016; G. Krebs et al., 2014; Paule-Mercado et al., 2018; Pitt et al., 2008; Song & Chung, 2017). This conversation leads to change in physical, chemical and biological disturbance of the watershed of a drainage system (Giacomoni, M.H. Gomez, R. Berglund, 2014; Paule-mercado, Salim, Lee, & Memon, 2018; Yao, Wei, & Chen, 2016). Nowadays floods events are more frequent and devastating as the rate of urban growth is so rapid than urban drainage system (Chen, Hill, & Urbano, 2009; Hénonin et al., 2010; Kourtis & Baltas, Vassilios A. Tsihrintzis, 2018; Schmitt, Thomas, & Ettrich, 2004). Again flooding is directly linked with heavier storms is more likely to be increased with climate change (Blair & Sanger, 2016; IPCC, 2008; Sara C. Pryor & Scavia, 2014). Moreover climate also contributes in increasing precipitation, rising temperature and sea levels resulting multiplying the effects of the events (Walega, 2013). Therefore understanding the relation between land development and climate change on storm water runoff is particularly essential from practical point of view and is socially justified (Paule-mercado et al., 2018; Walega, 2013). Monitoring, analysis and subsequent implementation of the preventive measures in order to integrated management of the urban drainage runoff (Paule-mercado & Lee, 2017; V A. Tsihrintzis & Rizwan, 1998).

Transformation of precipitation into storm runoff is a complex hydrological process which requires nonlinear and dynamic transitions which includes soil type, infiltration, percent impervious, evaporation, evapotranspiration, land use conditions etc. (Wang, Asce, Altunkaynak, & Asce, 2012; Xu, 1999; Yokoo, Kazama, Sawamoto, & Nishimura, 2001). Nowadays Sustainable Urban Drainage Solutions (SUDs) or Low Impact Development (LID) have taken attraction because the concepts considers

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different aspects of the urban drainage system i.e. runoff quality, amenity and recreational value, social and ecological protection etc. Chittagong city is the second largest city of Bangladesh comprising hills formed during tertiary time. Majority of the people along coastal areas living between 0 to 5 meter elevation from mean sea level. It lies at the coastal area and the most prominent natural hazards are cyclone with storm surge, water logging, landslide, earthquake and flash flood are the dominant ones. But at present water logging and landslides are the most burning issues (Islam & Das, 2014). Due to rapid urbanization along with climate change, Chittagong city dwellers are facing water logging problem in last few years. The average rainfall of Chittagong is 3378 mm which is quite high than other locations in Bangladesh. Mostly rainfall occurs between May to October. In July, the precipitation reaches its peak, with an average of 743 mm (BMD, 2017).Naturally hydrological condition of an area comes first as it directly involve in water logging events (Zhang & Pan, 2014). The land use patterns of an area have influences over the hydrological condition while the increasing urbanization reduces water body and natural streams. There is an increasing trend observed for land use change due to migrating people from rural parts and this has an advance effect on the hydrological condition of city areas which sooner or later leads to water logging (Mohit & Akter, 2014). Chittagong city saw at least 12 canals vanish in the last 48 years, during which time the waterlogging problem accelerated. A mere 22 canals were found to be emptying into the Karnaphuli River and there was no trace of 12 canals in the premier port city where 8 of the 22 existing canals are also dying (Chowdhury, 2017). In recent years, major canals lost 42% carrying capacity due to siltation, with 87% of the existing silt traps being dysfunctional (Hussain, 2017). Over 14,000 ponds and other water bodies have disappeared is last 18 years in Chittagong. According to a survey conducted by District Fisheries Department in 1991, the number of water bodies in Chittagong city was 19,250 while the Featured Survey conducted by CDA in 2006-2007 indicated existence of 4,523 water bodies there. About 100 sq. km. water of Chittagong city is pumped out through five canals- Chaktai khal, Mahesh khal, Sub area khal, Monohar khal and Hizra khal.

Scientists have given much concern about the functionality of the traditional drainage system due to its adverse effect on environment. In traditional urban drainage system, surface runoff from impervious areas may increase the occurrences of frequent flooding also may cause sudden rise in water level and may cause poor water quality in natural water bodies. As the rainfall diverted through pipe system in traditional method, the total amount of infiltrated underground reduced which causes depletion of the ground water table (Grimm, 2007,EA, 2007). There is also limited capacity and flexibility of traditional drainage system to adopt urbanization effect and climate vulnerability(P. Krebs & Larsen, 1997). Hence the concept Sustainable urban drainage (SUDs) comes to mitigate these problems. Sustainable urban drainage (SUDs) refers to management of water in small scale and facilities surface runoff in a more sustained way focusing on maintaining good health, preserving water resources and protecting biological diversity and natural resources (Bruijn et al., 2009; Mcdonald, 2018; Willems & Olsson, 2009; Zhou, 2014). The simplest rational method considering “runoff coefficient” have already been introduced to determine the total runoff. This been done multiplying the coefficient with total rainfall (Zekai ,S., Altunkaynak, 2006). Then imperviousness along with other factors such as time of concentration, soil properties, land use conditions of a catchment have been converted into a regression formulation or tabulated values to have more accurate prediction about runoff. Despite all the attempts, city people are facing tremendous problem related to drainage issues. Water logging are much more frequent during monsoon than that of before. Areas of vegetation have been reduced as result of urbanization. Climate pattern also changes rapidly due to geographical location of the city. Urban drainage behavior also changes with the changes of land use pattern and climate changes. The study has undertaken three objectives to know the behavioral change of the urban drainage under different changed climate and land use pattern.

a. To evaluate the changes of drainage network in different land use and climatic conditions
b. To simulate the existing drainage network by using primary data to replicate real scenario.
c. To evaluate the performances of the existing drainage network in different land use and climatic conditions.

Methodology

Study Area

Mahesh Khal, one of the major khal in Chattagram city connected with the Karnaphuli River is taken as study area as shown in Fig. 1. The catchment lies between latitude $(22°17'49.751"N - 22°20'22.2612"N)$ and $(91°46'30.6948"E - 91°48'45.2412"E)$ and occupies the area about 8.578 Km^2 (857.8 ha) with 16.37% inclination. The canal is located in between Sadarghat and Khal 10 station. The study area is classified in 17 catchment (Sub-Basin) and these area is mainly used for commercial and residential purposes. 31.52% of the area is vegetation and open space, 14.97% is water body and 53.51% is build up area. The length of the canal is about 6.3 Km considered for this study and divided into 6 reaches. The canal collects the natural flow along with water draining from Sub-Basin area at the upstream and finally discharges into Karnaphuli River at downstream. Total 11 points have been selected for collecting data i.e. cross section, bottom materials, tide table, discharge etc.

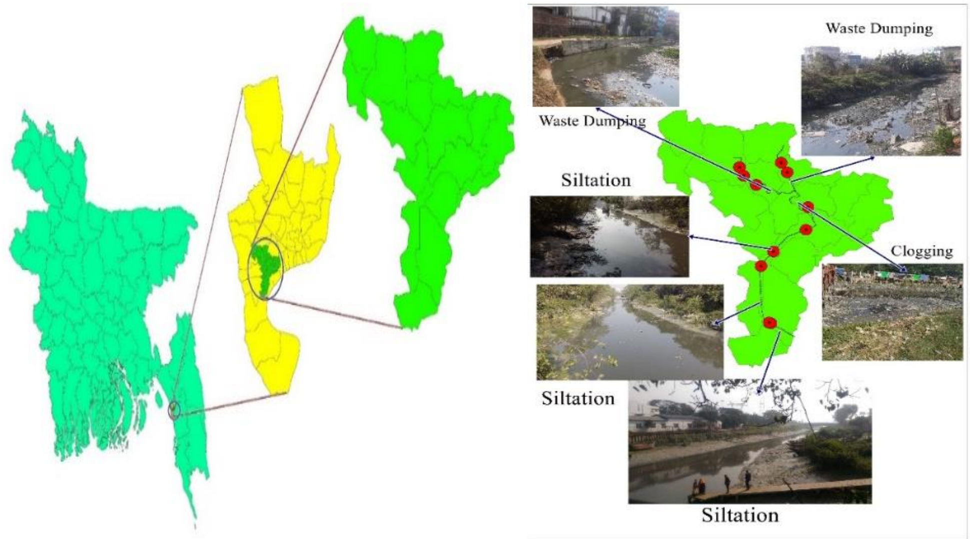

Fig. 1. Study area (a) location of study area showing Mahesh Khal (b) physical conditions in different place.

Methods

The methodology showing in Fig. 2 starts with collection of primary and secondary data. Primary data includes cross section, side slope, bottom slope, bottom materials, tide level, discharge of the canal also types of land use land cover (LULC), flow path etc. Due to lack of data, the cross section of the canal was taken manually. Total 11 study points were selected for data collection. The bottom width of the canal was divided into several strips and depth of the bottom of the canal was determined by rope with a mass attached at the bottom of the rope.

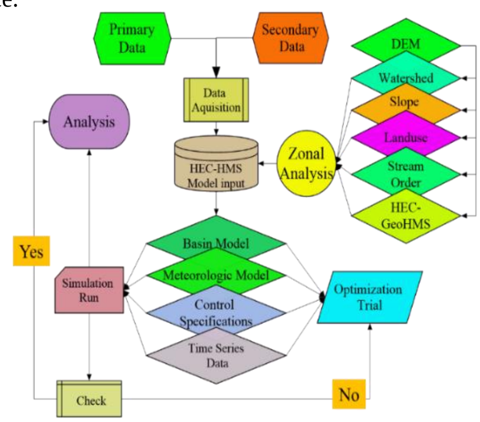

Fig. 2. Details Methodology

The length of the canal was determined by ArcGIS 10.4 using field calculator. The average bottom slopes were determined using GPS and adjusted with data found from HEC-GeoRAS. Bottom materials were investigated physically and recorded for the selection of Manning's n. Field land use and land cover data were recorded in different locations for accuracy assessment of the land use map prepared from DEM by ArcGIS 10.4. Field investigation was also included opinion survey for problem identification, flow pattern of the canals, causes of overflow etc. Secondary data are collected from different sources. Digital Elevation Model (DEM), Land use map, soil data map, precipitation, tide tables, discharge etc. are the main secondary data used for the study. Secondary data and their sources are given in Table 1.

The equation used for the calculation of the time of concentration for the watershed has been taken from literature and is specified in Eq. (1) (Thompson, 2006).

$$Time\ of\ Concentration, t_c = \frac{FL}{A^{0.1} \times S^{0.2}} \qquad \text{Eq. (1)}$$

Where,

t_c(min) = Time of Concentration

L (Km) = Length of the stream

A (Km^2) = Sub-basin area.

S (m/Km) = Overland slope

F = 58.5 When A in Km^2

SCS unit hydrograph was adopted for flow routing under transform method. For the purposes the required data is lag time (min) and the calculation is based on empirical equations mentioned in Eq. (2).

$$Lag\ Time, t_{lag}(h) = \frac{2.587 \times L^{0.8} \times (\frac{1000}{CN} - 9)^{0.7}}{1900 \times H^{0.5}}$$

Eq.(2) (Schwab, G.O., Fangmeier, D.D., Elliot, W.J., Freveret, 1993)

Where,

L (m) = Hydraulic watershed length = $110A^{06}$

A (ha) = Sub-basin area.

CN = Curve number.

H (%) = Average sub-basin land slope.

=Calculated based on (Chow, 1964)

i.e. $H = \frac{100 \times CI}{A}$

Eq. (3)

(C is the summation of the length of the contour lines that pass through the watershed drainage area on the quad sheet and I is the contour interval) .

Table 1. Necessary Data Sources of the Study

Data	Source	Address	Resolution /Periods /Others
DEM	United States geological Survey (USGS)	https://earthexplorer.usgs.gov/	30m
Land Use Map	GlobeCover	http://due.esrin.esa.int/page_globcover.php	1 : 500000
Soil Data Map	Food and Agricultural Organization (FAO)	http://www.fao.org/geonetwork/srv/en/metadata.show?id=14116	1000m
Precipitation	National Aeronautics and Space Administration (NASA) Bangladesh meteorological department (BMD)	https://earthdata.nasa.gov/ www.bmd.gov.bd	2018
Discharge	Bangladesh Water Development Board (BWDB)	https://www.bwdb.gov.bd/	2018

The USDA Natural Resources Conservation Service (NRCS) method previous, known as SCS has been used for the computation of storm water runoff rates, volumes and hydrograph. The NRCS Curve Number (CN) is the key component of NRCS method which depends on soil permeability, surface cover, hydrologic condition etc. The most commonly data used for CN value is the June, 1986 Technical release 55 – Urban Hydrology for small watershed (TR-55)(USDA, 1986).

Result and Discussion

Land Use Analysis

Remote sensing and GIS technique is the most important tool for studying the land use and land cover analysis. Large land area can be mapped with low cost and rapidly with high accuracy. Major three land use classification have been identified for the study area and results are presented in the Fig. 3 and Table 2.

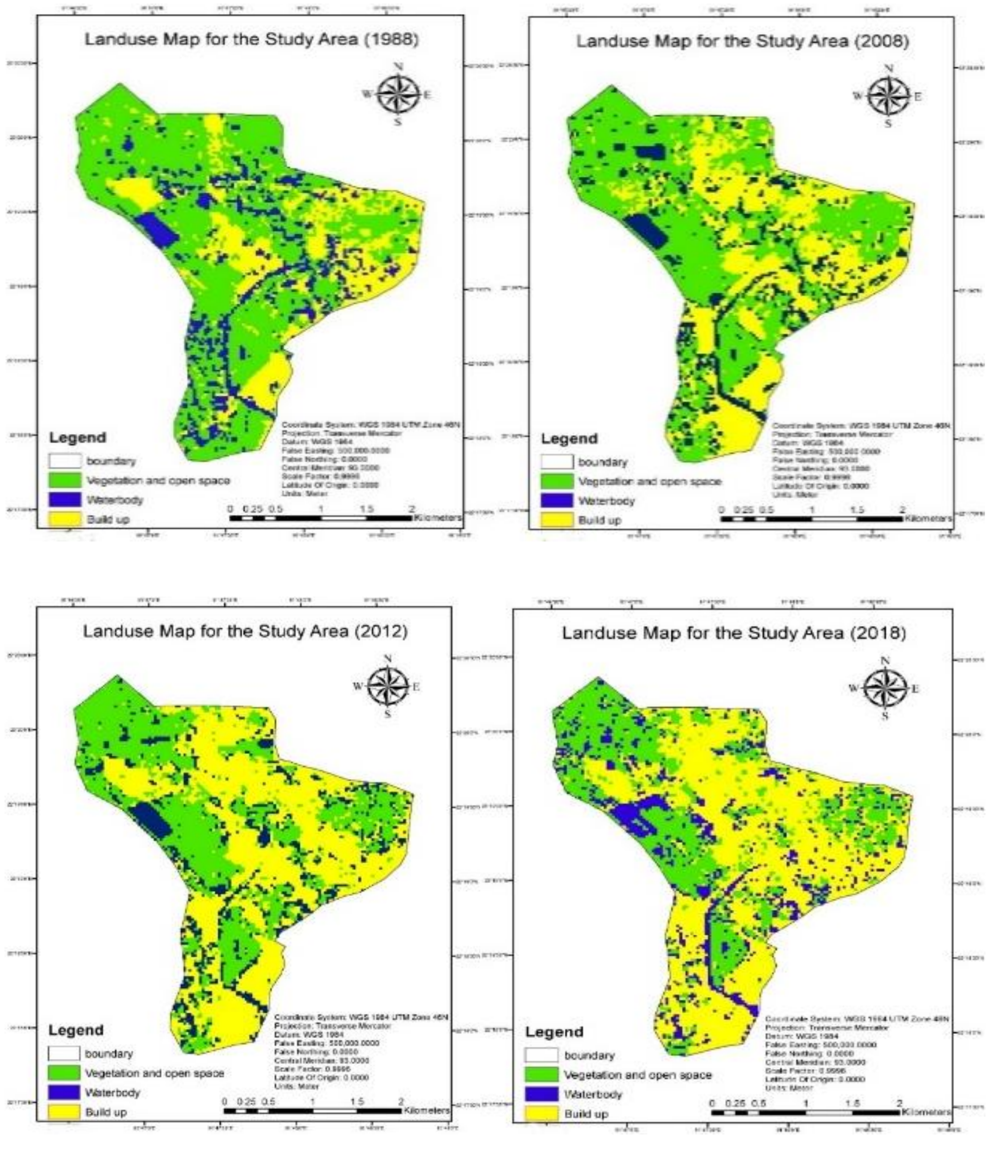

Fig. 3. Land use maps of the study area for the year 1988, 2008, 2012 and 2018

Table 2. Land use analysis of the study area

Type of land use	Area-1988 (%)	Area (sq km)	Area-2008 (%)	Area (sq km)	Area-2012 (%)	Area (sq km)	Area-2018 (%)	Area (sq km)
Vegetation & open area	59.64	5.1219	48.85	4.19	36.71	3.15	31.52	2.7072
Water	15.34	1.3176	14.37	1.2339	14.41	1.24	14.97	1.2852
Build up	25.02	2.1483	36.78	3.159	48.88	4.20	53.51	4.5954

The classification process was repeated for respective year. The generated classified land cover map was verified using ground data and Google earth. The Fig. 3 shows the Land use maps of the Mahesh khal watershed area from the year 1988 to 2018 with different interval. The Fig. 3 clearly illustrated that build up areas are increasing in an alarming rate whereas open and vegetation areas is decreasing day by day. Table 2 shows the detailed result obtained from the land cover classification of three types of land use analysis.

From the year 1988 to 2018 the build-up area increased about 28.49%. Initially in the year of 1988 the build-up area was about 2.15 square kilometres which was 25.02% of the total area of 8.59 square kilometres. The trend of change in build-up areas was slower up to 2008 as compared with the changes found later years. The build-up area was about 3.16, 4.20, 4.60 square kilometres for the year of 2008, 2012 and 2018 which is 36.78%, 48.88% and 53.51% of the total area. Out of total 8.59 square kilometres of total land areas 5.12 square kilometres areas of land occupied as vegetation and open areas which posed the highest portion of the total area and was about 59.64% of the total area in 1988. Within the 30 years of time span the areas lost its 28.12% of the vegetation and open areas. The trend of change is almost same but in reverse order as compared with build-up area and can be termed as alarming.

Data Preparation

The basin and canal parameters were extracted from the attributes table for 17 sub-basins and for canal from ArcGIS 10.4 as prepared earlier and summarized in Table 3. Area, slope, percent (%) impervious are directly derived from ArcGIS 10.4. Hydraulic length, initial abstraction, lag time are derived using respective equation's mentioned in methodology chapter. Curve Number (CN) for each sub-basin was used from TR-55 Curve number Tables. The corrected curve number (CN*) found after optimization trials in HEC-HMS.

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Table 3. Physical properties of Sub- basins used in model

Basin ID	Basin Area, A (Sq. km.)[a]	Basin Area, A (ha)	Slope (percent rise) [b]	Slope, H (%)[c]	Hydraulic Length, L (m) [d]	Curve Number, CN [e]	Curve Number, CN*	Initial Abstraction, Ia (mm) [f]	Percent impervious (%)[g]	Lag Time, lag (min) [h]	d/s of sub basin
S0	0.30	30.20	1990.75	18.85	1523.36	84	79.06	13.45	37	16.38	R5
S1	0.40	40.01	2370.66	22.53	1675.36	80	76.83	15.32	4	17.29	R5
S2	0.47	47.46	1290.76	12.49	1114.83	87	83.56	10.00	63	13.57	R6
S3	0.49	48.88	1742.09	16.59	1134.61	82	78.75	13.71	43	13.92	R3
S4	0.32	31.97	1353.96	13.06	879.52	86	75.85	16.17	73	13.95	R5
S5	0.32	32.03	1429.04	13.74	880.46	87	76.73	15.40	68	13.27	R6
S6	0.59	58.89	1359.65	13.11	1268.81	86	82.59	10.71	56	15.16	R4
S7	0.36	35.65	1785.73	16.99	938.91	82	78.75	13.71	44	11.82	R4
S8	0.49	48.87	1759.47	16.75	1590.26	83	79.71	12.93	38	17.62	R3
S9	0.19	19.02	1909.30	18.11	1044.10	86	82.59	10.71	62	11.04	R3
S10	0.45	44.53	1733.08	16.51	1072.89	85	81.63	11.43	75	12.19	R3
S11	0.37	37.32	1800.65	16.13	965.10	85	81.63	11.43	45	11.33	R2
S12	0.34	33.91	2315.28	21.98	911.22	87	76.73	15.40	37	10.78	R3
S13	0.41	40.93	1489.56	14.30	1019.99	86	82.59	10.71	56	12.19	R1
S14	0.68	68.07	1437.55	13.82	1384.08	83	79.71	12.93	42	17.36	R1
S15	0.44	44.13	1811.36	17.22	1067.12	87	83.56	10.00	78	11.16	R2
S16	0.75	74.71	1872.21	17.78	1463.55	85	81.63	11.43	45	15.06	R5

CN* Corrected CN values;
[a, b, c, d] DEM;
[e]Curve Number Chart; [f]
empirical equation (USACE, 2000); [g]collected from land use map;
[h]Schwab's equation;

Table 4. Reach parameters used in the model

Reach Name	Length (m)[a]	Top width (m) [b]	Depth (m) [b]	Bottom width (m) [b]	Bottom Slope (m/m) [b]	Side slope (1:z) [b]	Manning's n [c]
R1	557.18	39.01	5.63	6.67	0.0011430	0.35	0.04000
R2	935.75	34.51	4.80	11.57	0.0021100	0.42	0.04500
R3	1196.06	39.22	3.51	10.57	0.0045000	0.24	0.04500
R4	1118.85	33.05	3.71	19.56	0.0051300	0.55	0.07750
R5	1601.93	17.27	3.34	8.06	0.0065400	0.73	0.10000
R6	896.04	10.82	2.87	7.20	0.0063700	1.59	0.07000

[a]DEM
[b]Field Survey
[c]Manning's n chart

Table 5. Others parameters used in model

Storm depth (mm) [a]						
Present	2 year	5 year	10 year	25 year	50 year	100 year
74.13	91.06	122.62	143.57	169.99	189.58	209.06

Others parameters		
Properties	Value	Unit
Catchment Area, A [b]	8.58	Km^2
Overland Slope, S [b]	16.37	%
Overland Slope, S [b]	163.70	m/Km
Length of the stream, L [b]	6.30	Km
Time of Concentration, Tc [c]	107.24	min

[a]IDF curve
[b]DEM
[c]Rational method

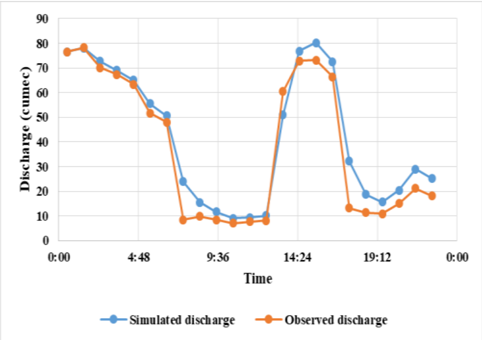
Reach parameters are shown in Table 4. The values are found through field survey. Length, top width, depth, bottom width, side slope are determined direct measurement in the field. Bottom slopes have been determined using GPS instrument with respect to reduced level and finally validated and adjusted with the data extracted from DEM using 3D analyst in ArcGIS 10.4. Manning's n value used for the canal found from TR-55 Manning's n table and validated in optimization trials in HEC-HMS.

Storm depths for different return periods have been shown in columns (2), (3), (4), (5), (6) of the Table 5 for the 2, 5, 10, 25, 50 and 100 year respectively. The storm depths are adopted from IDF curve. Others parameter includes total catchment area for the catchment, overland slope, total length of the canal and the time of concentration etc.

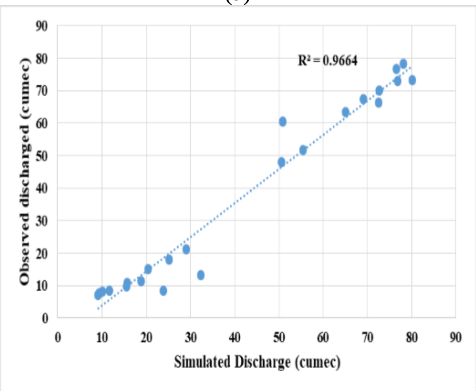
Validation of the model

Successful implementation of hydrological models mainly dependent on how accurately the model is calibrated. Model calibration is done to match the values of runoff volume, peak discharge and time of hydrograph among observed and simulated values and results sown in Fig 4. The model calibration can beconducted both automatically and manually. In the present study, automatic calibration known as.

"Trial Optimization" was used to obtain the optimum parameter values that gives the more similar values among observed and simulated values as manual calibration could be erroneous The assumed parameters undergoes an iterative adjustments under certain boundary conditions. The calibration was done between simulated and observed discharge and the results shown in Fig. 4(a). HEC-HMS model basically calibrated using event based simulation. A particular event (24 July, 2018) was selected for calibration of the HEC-HMS model parameters. The hydrograph generated from the model is compared with the observed direct runoff. Two important parameters Curve Number (CN) and manning's n were selected for calibration. Initial and corrected parameter values are show in Table 3 and Table 4. The corrected and calibrated values are considered for the further analysis and performance evaluation of the drainage system.



(a)



(b)

Fig. 4. Validation of the model (a)Observed and simulated discharge (b) accurecy analysis

The observed and simulated values were assessed using R^2 indicator and the R^2 value obtained after implementation of all calibrated values for this particular event is 0.9664 which indicates good accuracy of the calibration as shown in Fig. 4(b).

Performance evaluation based on present situation

For the simplicity, all the performance evaluation was conducted considering metrological effect only. Tidal effect, metrological effect, backwater effect and inflow were not considered. The reasons behind this approach are not availability of sufficient data set, no future master plan for the study area, limitations of the HEC-HMS model, uncertainties etc. The capacity of the canal has been considered subtracting the average peak tide discharge from actual capacity. Hence the capacity for the canal with respect to metrological consider for further performance evaluation. The cross section considered for determination of the actual capacity is the average cross section of the whole canal. The discharge due to tidal effect have considered the average peak discharge available of the canal throughout the year. The result found that the actual capacity of the canal is 103.85 m^3s^{-1}whereas the average peak tidal height along with inflow has been found 3.75m and corresponding discharge due to tidal effect and inflow has been found 87.34 m^3s^{-1}. Hence the capacity with respect to metrological effect is only 16.51 m^3/sec during the high tide. Such kind of tidal effect cause frequent flooding in these area with limited rainfall.

Performance evaluation varying Curve number (CN)

Fig. 5 illustrates the change of discharge with different CN number varying from 30 to 98 in different return periods. Basically Curve Number (CN) value is a hydrological parameter that used to predict the direct surface runoff. Considering water present in canal for tidal effect, the peak discharge would be within the capacity in present condition and also in 2 years return period for CN values up to 30, 40, 50 respectively but for further increased values of CN, it has found to exceed the carrying capacity limit of the canal. As with the increase of CN values, the impervious areas increase resulting more direct surfaces runoff that's why the discharge has been found higher for higher values of CN.

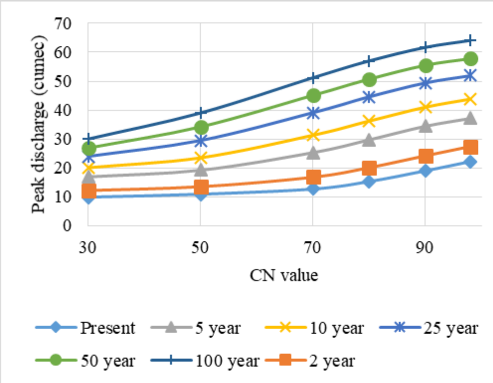


Fig. 5. Variation of discharge with CN value in different return periods

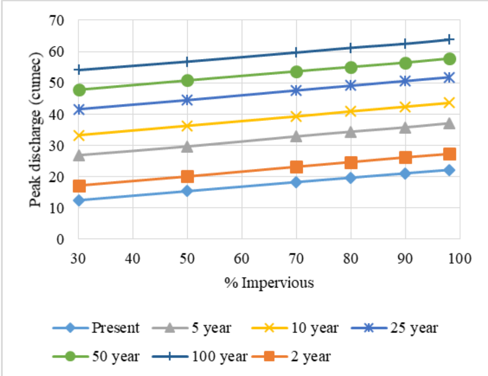


Fig. 6. Variation of discharge with % impervious in different return periods

The deviation changes at a constant rate in different return periods for a particular CN value. The average values found for 2 and 100 year return periods are 26 m^3s^{-1} and 50.5 m^3s^{-1}respectively with respect to different CN value. The result also shows that the deviation changes more rapidly up to CN value 70 and for further increase of CN value the deviation is almost same. The average discharge value found for CN value 30 and 90 are 20 26 $m^3s^{-1}$and 43.47 26 m^3s^{-1}respectively.

Performance evaluation varying % impervious

The change of discharge with different % impervious land varying from 30% to 98% in different return periods has been illustrated in Fig 6. Percent (%) impervious indicates the area which will contribute 100% surface runoff without any loss (i.e. infiltration, percolation etc.). The discharge increase linearly with the increase of the % impervious as shown in Fig. 6. The more impervious areas increases, there will be more surface runoff and hence the discharge would be higher that’s why the value increases with the increase of % impervious.

The study also found that average value of discharge for 2 year and 100 year return period have been found 18.2m^3s^{-1} and 59.68m^3s^{-1}respectively. The standard deviation found almost same for all return periods and the value is about 3.78. On the other hand, the average value changes from 33.34 to 43.37 $m^3s^{-1}$for the % impervious value of 30 and 98.

The standard deviation value found almost same for different % impervious value and the value is about 15.48. The carrying capacity of the canal exceeds almost every values of % impervious for any return periods considering back flow from river. But if tidal water not considered and only meteorological effect considered for carrying capacity of the canal, the canal will be active for all the values of % impervious up to 100 year return period.

Performance evaluation in different return periods

The change of discharge with time has been found in the for different return periods and shown in Fig. 7. For a day period, the maximum discharge has been found from 9am to 2pm. In present situation the maximum discharge value is about 15cumec which increase with the increase of return period and become 55cumec in 100years return period.

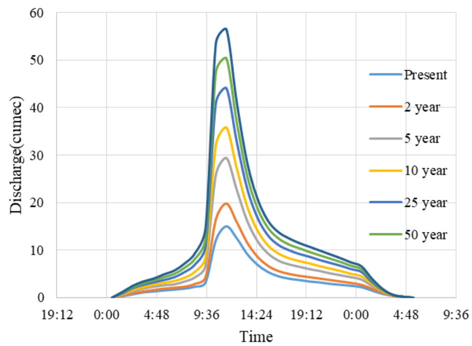
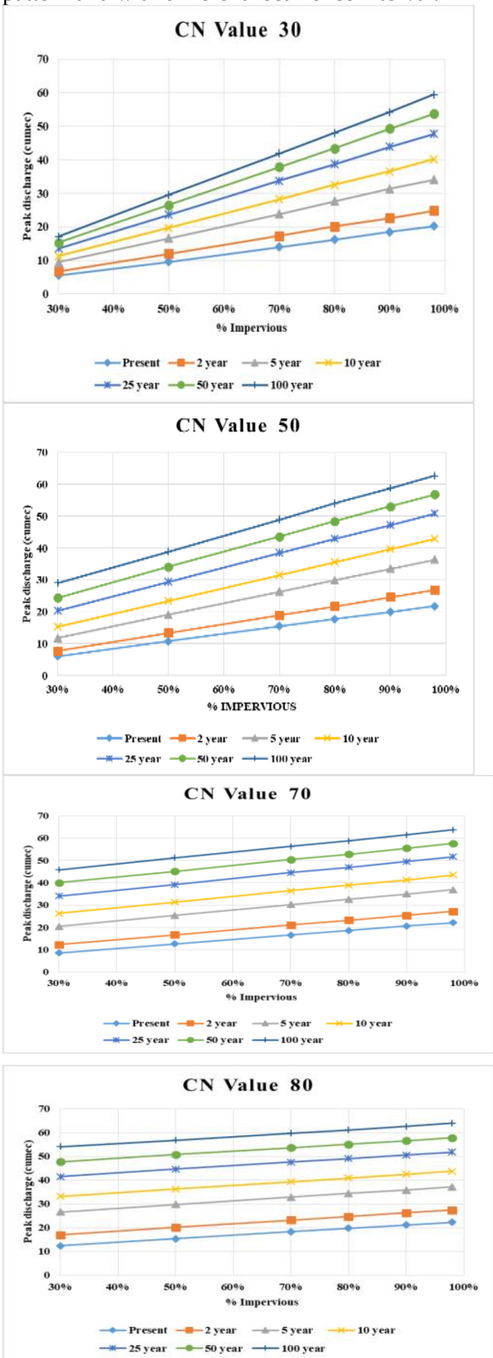


Fig. 7. Variation of discharge in return periods

Performance evaluation varying CN value with % impervious for different return periods

The graphs in Fig. 8 illustrate the change of pick discharge with respect to change in land use pattern and with different recurrence interval.



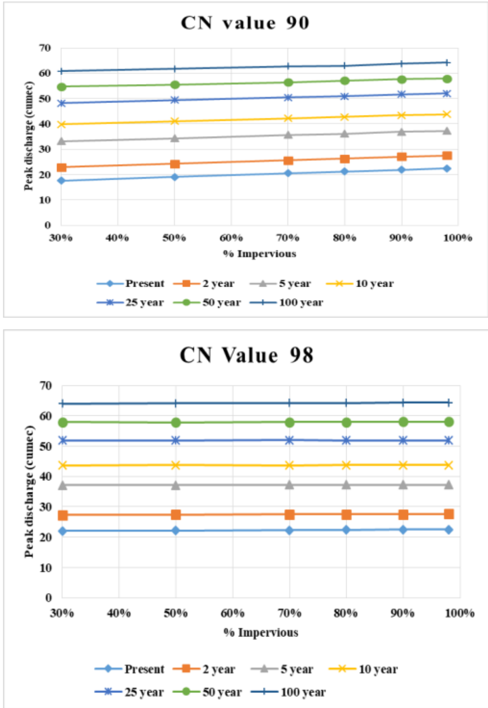


Fig. 8. Variation of peak discharge with different CN values

The CN, represent the change in land use pattern has been considered to be varies from 30 to 98 considered for worst situation. For CN values 30,50,70,80,90,98 in present situation, the total discharge capacity will exceed for % impervious 90%,80%,70%,60%,30% and 30% respectively considering metrological effect only.

Similarly for the return period 2 years the values found 80%,70%,60%,40%,30% and 30% respectively. In this line of context, for 5 years return period the limiting % impervious values would be respectively 50%,40%,30%,30%,30% and 30% and for 10 years return period % impervious values found 50% for CN value 30 and 30% for rest of the CN values. For return period 25 years, 50 years and 100 years it has been found that the discharge will cross the maximum capacity for % impervious values near about 30% only.

Conclution

Rapid growth of urbanization and dense population has great effect on every sector of environment including increased percent impervious areas, surface runoff, decreased vegetation, open space, water quality, and also in physical, chemical and biological disturbance of the watershed of a drainage system. The build-up area increased about 28.49% from the year 1988 to 2018. It can be easily predicted that the build-up areas will increase and reach to above 90% of the total area within short time span if no measures are taken and checked back to ensure sustainable urban planning. Within the 30 years of time span the areas lost its 28.12% of the vegetation and open areas. The trend of change is almost same but in reverse order as compared with build-up area and can be termed as alarming.

The result found that the actual capacity of the canal is 103.85 m^3s^{-1}whereas the average peak tidal height along with inflow has been found 3.75m and corresponding discharge due to tidal effect and inflow has been found 87.34 m^3s^{-1}. The peak discharge would be within the capacity in present condition and also in 2 years return period for CN values up to 30, 40, 50 respectively but for further increased values of CN, it has found to exceed the carrying capacity limit of the canal considering water present in canal for tidal effect. The average values found for 2 and 100 year return periods are 26 $m^3s^{-1}$and 50.5 m^3s^{-1}respectively with respect to different CN value. 90 are 20, 26 $m^3s^{-1}$and 43.47 26 m^3s^{-1} respectively.

City Planners must pay attention on implementaion of SUDs in urban areas in order to control quality, quantity and aminity values of the urban drainage discharge. Moreover soft measures need to be implemented to decrease the runoff discharge from urban areas. Rainwater harvesting, well setup, using porous pavemnt etc. can be implemented to decrease urban runoff and to increase the runoff quality and aminity.

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FLOOD MODELING OF THE SURMA-KUSHIYARA RIVER BASIN USING HEC-HMS

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Abstract

Flood modeling and simulation helps in anticipating flood events thus can minimize flood damage. This study simulated flood occurrences at Surma-Kushiyara river basin which is situated at the north-east part of Bangladesh. Digital elevation model (DEM) of the study areas was used in ArcGIS 10.3 interface to delineate basin, sub-basin and stream networks. With the help of HEC-GeoHMS hydrologic parameters of the river basin such as basin slope, basin area, river length etc were extracted from DEM. These extracted hydrologic parameters were applied to imitate the rainfall-runoff model using HEC-HMS. In this study model has been calibrated for two years and validated for one year. Surma-Kushiyara river basin and upper Surma-Kushiyara river basin has been calibrated for the years of 2009, 2010 and validated for the year of 2011.In addition, to check the efficiency of the results, statistical evaluation was also performed. The study had generated an inclusive description of the basin with decent precision and the R-squared value of the rainfall-runoff model for the Surma-Kushiyara river basin and upper Surma-Kushiyara river basin was found to be 0.93 and 0.79 respectively. This study concludes that rainfall-runoff model using HEC-HMS gives satisfactory result of flood simulation in the Surma-Kushiyara river basin.

Keywords: *Digital elevation model, flood, HEC-GeoHMS, HEC-HMS, Rainfall-Runoff model, statistical evaluation.*

Introduction

Flood indicates a high stage in river and normally the level at which the river overflows its banks and inundates adjoining area (K Subramanya, 2008). Various factors are responsible for this natural incident. Factors include urbanization, river erosion, removal of forest area, insufficient drainage network and systems; however, heavy and continuous rainfall is the main contributing factor. After a rainfall, amount of water that reaches the outlet waterways depends on catchment characteristics. River characteristics and adjacent structures also affect flooding as well as the economic, social and environmental consequences of flood (Garrett, 2011). Therefore, it is important to analyze any flood events and response of catchment to excessive rainfall. Using GIS platform together with rainfall-runoff modeling can help in this regard. GIS uses digital elevation model (DEM) to generate catchment characteristics as well as delineating catchment and determining drainage line which is used as an input parameter in rainfall-runoff model (Ramly and Tahir, 2016).

Among many rainfall-runoff model software programs, HEC-HMS is a widely used program for rainfall-runoff modeling. It gives relationship between runoff from a catchment in response to rainfall in that catchment. HEC-HMS as rainfall-runoff model has been applied to various studies for flood forecasting in various river basins (Knebl *et al.*, 2005; Oleyiblo *et al.*, 2010). Apart from flood forecasting, it can also be used in land use change analysis(Ali *et al.*, 2011) and stream flow analysis (Chu *et al.*, 2009; Zhang *et al.*, 2013).

Bangladesh is situated in low-lying deltaic flood plain of the GBM (Ganges–Brahmaputra–Meghna) river basin. Monsoon rainfall and poor drainage system causes large scale flooding in the country (Winston *et al.*,2010). From June to October heavy monsoon rainfall occurs and annual average rainfall fluctuates from 1200 mm to 5800 mm from west to northeast region of Bangladesh (Rahman *et al.*, 1996). About 20% region of Bangladesh (31,000 km^2) becomes flooded in typical flood year and 80% of the region is regarded flood vulnerable (Mirza, 2002). Floods in 1988, 1998, 2004 and 2007are considered as disastrous and lead to one to two million metric tons of rice loss, or 4–10 % of the yearly rice production (Islam *et al.*, 2009). The northeast hydrological region is one of the depressed part of the Bangladesh consisting principal rivers are the Barak (Surma and Kushiyara), Juri, Manu and Khowai all of which originate from Assam and Meghalaya hilly areas of India. During heavy rainfall in the upland area, water moves quickly towards the southwestern direction through a number of rivers and tributaries and causes flood in haor-featured basin. In 2004 flooding, haor areas

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situated in the northeast side (Netrokona, Sherpur, Sunamganj, Sylhet, Moulvi Bz. &Habiganj) of the country suffered from flash floods that ruined a significant amount of the boro rice crop. The monsoon flooding eventually devastated 36 million people dueling in the northwestern, northeastern and central districts (GOB, 2005).

Northeast region of Bangladesh comprised of hilly areas. As the region is comprised of hilly areas, the terrain is sunken below the surrounding area. Due to lack of proper hydrological data in the northeast depressed zone of Bangladesh, creating a hydrologic model to aid flood forecasting of this region is a troublesome matter.

Objectives of the Study

(a) To model the flood flow of Surma-Kushiyara river basin and doing so a hydrologic model using HEC-HMS has developed.

(b) To calibrate the HEC-HMS model for flood simulation.

(c) To assess the performance of calibrated model in simulation of flood.

Study Site

Surma-Kushiyara river basin is an important river system of north-east part of Bangladesh. It is a transboundary river basin and has an area of approximately 39,009 km^2. It is situated between 89°58'48.908" and 93°2'46.873" east longitudes and in between 23°59'38.816"and 25°44'49.154" north latitudes. The upper Surma-kushiyara river basin which is located within the Surma-Kushiyara river basin is situated between 92°58'58.368" and 91°45'26.386" east longitudes and in between 25°8'28.593" and 24°0'9.382" north latitudes shown in Fig. 1. In this basin, there are 3 rainfall station and 3 discharge measuring gauge station. Surma, Kushiyara, Barak, Juri, Manu, Khowai are some of the major rivers in this basin.

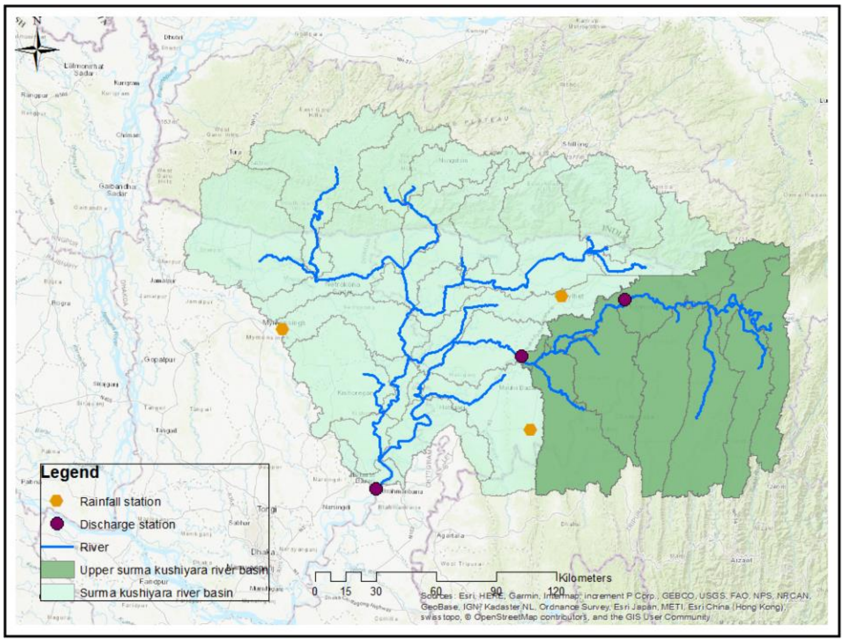

Fig. 1. Location map of the study area

Methodology

Based on the important flood vulnerable areas within the study area and as well as for better simulation of the model, the total study area was divided into two basins namely upper Surma-Kushiyara river basin and Surma-Kushiyara river basin. In order to generate the hydrological model for the study area a 5-steped model was developed. The steps were followed, (i) geographic location of the studied area was obtained from satellite imagery; (ii) various kind of data such as rainfall data, discharge data, water level data were collected and compiled; (iii) using HEC-GeoHMS, useful hydrologic parameters of the basin were extracted from DEM; (iv) extracted hydrologic parameters and processed data were imported to HEC-GeoHMS; (v) incorporating observed data with model simulated data for model calibration and validation.

Data Collection and Processing

SRTM 1 Arc-second global DEM data was used for the study. The data was collected from USGS Earth explorer website. In order to fully cover the catchment area six DEM raster files were collected and merged. Daily rainfall data were collected from Bangladesh Meteorological Department for the three rainfall stations located in Sylhet, Sreemangal and Mymensingh. Daily stage data and weekly discharge data were collected from three discharge stations of Bangladesh Water Development Board which are located in Bhairab bazar, Sherpur and Sheola. Using rating curve daily discharge data were obtained from corresponding daily stage data.

In order to obtain input parameters for hydrologic model setup, at first terrain preprocessing was carried out using HEC-GeoHMS terrain preprocessing platform. Steps for terrain preprocessing were DEM reconditioning, filling sinks, obtaining flow direction, flow accumulation, defining stream, segmentation of stream, delineating catchment grid, converting catchment grid to catchment polygon, processing drainage line and adjoin catchment. After completing terrain preprocessing, physical characteristics such as river length, river slope, basin slope, basin centroid, and longest flow path were calculated using HEC-GeoHMS.

Rainfall-Runoff Model Set Up: HEC-HMS

Rainfall-runoff model set up is done using Hydrologic Engineering center's Hydrologic modeling System (HEC-HMS) which was developed by U.S Army Corps of Engineers. The model simulates hydrologic response in dendritic watershed. In order to, simulate a rainfall-runoff model via HEC-HMS, the HEC-HMS project must have the following components: a basin model, a meteorological model and control specifications. Physical properties of the basin is represented by the basin model. Using HEC-GeoHMS, the physical characteristics of the watershed was developed and it was then imported in HEC-HMS. The meteorological model uses precipitation data, evapotranspiration data and snow melt data for model set up. Both point and gridded data can be used in the meteorological model. In this study, point data was used. Using the three rain gauges data in the study area, Inverse Distance Weighted (IDW) interpolation was used to get the precipitation data for the entire catchment. The control specifications component contains simulation time and time interval for the simulation.

The SCS (soil conservation service) curve number method was used for infiltration loss and the SCS unit hydrograph was used to transform precipitation excess to direct run-off. Constant monthly base flow was used to model the base flow component for the catchment. Finally the Muskingum routing method was adopted to route the reach. In order to best fit observed and simulated hydrograph, HEC-HMS built-in optimization procedure was followed to adjust the parameters so that observed and simulated hydrographs fits precisely. Fig. 2 and Fig. 3 respectively present the HEC-HMS model for upper Surma-Kushiyara river basin and Surma-Kushiyara river basin. The model was calibrated for year 2009 and 2010 and validated for year 2011.

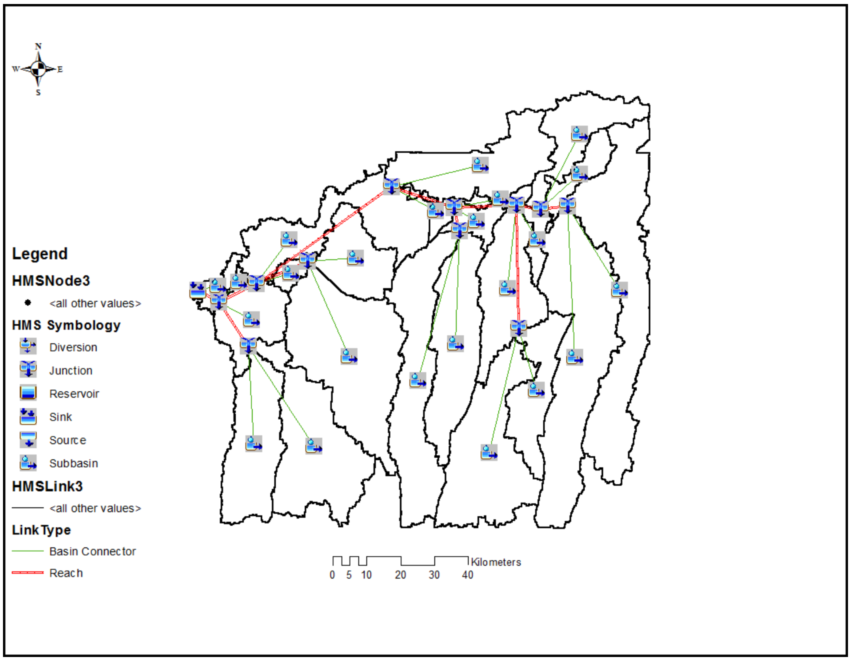

Fig. 2. HEC-HMS model set up for upper Surma-Kushiyara River basin

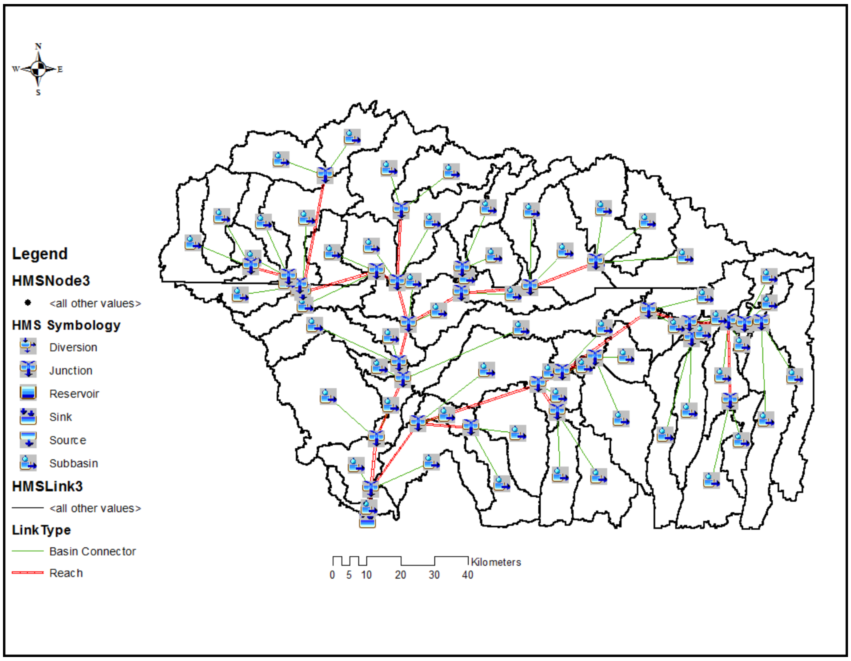

Fig. 3. HEC-HMS model set up for Surma-Kushiyara River basin

Results and Discussion

Calibration of HEC-HMS Model for Flood Simulation

As mentioned in the model set up, the model was calibrated using the data for two years and validated with one year data. Both Surma-Kushiyara river basin and upper Surma-Kushiyara river basin were calibrated for years 2009 and 2010. But due to lack of observed discharge data, simulation of HEC-HMS model for Surma-Kushiyara river basin was done for the months of July to October of 2009 and 2010.The observed and simulated discharge is presented in Fig. 4 and Fig. 5 respectively for the year 2009 and 2010. Similarly, simulation for upper Surma-Kushiyara river basin was done for the months of April to October of 2009 and 2010and similarly presented in Fig. 6 and Fig. 7 respectively. It can be observed from figure 4 that a distinctive simulated and a distinctive observed peak discharge both occurred around the same day. In contrast, multiple peak discharges can be perceived in observed discharge data in Fig. 5, Fig. 6 and in Fig. 7. Model simulated discharge also have multiple peaks and approximately identical to observed discharge data referred in Fig. 5, Fig. 6 and in Fig.7.

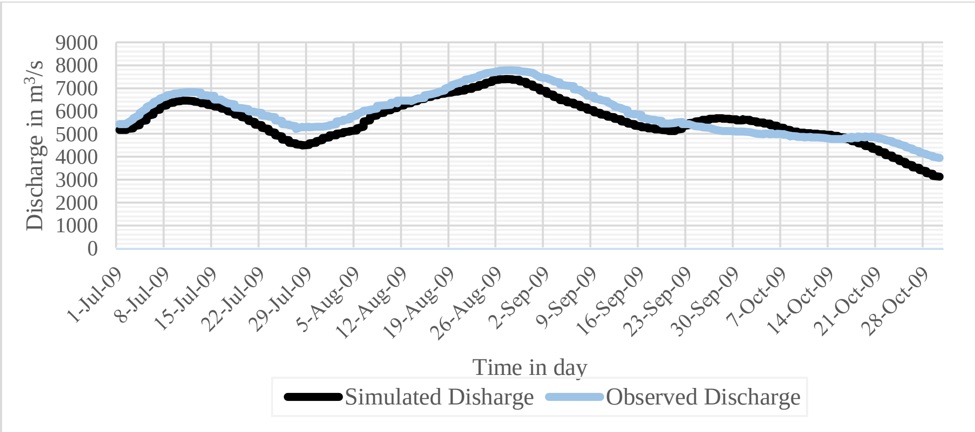

Fig. 4. Model Calibration for Surma-Kushiyara River basin for the year of 2009

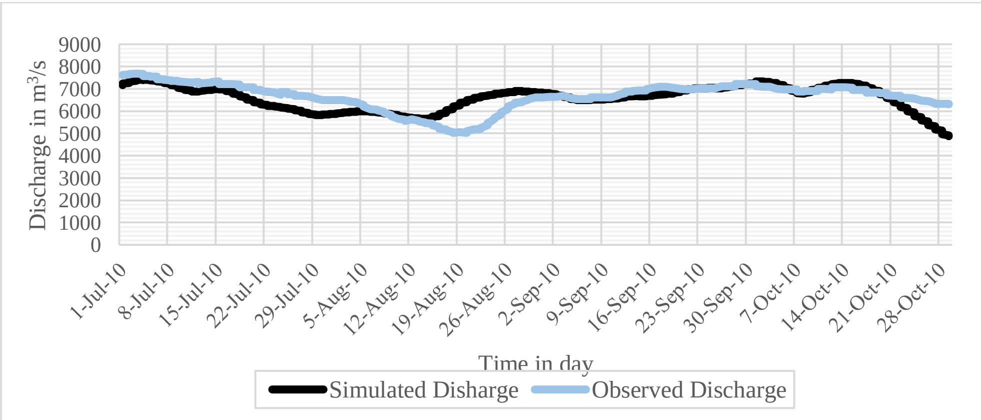

Fig. 5. Model Calibration for Surma-Kushiyara River basin for the year of 2010

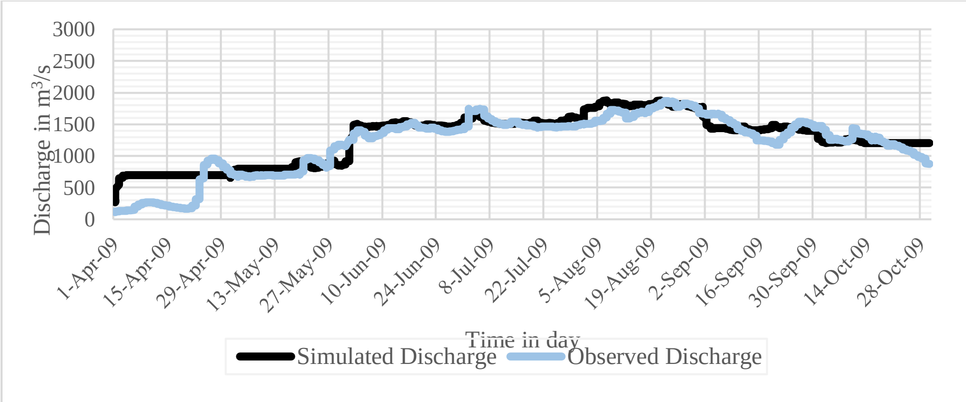

Fig. 6. Model calibration for upper Surma-Kushiyara River basin for the year of 2009

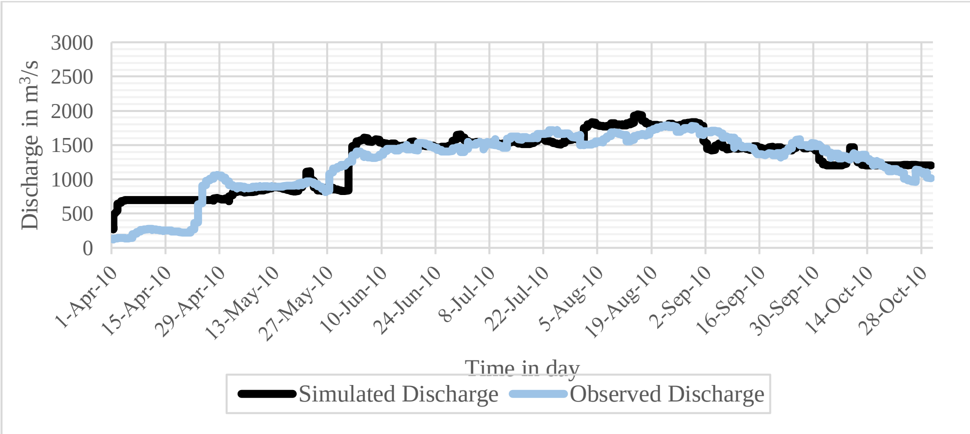

Fig. 7. Model calibration for upper Surma-Kushiyara River basin for the year of 2010

Root mean square error, Nash-Sutcliffe model efficiency coefficient and mean absolute percentage error were calculated (Table 1) using model calibrated results and observed data to showcase the efficiency of the model.

Table 1. Statistical evaluation of the calibrated results

Simulation duration	Basin	Gauge station used for calibration	Root Mean Square Error (RMSE)	Nash-Sutcliffe model efficiency coefficient (NSE)	Mean absolute percentage error (MAPE)
1st July 2009 to 31st October 2009	Surma-Kushiyara river basin	Bhairab Bazar	166.3	0.764	7.44
1st July 2010 to 31st October 2010	Surma-Kushiyara river basin	Bhairab Bazar	205.26	0.527	5.81
1st April 2009 to 31st October 2009	upper Surma-Kushiyara river basin	Sherpur	197.90	0.814	34.48
1st April 2010 to 31st October 2010	upper Surma-Kushiyara river basin	Sherpur	200.2	0.79	30.42

Performance of Calibrated Model in Simulation of Flood for Year 2011

Model validation, which is an essential test for any simulation case, is achieved by applying the model to the second set of data for the period of July to October of 2011 for Surma-Kushiyara river basin and April to October of 2011 for the upper Surma-Kushiyara river basin. The verification process of the model has been achieved by making a comparison between the observed and computed discharge of the basin. Results of the verification process show that the observed and simulated discharge are very much identical to each other as shown in Fig. 8 and Fig. 9.

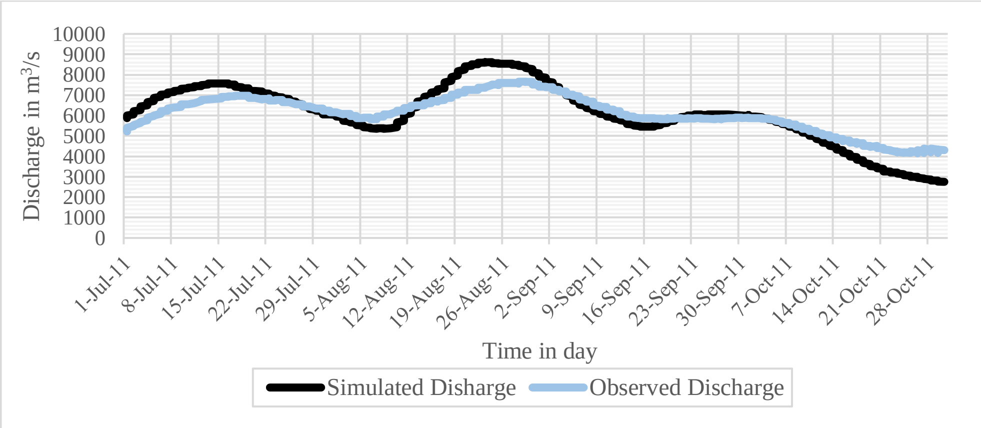

Fig. 8. Model validation for the Surma-Kushiyara River basin for the year of 2011

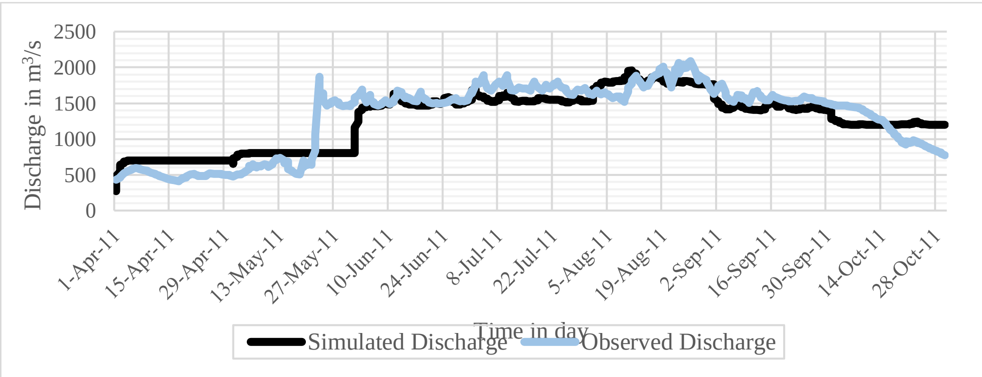

Fig. 9. Model validation for the upper Surma-Kushiyara River basin for the year of 2011

Table 2. Statistical evaluation of the validated results

Simulation duration	Basin	Gauge station used for validation	Root Mean Square Error (RMSE)	Nash-Sutcliffe model efficiency coefficient (NSE)	Mean absolute percentage error (MAPE)
1st July 2011 to 31st October 2011	Surma-Kushiyara river basin	Bhairab Bazar	223.5	0.52	8.69
1st April 2011 to 31st October 2011	upper Surma-Kushiyara river basin	Sherpur	233.04	0.772	18.09

A Nash-Sutcliffe model efficiency coefficient generally ranges from minus infinity to 1. Simulation models which give NSE values ranging from 0 to 1 can be acceptable whereas NSE values closer to 1 represents a more accurate result. In this study, NSE values generated from model predicted discharge and observed discharge both in calibration and validation are above 0.5 (Table 1 and Table 2). Other statistical evaluation such as RMSE and MAPE shows that the results can be acceptable. Another coefficient of determination, the R-squared value of the rainfall-runoff model for the Surma-Kushiyara river basin and upper Surma-Kushiyara river basin was found to be 0.93 and 0.79 which is shown in Fig. 10 and Fig. 11 respectively. The result shows that the rainfall-runoff model gives a well fit simulation and the results can be acceptable.

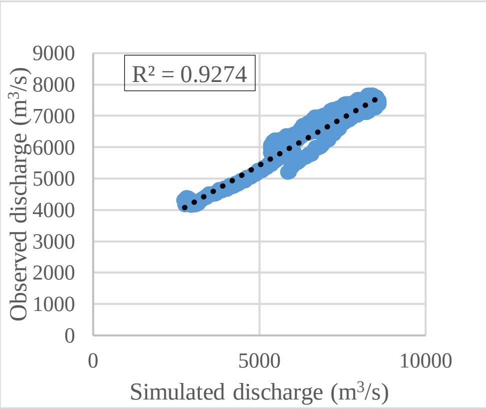

Fig. 10. Correlation between observed and simulated discharge for the Surma-Kushiyara River basin for validation period (year of 2011)

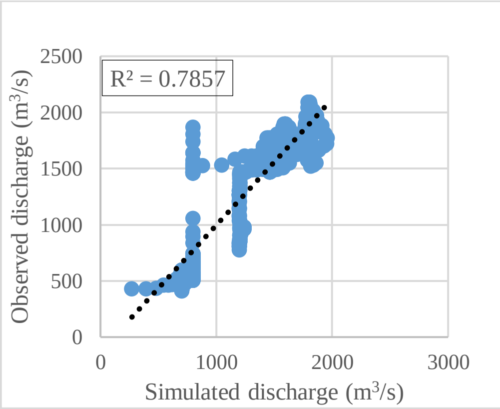

Fig. 11. Correlation between observed and simulated discharge for the upper Surma-Kushiyara River basin for validation period (year of 2011)

Conclusion

From the above results, it is evident that the model simulated peak discharge matches well with the peak discharge of the observed data. Volume of the flood and timing were moderately accurate. Statistical evaluations such as RMSE, NSE, MAPE and R^2 has been done for both the calibration and validation period. Based on the statistical evaluations following conclusions can be summarized:

(a) For the validation period of the year 2011, RMSE value was 223.5 and 233.04 for Surma-Kushiyara river basin at Bhairab Bazar station and upper Surma-Kushiyara river basin at Sherpur station respectively.

(b) Nash-Sutcliffe model efficiency coefficient (NSE) value generated from model predicted discharge and observed discharge in validation period was 0.52 and 0.772 for Surma-Kushiyara river basin and upper Surma-Kushiyara river basin respectively. Both values are less than 1 which shows good efficiency of the model.

(c) MAPE value for the model validation period was found 8.69 and 18.09 for Surma-Kushiyara river basin and upper Surma-Kushiyara river basin respectively.

(d) R-squared value for the model validation period was 0.93 and 0.79 for Surma-Kushiyara river basin and upper Surma-Kushiyara river basin respectively which is very close to 1, shows that the rainfall-runoff model gives a well fit simulation.

Statistical evaluations RMSE, NSE, MAPE and R^2 show that the model predicted the result with good precision. Although there is a scarcity of hydrological data in the Northeast region, the flood simulation model to aid flood forecasting in studied region was developed which can be used to model flood flows of Surma-Kushiyara river basin with good accuracy. Furthermore, it can be said that, with the improvement of data availability in the studied area, the accuracy of the model can be improved and a more accurate simulation can be possible.

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SEDIMENTATION OF HARI-TEKA RIVER OF THE POLDER SYSTEM IN THE SOUTHWEST REGION OF BANGLADESH

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Abstract

The rivers of the Southwest region in Bangladesh carry a large amount sediment to build up the Bengal delta. The process of sediment deposition and erosion is significant for measuring the rate of sedimentation of a tidal river in delta management. Polder system of the Southwest region isolates the tidal river from the floodplain, which hampers natural sediment management by delta development, and advances waterlogging in the floodplain by creating drainage congestion of the river. The study identifies Hari-Teka River of the Southwest region to measure the rate of sedimentation. It follows the sounding pole method to calculate the depths, and cross-sectional area method to estimate the cross-sections of this river at Chahera and Sholgatia of Dumuria upazila under Khulna district, and Kopalia of Manirampur upazila under Jashore district. This research finds that the rate of sedimentation of Hari-Teka River was 0.43 $m^3m^{-2}year^{-1}$ that was annually 29% regarding the total cross-sectional area. This result has policy implications to remove drainage congestion of the river, and decline waterlogging from the floodplain by facilitating Tidal River Management (TRM) for the sustainable delta management.

Keywords: *sedimentation, Hari-Teka River, Polder, waterlogging, Tidal River Management (TRM), Southwest region.*

Introduction

Bangladesh is a riverine country and belongs to the second largest river system in the world: the confluence of the Ganges (Padma), Brahmaputra (Jamuna), and Meghna Rivers (originating from the world's largest mountain range, the Himalayas) and their tributaries flow through the country passing Nepal, China, and India and fall to the Sea Bay of Bengal, evolving into the Bengal Delta (Hellin *et al.*, 2004). This is both a blessing, as it carries the huge nutrient-born sediment that makes agricultural land fertile for good crops, and a curse, as the monsoonal floods in the floodplains of the delta cause disruption to agricultural production, contamination of drinking water, and the displacement of thousands of people (Adri, 2009). There were devastating floods in the Southwest region of Bengal Delta in 1954, 1955, and 1956, which seriously hampered the agriculture and livelihoods of coastal people. The East Pakistan (former name of Bangladesh) government felt the necessity to establish an agency named the East Pakistan Water and Power Development Authority (EPWAPDA) in 1959 in order to manage rivers, control floods, and increase agricultural production by providing proper irrigation facilities (Kibria, 2011; Mutahara, 2018; Masud and Azad, 2018b). EPWAPDA was recommended by the Krug Mission of United Nations, and is presently known as the Bangladesh Water Development Board (BWDB) (PDO-ICZM, 2002; Gain *et al.*, 2017a).

EPWAPDA took on a highly ambitious project (Gain *et al.* 2017a), inspired by the Dutch experience, named the Coastal Embankment Project (CEP). This was mostly financed by the USAID (Tutu, 2005), and involved building high earthen embankments along the tidal rivers of the southern coastline to protect the agricultural land from the tidal influx of saline water as well as from floods and cyclones (Shampa and Paramanik, 2012; Talchabhadel, 2017; Mutahara, 2018). There were several types of drainage structures; for example, regulators and sluice gates (De Die, 2013) were installed across river and canal sections and along the tidal basin in the CEP for managing tidal rivers in the coastal area. BWDB, the government authority, built a coastal embankment in the 1960s and installed 37 polders in the Jashore-Khulna districts under CEP to protect agriculture and settlements from saline water and floods (Gain *et al.*, 2017a; Mutahara, 2018; Masud *et al.*, 2018a). The polder system was effective in safeguarding agricultural land from brackish water due to tidal effects, protecting the coastal community from floods and cyclones (De Die, 2013; Paul *et al.*, 2013), yielding two to three crops per annum, intensifying crop production, and bringing prosperity and joy to coastal people up until the 1980s (Kibria, 2011). However, it was not fruitful (Masud and Azad, 2018b) in the long run due to lack of consideration of the geo-physical settings and hydro-morphological characters of the Southwest region (Roy *et al.*, 2017). The coastal embankment failed to ensure proper

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management of tidal rivers (Gain *et al.*, 2017a, 2017b). As a result, the tidal rivers of the Southwest region have been silted up and are going to dry one after another due to the consequences of CEP. It created troublesome waterlogging in *Beel Dakatia, Beel Bhaina, Beel Khukshia, Beel Buruli, Beel Madhugram, Beel Kopalia, Beel Bakor, Beel Patra, Beel Baruna, Beel Kaderia* and another 20-30 *beels* in the Bhabadaho area (De Die, 2013; Gain *et al.*, 2017a; IWM, 2017). Coastal people were exposed to great danger; they had no experience of waterlogging before the 1980s. Mainly, they had experienced floods in pre-polders, where the flood water returned to the sea within one or two weeks. However, in waterlogging, flood water was still stagnant not only in agricultural land but also in the settlement area for six to eight months up to the pre-monsoon season (Masud and Azad, 2018b). Waterlogging was very destructive; it damaged agriculture, destroyed settlements, degraded the environment, and caused ill health, unemployment, and permanent migration for coastal people.

CEP has been hampering the tidal river-floodplain ecosystem by dismissing sediment management (see Figure 1.2), stopping delta development by adding polders to the floodplain, regulators across the tidal river, and sluice gates for connecting canals with the river since the 1960s, and causing waterlogging from the mid-1980s to the present day.

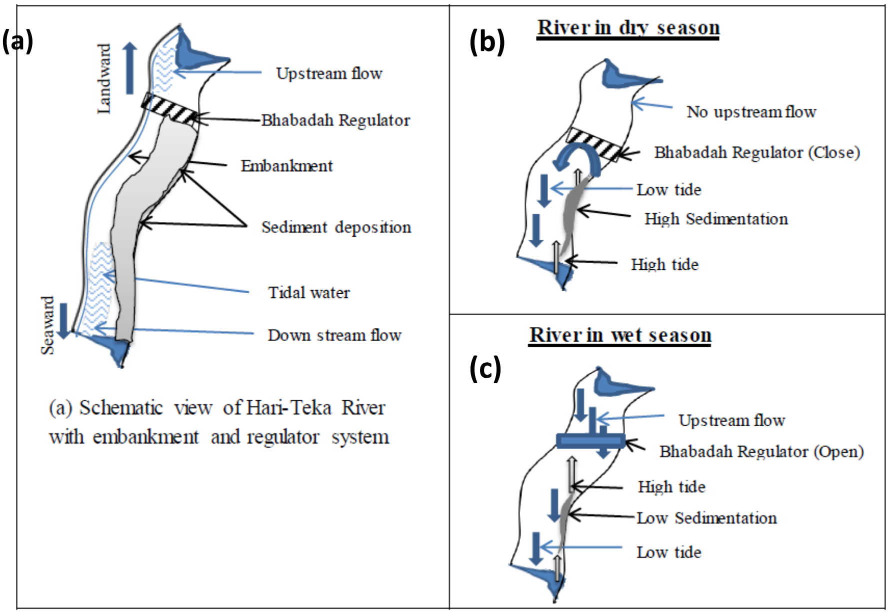

Fig. 1. (a) Aggravation of sedimentation process of tidal river due to CEP, (b) High sedimentation in dry season and (c) Low sedimentation in wet season

Fig. 1 shows that (a) the river was delinked from upstream to downstream by the regulator which reduced upstream flow and increased siltation on river beds, and the embankment restricted over bank flow during monsoon that damaged the sediment balance between tidal river and floodplain; (b) there is no upstream flow during dry season which improves the rate of sedimentation; (c) upstream flow is available during wet season which declines the rate of sedimentation. It is important to ascertain the process of sedimentation and erosion (Deijl*et al.* 2017) for measuring the sediment deposition rate of a tidal river in delta management.

A tidal river has lost its turbulent energy and so suspends sediment in slack tide; this supports deposition for creating a delta (Wright and Schoellhamer, 2005). There are many methods in contemporary literature, e.g. the Bathymetric Survey Method (Takekawa *et al.* 2010), Sediment Coring Method (Jonathan *et al.*, 2001), Sediment Trap Method (Darke and Megonigal, 2003; Deijl *et al.*, 2017), Cross-Sectional Area Method (Masud *et al.*, 2018d) and Sedimentation

Pin Method (Woo *et al.* 2007; Takekawa *et al.* 2010) for observing the sedimentation rate of both the tidal river and tidal basin. This research followed the cross-sectional area method to measure the sedimentation rate of the Hari-Teka River. Therefore, this research fixes two objectives mentioned as below:

i. To measure the cross-section of the Hari-Teka River and
ii. To estimate the rate of sedimentation.

Methodology

Sediment deposition is the settled down of suspended sediment or particles (sand, silt, clay, organic and inorganic materials) to the river bed. This settling often occurs when water flow slows down or stops. The high rate of sediment deposition is an environmental concern for tidal river-floodplain ecosystem.

(i) *Cross-sectional area method for estimating the rate of sediment deposition*

In this research, the rate of sediment deposition was estimated by measuring cross-sectional area. The cross-sectional area is calculated by the following formula (Subramanya, 1988):

$$A = \Delta A_1 + \Delta A_2 + \text{-----} + \Delta A_n \quad \text{-------------} \ (1.1)$$

The cross-sectional area of the river (see Figure 4.2) is segmented into 10 to 15 segments (depend on its width).

Area of the first segment and the last segment for the cross-sectional area of a River has made triangular forms and these are measured by the following equations:

$\Delta A_1 = \bar{W}_1 \, y_1$

Average width of (W_1 and W_2), $\bar{W}_1 = \frac{\left(W1+\frac{W2}{2}\right)^2}{2W1}$

Here, ΔA_1, y_1, W_1 are stand for area, depth and width respectively for the first segment and W_2 stands for the width of the second segment.

$\Delta A_n = \bar{W}_{n-1} y_{n-1}$

Average width of (W_n and W_{n-1}),

$\bar{W}_{n-1} = \frac{\left(\mathrm{Wn}+\frac{Wn-1}{2}\right)^2}{2Wn}$

Here, y_{n-1}, W_{n-1} are stand for depth and width respectively of (n-1)th segment and ΔA_n, W_n stand for area and width of nth segment.

Area of the rest segments (from ΔA_2 to ΔA_{n-1}) are measured by the following equations:

$\Delta A_i = \bar{W}_i y_i$

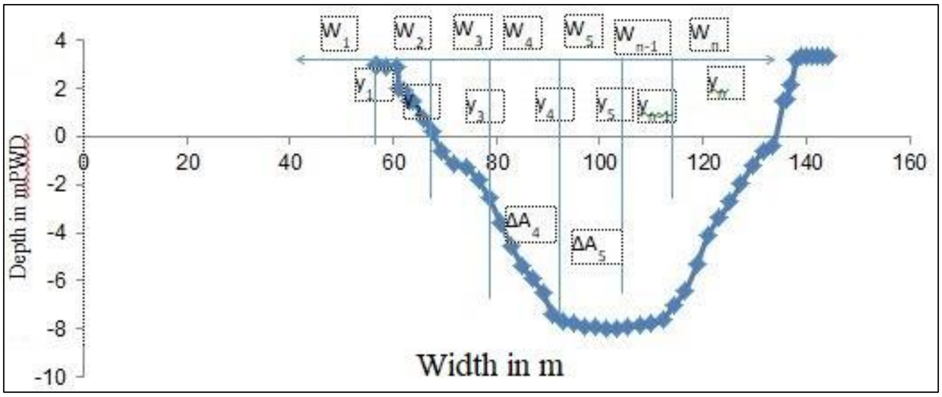

Fig. 2. The cross-section of a river

Note: 'PWD' means Public Works Datum

A_{f1}, A_{f2} and A_{fn} as well as A_{l1}, A_{l2} and A_{ln} are the cross-sectional area for n places of a river for the first time and last time respectively of a time period in m^2

L = the assumption length is 1 m for the distance between two places within the river channel in m,

D_t = Total no. of days are required for sediment deposition,

Y_t = Total no. of years are required for sediment deposition.

A rope connected two marking points on the shore (one was placed in the left bank and another was placed in the right bank) across the channel.

The starting point of the rope on the shore was marked as a reference point. The width of the river was divided into several segments by the rope and tags. Then the depth (value for Y axis) was measured by sounding pole for every width of the known distance (value for X axis). A boat was used to conduct the depth measurements. After that, the cross-sectional area was calculated by the values of both axes.

The depths of the river were measured using the sounding pole method during low tide. In basic terms, the water depth was reduced by 1.5m to 2.5m during neap tide of the tidal river. Therefore, it was easy to measure the accurate depths during low tide. First, the lower part (under the surface water) was estimated; after that, the upper part was measured by sounding pole, and then the depths were calculated by integrating the results of both parts.

Selection of the study area

Southwest coastal Delta of Bangladesh is located near the Bay of Bengal where the Ganges-Brahmaputra Rivers originated from Himalaya Mountain flow into Bangladesh (Kibria, 2011). Its tributaries (Modhumati River, Vairab River, Rupsha River, Pashur River, Kobadak River, Sholmari River, Bhadra River, Hamkura River, Hari River, Gengrile River, Hari-Teka River, Mukteshawri River, Teligati River, Betna River, Moricchap River) that carry a huge siltation (Uttaran, 2013) are flowed over this region and mingled with the sea (Masud *et al.*, 2018a). Anthropogenic and structural initiatives for instance, Farraka barrage since 1975 includes reduced freshwater flow during dry season due to the upstream withdrawal of water (Shampa and Paramanik, 2012; De Die, 2013; GED, 2015; Gain *et al.*, 2017a; Roy *et al.*, 2017). Besides, the coastal polders have prevented silt from the rivers from being deposited on floodplains, causing high rates of sedimentation on the river bed and resulting huge drainage congestion (De Die, 2013; Gain *et al.*, 2017a; 2017b). Therefore, this coastal region has been undergoing water problems and adopting several methods for water resources management since the last quarter of 20th century (Masud *et al.*, 2018a; 2018d).

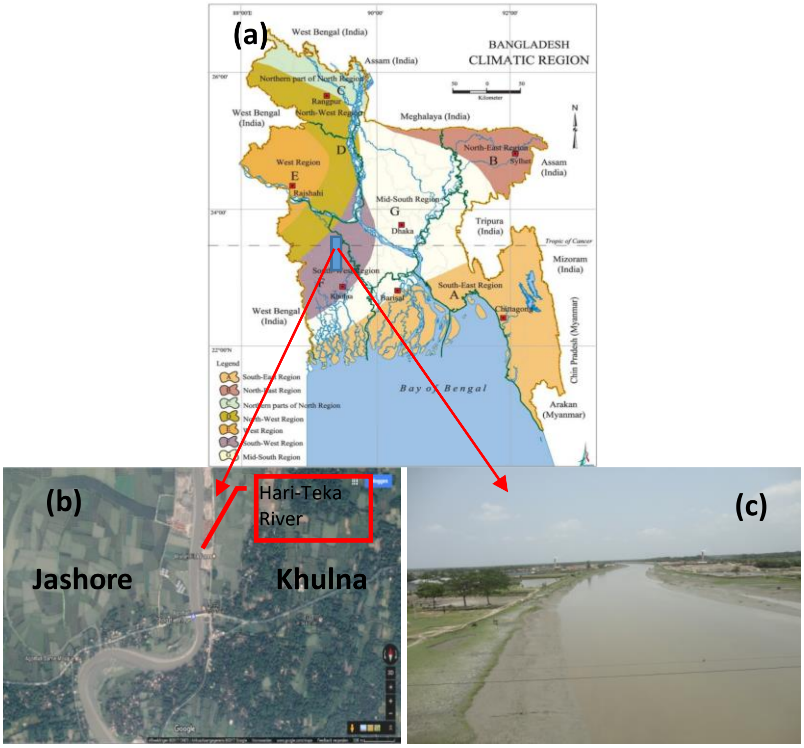

Fig. 3. (a) section F is the Southwest region of Bangladesh (b) Hari-Teka River (Source: Google) of the South west region (c) Hari-Teka River flows to the upstream at Sholgatia bridge, Dumuria, Khulna

The study selected Hari-Teka River (see Fig. 3) which is important for Bhabadah area to drain out excess water during monsoon from 52 *beels* (EGIS, 1998) of the Jashore and Khulna districts. This river is connected with the lower Bhadra River and Teligati River at downstream, and Mukteshawri River and Bhairab River at upstream. Bhabadah area has been facing waterlogging problems since the 1980s. There three TRMs (Tidal River Management) i.e. *Beel Bhaina*-TRM, *Beel Kaderia*-TRM and *Beel East Khukshia*-TRM have been implemented through Hari-Teka River in the first age of the 21th century to solve waterlogging problems of the Bhabadah area (Mutahara, 2018; Masud *et al.*, 2018d).

Therefore, this research selected three locations i.e. Chahera and Sholgatia of Dumuria upazila under Khulna district and Kopalia of Manirampur upazila under Jashore district (from downstream to upstream) of the Hari-Teka River to measure the cross-sectional areas in April, August and December of 2018 and estimate the rate of sedimentation without TRM intervention under polder.

Results and Discussion

(i) Measurement of the depths of tidal river

The study measured depths (see Fig. 4) regarding the distance from East bank of Hari-Teka River at Chahera, Sholgatia and Kopalia in different months of 2018 by sounding pole method. Land elevation of the bank shore for every location was 3 m. Firstly, we measured the depth in m then it converted into m PWD.

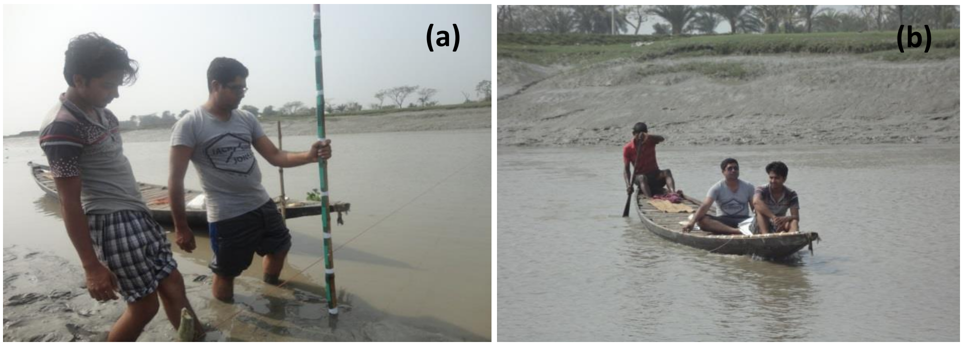

Fig. 4. (a) Measuring the depth of Hari-Teka River at Chahera of Dumuria upazila under Khulna district (b) the boat was used for measuring cross-section of Hari-Teka River.

Table 1. The depths of Hari-Teka River at Chahera in 2018

April' 2018		August' 2018		December' 2018	
Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)
0	3.06	0	2.96	0	3.11
1	2.67	1	2.72	1	3.08
4	2.53	4	2.62	4	2.87
6.3	2.37	7	2.4	7	2.65
9.3	2.13	10	2.12	10	2.47
13.8	1.62	13	1.79	13	2.34
16.8	1.27	16	1.41	16	2.13
18.3	0.59	19	0.75	19	1.77
21.3	0.36	22	0.47	22	1.56
24.3	0.02	25	0.24	25	1.39

April' 2018		August' 2018		December' 2018	
Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)
27.3	-0.22	28	0.04	28	0.84
30.3	-0.43	31	-0.17	31	0.26
33.3	-0.54	34	-0.26	34	-0.07
36.3	-0.59	37	-0.33	37	-0.49
39.3	-0.89	40	-0.51	40	-0.57
43.3	-1.08	43	-0.61	43	-0.72
45.8	-0.81	46	-0.29	46	-0.65
48.8	0.12	49	0.89	49	-0.11
51.8	1.53	52	1.73	52	1.37
54.8	2.32	55	2.44	55	2.99
56.3	3.1	56.2	2.99	56	3.19

Source: Field survey, 2018

Table 1 presents the width of Hari-Teka River was 56.3 m, 56.2 m and 56 m and the maximum depth was -1.08 m PWD at 43.3 m distance, -0.61 m PWD at 43 m distance and -0.72 m PWD at 43 m distance from East bank of the river respectively on 21 April, 14 August and 27 December in 2018 at Chahera.

Table 2. The depths of Hari-Teka River by distance from West bank at Sholgatia

April' 2018		August' 2018		December' 2018	
Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)
0	3.06	0	3.01	0	2.76
1	2.25	1	2.37	2	2.55
4	1.75	4	1.91	5	0.66
7	0.77	7	1.02	8	-0.44
10	-0.23	10	-0.02	11	-0.51
13	-0.86	13	-0.56	14	-0.9
16	-1.52	16	-1.05	17	-1.4
19	-2.51	19	-1.97	20	-1.64
22	-2.12	22	-1.63	23	-1.03
25	-1.83	25	-1.19	26	-0.55
28	-1.34	28	-0.96	29	0.6
31	-0.75	31	-0.34	32	1.39
34	-0.21	34	0.51	35	2.13
37	0.68	37	1.47	38	2.42
40	1.27	40	2.16	41	2.44
43	2.15	43	2.61	44	2.89
46	2.37	46	2.67	46	2.83
49	2.29	49	2.55	48	2.78
52	2.45	52	2.57	50	2.8
53	2.99	52.9	2.95	52.5	2.84

Source: Field survey, 2018

Table 2 presents the width of Hari-Teka River was 53 m, 52.9 m and 52.5 m and the maximum depth was -2.51 m PWD at 19 m distance, -1.97 m PWD at 19 m distance and -1.64 m PWD at 20 m distance from West bank of the river respectively on 21 April, 14 August and 27 December in 2018 at Sholgatia.

Table 3. The depths of Hari-Teka River regarding distance from South bank at Kopalia

April' 2018		August' 2018		December' 2018	
Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)	Distance from bank (m)	Depth (m PWD)
0	3.26	0	3.19	0	2.86
1.1	2.71	1.2	2.96	0.5	2.78
2.6	2.36	4.2	2.29	3.5	2.49
5.6	1.89	7.2	1.76	6.5	2.57
8.6	1.17	10.2	0.86	9.5	2.54
11.6	0.55	13.2	0.9	12.5	1.93
14.6	0.18	16.2	0.26	15.5	1.01
17.6	-0.79	19.2	-0.82	18.5	-0.01
19.1	-1.32	22.2	-0.89	21.5	-0.73
20.6	-1.42	25.2	-0.95	24.5	-0.85
22.1	-1.48	28.2	-0.77	27.5	-0.51
25.1	-1.57	31.2	-0.44	30.5	0.74
27.5	-1.34	34.2	0.58	33.5	1.25
28.3	-1.17	37.2	1.64	36.5	1.94
31.3	-0.79	38.7	2.38	39.5	3.05
34.3	0.36	39.2	2.87		
35.8	0.82	40	3.12		
37.3	1.39				
38.8	2.52				
40.3	3.11				

Source: Field survey, 2018

Table 3 exhibits that the width of Hari-Teka River was 40.3 m, 40 m and 39.5 m and the maximum depth was -1.57 m PWD at 25.1 m distance, -0.95 m PWD at 25.2 m distance and -0.85 m PWD at 24.5 m distance from South bank of the river respectively on 22 April, 14 August and 28 December in 2018 at Kopalia.

(ii) Measurement of the cross-sectional area of tidal rivers

The study measured cross-sectional area by the widths and depths for every location following equation (1.1). Cross-sectional area was reduced with progressing time for Hari-Teka River due to sedimentation. The rate of sedimentation was estimated from the changes in cross-section of a time period following equation (1.2).

Table 4. The cross-sectional area of Hari-Teka River at Chahera on April

X (Distance)	Y (Depth in m PWD)	Width (m)	Depth (m)	Average Width (m)	Area (m^2)
0	3.06	W_1=4	y_1= 0.53	$\bar{W}_1$= 4.02	ΔA_1= 2.1
4	2.53	W_2= 6.8	y_2= 1.03	$\bar{W}_2$= 6.4	ΔA_2= 6.6
10.8	2.03	W_3= 6	y_3= 1.79	$\bar{W}_3$= 6	ΔA_3= 10.7
16.8	1.27	W_4= 6	y_4= 2.91	$\bar{W}_4$= 6.75	ΔA_4= 19.6
22.8	0.15	W_5= 7.5	y_5= 3.49	$\bar{W}_5$= 6.75	ΔA_5= 23.6
30.3	-0.43	W_6= 6	y_6= 3.65	$\bar{W}_6$= 6.5	ΔA_6= 23.7
36.3	-0.59	W_7= 7	y_7= 4.14	$\bar{W}_7$= 5.5	ΔA_7= 22.8

X (Distance)	Y (Depth in m PWD)	Width (m)	Depth (m)	Average Width (m)	Area (m^2)
43.3	-1.08	W_8= 4	y_8= 3.45	$\bar{W}_8$= 3.5	ΔA_8= 12.1
47.3	-0.35	W_9= 3	y_9= 2.35	$\bar{W}_9$= 3	ΔA_9= 7.1
50.3	0.75	W_{n-1}= 3	y_{n-1}= 1.25	$\bar{W}_{n-1}$= 3.38	ΔA_{n-1}= 4.2
53.3	1.85	W_n= 3	y_n= 0		
56.3	3.1				A= 133 m^2

Source: Calculation based on field survey data, 21 April 2018

Table 4 shows that the cross-section of Hari-Teka River was segmented into 11 segments. Then the cross-sectional area was measured following equation (1.1) by the summation of small areas of the segments. The area was 133 m^2 at Chahera on 21 April 2018. Similarly, the cross-section of Hari-Teka River was measured on 14 August and 27 December in 2018, and it was respectively 115 m^2 and 108 m^2 at Chahera.

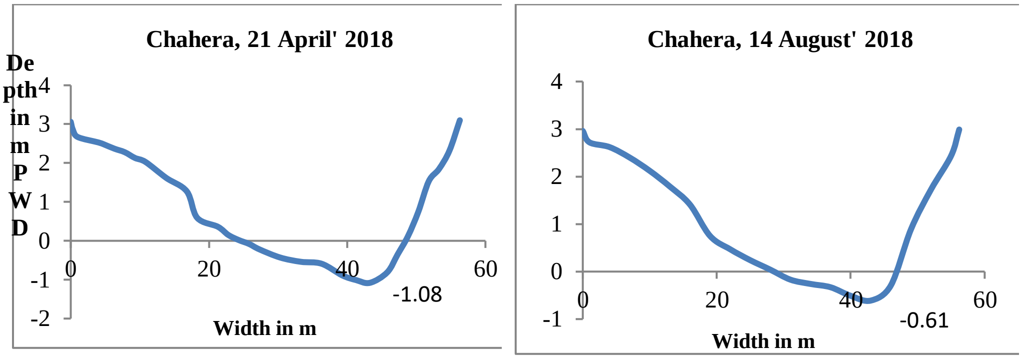

Fig. 5. Cross-sections of Hari-Teka River at Chahera

Fig. 5 presents the width was reduced by 0.1 m and the depth was reduced by 0.47 m from April to August in 2018.

Table 5. The cross-sectional area of Hari-Teka River at Sholgatia on April

X (Distance)	Y (Depth in m PWD)	Width (m)	Depth (m)	Average Width (m)	Area (m^2)
0	3.06	W_1= 4	y_1= 1.31	$\bar{W}_1$= 4.1	ΔA_1= 5.4
4	1.75	W_2= 6	y_2= 3.29	$\bar{W}_2$= 6	ΔA_2= 19.7
10	-0.23	W_3= 6	y_3= 4.58	$\bar{W}_3$= 4.5	ΔA_3= 20.6
16	-1.52	W_4= 3	y_4= 5.57	$\bar{W}_4$= 4.5	ΔA_4= 25.1
19	-2.51	W_5= 6	y_5= 4.82	$\bar{W}_5$= 6	ΔA_5= 28.9
25	-1.83	W_6= 6	y_6= 3.74	$\bar{W}_6$= 6	ΔA_6= 22.4
31	-0.75	W_7= 6	y_7= 2.31	$\bar{W}_7$= 6	ΔA_7= 13.9
37	0.68	W_8= 6	y_8= 0.84	$\bar{W}_8$= 6	ΔA_8= 5
43	2.15	W_{n-1}= 6	y_{n-1}= 0.7	w_{n-1}= 8	ΔA_{n-1}= 5.6
49	2.29	W_n= 4	y_n= 0		
53	2.99				A= 147 m^2

Source: Calculation based on field survey data, 21 April 2018

Table 5 provides that the cross-section of Hari-Teka River was segmented into 10 segments at Sholgatia. The area was 147 m^2 at Sholgatia on 21 April 2018. Similarly, the cross-section of Hari-Teka River was measured on 14 August and 27 December in 2018, and it was respectively 122 m^2 and 104 m^2 at Sholgatia.

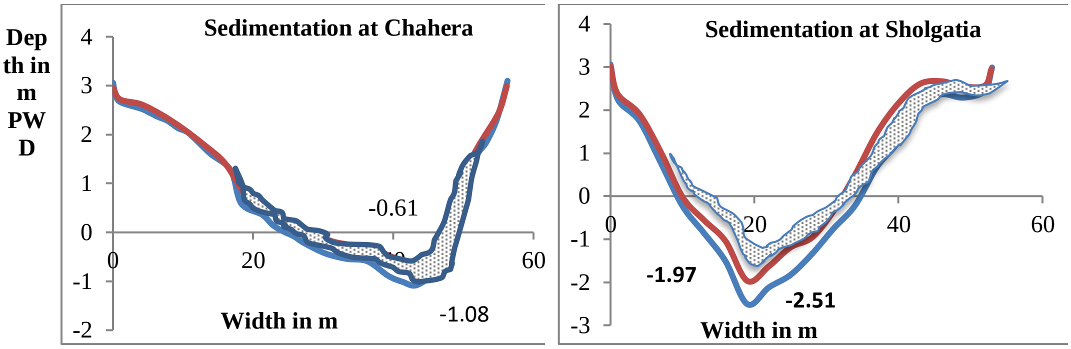

Fig. 6. Sedimentation of Hari-Teka River at Chahera

Fig. 6 demonstrates the sedimentation of Hari-Teka River at Chahera and Sholgatia from April to August in 2018. The cross-sectional area was declined by 18 m^2 and 25 m^2 respectively at Chahera and Sholgatia.

Table 6. The cross-sectional area of Hari-Teka River at Kopalia on April

X (Distance)	Y (Depth in m PWD)	Width (m)	Depth (m)	Average Width (m)	Area (m^2)
0	3.26	W_1= 4.1	y_1= 1.22	$\bar{W}_1$= 4.2	ΔA_1= 5.1
4.1	2.04	W_2= 6	y_2= 2.72	$\bar{W}_2$= 6	ΔA_2= 16.3
10.1	0.54	W_3= 6	y_3= 3.41	$\bar{W}_3$= 6	ΔA_3= 20.5
16.1	-0.15	W_4= 6	y_4= 4.74	$\bar{W}_4$= 4.5	ΔA_4= 21.3
22.1	-1.48	W_5= 3	y_5= 4.83	$\bar{W}_5$= 3.1	ΔA_5= 15
25.1	-1.57	W_6= 3.2	y_6= 4.28	$\bar{W}_6$= 3.1	ΔA_6= 13.3
28.3	-1.17	W_7= 3	y_7= 3.9	$\bar{W}_7$= 3	ΔA_7= 11.7
31.3	-0.79	W_8= 3	y_8= 2.75	$\bar{W}_8$= 3	ΔA_8= 8.25
34.3	0.36	W_{n-1}= 3	y_{n-1}= 1.72	w_{n-1}= 3.4	ΔA_{n-1}= 5.8
37.3	1.39	W_n= 3	y_n= 0		
40.3	3.11				A= 117 m^2

Source: Calculation based on field survey data, 22 April 2018

Table 6 shows that the cross-section of Hari-Teka River was segmented into 10 segments at Kopalia. Then the cross-sectional area was measured by the summation of small areas of segments. The area was 117 m^2 at Kaplia on 22 April 2018. Similarly, the cross-section of Hari-Teka River was measured on 14 August and 28 December in 2018, and it was respectively 99 m^2 and 70 m^2 at Kopalia.

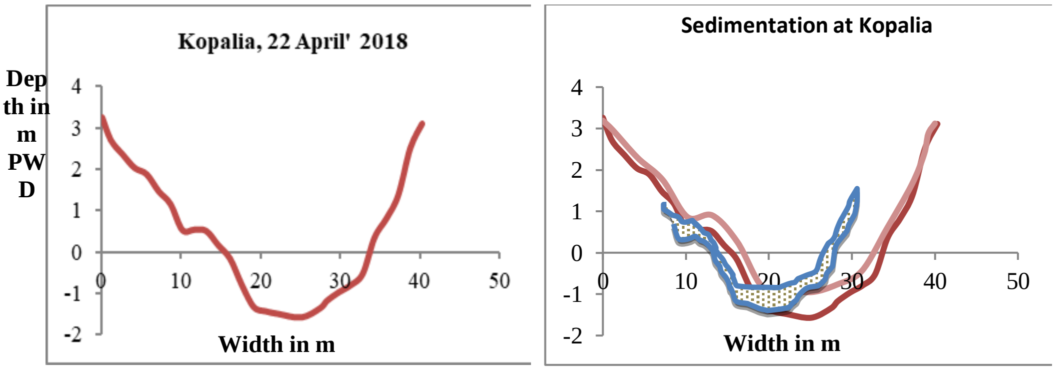

Fig. 7. Sedimentation of Hari-Teka River at Kopalia

Fig. 7 illustrates the sedimentation of Hari-Teka River at Kopalia from April to August in 2018. The cross-sectional area was declined by $18m^2$ at Kopalia. Estimation of the rate of sedimentation

The study calculates the reduction of cross-sectional area due to sediment deposition on the riverbeds for Chahera, Sholgatia and Kopalia of Hari-Teka River from April to December in 2018. Then it estimates the rate of sedimentation from the mean cross-section of these locations.

Table 7. The rate of sedimentation at different locations of Hari-Teka River

Location	Measurement of Cross-sectional area (m^2)			Reduction of Cross-section (m^2)	No. of days	Rate of Sedimentation ($m^3m^{-2}day^{-1}$)
	April	August	December			
Chahera	133	115	108	25	250	0.00075188
Sholgatia	147	122	104	43	250	0.001170068
Kopalia	117	99	70	47	250	0.001606838

Note: 21 April to 27 December = 250 days

Table 7 illustrates the sedimentation of Hari-Teka River at Chahera, Sholgatia and Kopalia from April to December in 2018. The rate of sedimentation was 0.00075 $m^3m^{-2}day^{-1}$, 0.00117 $m^3m^{-2}day^{-1}$ and 0.00161 $m^3m^{-2}day^{-1}$ for Chahera, Sholgatia and Kopalia respectively.

Therefore, by following equation (2.2) the sedimentation rate of Hari-Teka River was 0.00118 $m^3m^{-2}day^{-1}$ or 0.43 $m^3m^{-2}year^{-1}$. Besides, the annual sedimentation rate regarding cross-sectional area was 43%. It means that 43% of total cross-sectional area was silted up from Chahera to Kopalia (around 7 km) due to sedimentation of Hari-Teka River. For this reason, it creates drainage congestion which leads to waterlogging in the Bhabadah area.

Conclusion

TRM is considered as an eco-technical and indigenous management practices which allow temporary open the polder (by cutting a small section of the embankment) to connect the tidal river in to the floodplain for sediment management, delta development and removal of waterlogging in the Southwest region (Nowreen et al., 2014; van Staveren et al., 2017; IWM, 2010, 2017; Masud et al., 2018a). In this study, for the first time, we provide a quantitative assessment on sedimentation of Hari-Teka River by applying innovative methods such as cross-sectional area method with involving sounding pole method. After ending *Beel East Khukshia*-TRM in 2012, Bhabadah area went under close polder system again to progress sedimentation of the Hari-Teka River although there were several excavation projects to the upstream (near Bhabadah regulator) in 2017-18 and 2018-19 fiscal years. The renovation works from the government authority (BWDB) do not stop sedimentation of the Hari-Teka River. This research finds that the maximum depth was reduced from -1.08 m PWD to -0.72 m PWD, -2.51 m PWD to -1.64 m PWD and -1.57 m PWD to -0.85 m PWD, and the cross-sectional area was declined by 25 m^2, 43 m^2 and 47 m^2 at Chahera, Sholgatia and Kopalia respectively. The rate of sedimentation of Hari-Teka River was 0.43 $m^3m^{-2}year^{-1}$ and it was advanced from downstream (Chahera) to upstream (Kopalia). The result of annual sedimentation was 29% regarding the total cross-sectional area remarks that the river will be totally silted up (ending tidal flow) from 4 to 5 years if this sedimentation continues without TRM operation. Although the sounding pole method has few limitations regarding placing the pole rightly to measure the depth due to strong current of the river and monitoring the marking points (bamboo sticks) of the cross-section for a long period (for 8 months), it is easy and inexpensive method to provide accurate results for the shallow river.

Therefore, TRM is inevitable for sustainable delta management to solve waterlogging and drainage congestion problems in the Southwest region. In this study, we only assess the sedimentation of Hari-Teka River. Further research is needed to assess the benefits of TRM operation.

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A STUDY ON THE OPTIMIZATION OF DREDGING ALIGNMENT, PROPOSED RIVER TRAINING AND BANK PROTECTION WORKS AROUND JAJIRA-NARIA

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Abstract

This paper presents the results for finalization of the proposed dredging alignment & design, performance of the bank revetment for bank protection and effectiveness of the required river training works for land reclamation in the Padma river around Jajira & Naria upazila under Shariatpur district using scale modeling. The river reach of 36.0 km of Padma upstream of Padma-Meghna confluence at Chandpur and the river reach of 15km upstream & 5km downstream of confluence along the Meghna have been reproduced in this study. Different application test are conducted with different test scenarios (introducing revetment, dredged channel & T-head groynes) using different discharges. The total length of dredged channel is 11.8 km. It was found that dredged channel is mostly found to be silted up. It is also found that the dredged channel is partially silted up at average annual discharge (30,000 cumec) but at 100-year return period discharge (1,30,000 cumec), it is almost fully silted up. Therefore, if it is intended to maintain the proposed dredged channel, regular maintenance dredging would be required. At the end of the test, the percentage of siltation on average is measured about 70% fully silted of its length (at the upstream) and the rest part partially silted (at the downstream). The proposed revetment (9.35km) tested in different tests works well and it is recommended to implement in the field. It is necessary to implement the bank revetment in the field immediately for the protection of the problem area and to prevent the bank erosion in the coming year. The location, alignment, dimension and spacing of groynes tested in the recommended test is suggested to implement in the field as it provides better result. The proposed groynes are expected to divert the flow towards the mid-stream and to facilitate land reclamation by sedimentation between the groynes.

Keywords: *Padma river, bank protection, dredging, optimization, river training works and land reclamation.*

Introduction

The Padma carries immense volumes of water and constantly shifting its main channel due to the emergence of chars (sand bars) and islands at different locations of the river near Jajira and Naria upazilla in Shariatpur district. Apart from these, it has been eroding vast areas on one bank due to the collective effects of huge current, wave, tidal influences and upstream torrents. The outer (concave) bank of the river is gradually advancing towards the country side and the bend is becoming gradually sharper. This process of bank erosion is typical to the Padma and other major rivers of Bangladesh.

Recently severe bank erosion occurred at Naria. upazilla health complex, bazar, mosque, educational institutions, important roads, homestead etc. have already been engulfed by the river. The Kundeshwar, Sureshwar Launch Ghat Terminal and Chandipur bus stand area are also vulnerable to massive erosion of the Padma river. As a consequence, erosion affected people have been compelled to take shelter elsewhere losing their ancestral homes. Furthermore, education is hampered and hence, poverty is intensified in that region making the people unhappy, upset and frustrated.

Scale modeling is a tool to investigate the hydraulic and morphological impacts of any kind of intervention into a river system or on its floodplains. This tool has been widely used in the field of river engineering to support river management in terms of flood management, bank erosion management and sediment management as well as to provide decision support in optimal planning and design of different water infrastructures.

River dredging has often been employed to manage sediment for keeping the river dynamic and navigable. The overall objective of the physical model study is to investigate the efficacy of dredging options along Sureshwar and neighboring areas under Jajira and Naria upazilla in Shariatpur district and to investigate the hydraulic and morphologic effects of the dredging in relation to changes in flow field, sedimentation and river bank erosion.

Methodology

An overall morphological model investigation is carried out in order to achieve the study objectives. The study area is located at the dynamic Padma-Meghna confluence which

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influences the morphological developments in the study area. Moreover, it appears from the satellite images that the river stretch between Mawa and Chandpur may be considered as a reach where river pattern is braided. Therefore, the model extent should cover a river stretch between Mawa and Chandpur together with a river stretch of the Lower Meghna river starting from the confluence. The length of river reproduced in the overall distorted morphological model is tentatively about 36km along Padma upstream of its confluence with Meghna; about 15km upstream & 5km downstream of confluence along the Lower Meghna.In this scale model, various types of instrument and facilities are needed such as, a sharp-crested weir for measuring flow, point gauge for measuring water level, 3-D current meter for measuring velocity, high resolution camera for taking video and photographic view of model, stopwatch for taking instant time and plastic colored balls (floats) for tracing flow path of flowing water. The discharge in the model is measured using sharp-crested weir at the inflow section using Rebock's formula. Model velocity is quantified by current meter. Water slope can be found by analyzing the water level measurements of different point gauges installed in the model. Flow lines of the stream can be identified by dropping colored balls starting from calibration section and catching them at the end of the model. A stopwatch can be used to calculate surface velocity of the flow. In the scale model, model data requires to be analyzed for interpretation of test results. The initial bathymetry of the model is reproduced based on the collected field survey data collected. The model is calibrated on the basis of prototype water levels, flow velocities and sediment transport data. Manual sediment feeding is done with a view to assess the required rate of sediment feeding during the model run. Continuous monitoring of the model bed is done by taking soundings of the model bed.

Model Setup

An open-air model bed having dimension 100m × 80m of RRI has been selected for overall morphological model development. It provides all kinds of facilities related to model study. Then layout of model is given by grid system. After setting reference grid points in the model, channel planform is given using these grid points and the bed & bank levels are fixed up by levelling instrument as per bathymetry using Rise & Fall method. This requires some cutting and filling of sand from the model bed.

The model is investigated on a mobile bed. The hydraulic similarity is established in the model to a distorted scale. The model is a Froude model and is studied over bathymetry of October/2018. The model is constructed with horizontal scale of 1:600 and vertical scale of 1:80. The model has been designed to fulfil both flow and sediment transport criterion simultaneously. It means the model velocity is higher than the critical flow velocity for the initiation of sediment motion. This is because for any velocity higher than the critical, the scour dimensions are only function of flow direction and structure geometry. The model will, therefore, reproduce the scour holes correctly. The model layout is shown in **Fig.1.**

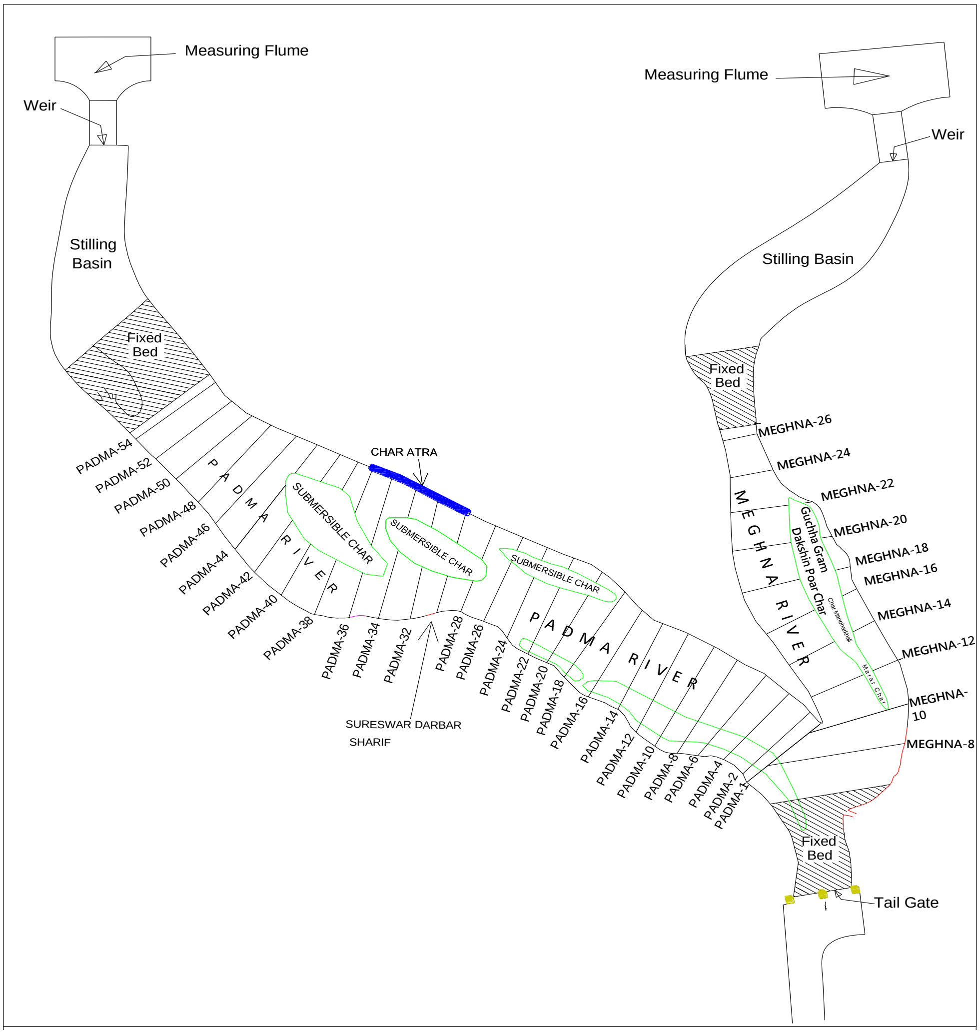


Fig. 1. Layout of the model

Test Scenarios

In this model, calibration test (T0) with existing condition and six application tests (T1-T6) with proposed interventions (dredged channel, revetment & T-head groynes) have been conducted. The proposed test scenarios of these test runs along with various discharge / WL conditions are mentioned in **Table 1.**

Table 1. Test Scenarios of the model

Test No.	Test Program	Flow Conditions
Calibration Test (T0)	Test with existing conditions. The model bed is prepared as per bathymetry of August 2018.	Measured discharge (Q=53,560 cumec) and corresponding water level 4.0 mPWD at C/S24 (3.5 km d/s of Sureshwar Darbar Sharif on the Padma river.
1st application test (T1)	Test as per preliminary design of proposed dredged channel supplied by BWDB + Existing condition	100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.
2nd application test (T2)	Modified alignment/design of dredged channel based on the test result of test T1+ Proposed bank revetment (8.50 km at U/S+0.85 km at D/S=9.35km) supplied by BWDB	100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.
3rd application test (T3)	Bank revetment as tested in T2 + Series of 7 (seven) groynes as proposed by BWDB + No dredging. 3 (three) groynes are big having length of shank & T-head each 1.0 km and the rests are small having length of shank & T-head each 0.5 km. All the groynes are perpendicular to the bankline. U/S & D/S length of T-head of groyne is 0.3L & 0.7L respectively. T-head angle is 60-degree (U/S) & 120-degree (D/S).	100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.
4th application test (T4)	Bank revetment as in test T3 + Modified alignment/design of movable groynes (6 nos.) as proposed by BWDB having shank length 1000m, 1300m, 360m, 400m, 500m & 500m and corresponding T-head length (L) 500m, 400m, 360m, 400m, 500m & 500m respectively) + No dredging. All the groynes are perpendicular to the bankline. U/S & D/S length of T-head of groyne is 0.7L & 0.3L respectively. T-head angle 80-degree (U/S) & 100-degree (D/S).	100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.
5th application test (T5)	Bank revetment as in test T4 + Series of 6 (six) groynes as proposed by BWDB having shank length (L) 1000m, 1300m, 400m, 400m, 500m & 500m and T-head length (L) 500m for each groyne + Dredging as in test T2. All the groynes are perpendicular to the bankline. U/S & D/S length of T-head of groyne is 0.7L & 0.3L respectively. T-head angle 70-degree (U/S) & 110-degree (D/S).	100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.
6th application test (T6)	Bank revetment as in test T5 + Series of 7 (seven) groynes as proposed by BWDB having shank length (L) 1000m, 1300m, 500m, 400m, 400m, 500m & 500m and T-head length (L) 500m for each groyne + Dredging as in test T5. All the groynes are perpendicular to the bankline. U/S & D/S length of T-head of groyne is 0.7L & 0.3L respectively. T-head angle 70-degree (U/S) & 110-degree (D/S).	Average annual discharge (30,000 cumec) and corresponding water level 2.3 mPWD at C/S24. Measured discharge (53,560 cumec) and corresponding water level 4.0 mPWD at C/S24. 100-year discharge (1,30,000 cumec) and corresponding water level 5.7 mPWD at C/S24.

Results and Discussion

The effectiveness of the proposed groynes in test T6 was found best in terms of flow diversion and sediment deposition trend between the groynes with varying discharges and water levels and present river flow pattern (**Fig. 2-4**). The performance of the dredged channel was found better compared to other tests. Dredged channel was found to pass significant amount of flow at the initial stages of model run. The rate of siltation through the channel was observed more during the model run of high discharge and water level, because more sediment is carried out at this condition and the water flow spreads all over the char land. But at the end of the model run the upstream part of the dredged channel found almost to be silted up. That means regular maintenance dredging would be required to keep active the dredged channel if dredging is implemented in the prototype. It also appears from the test that bank revetment is sufficient to combat bank erosion at Jajira-Naria but it could not be useful for land reclamation

T-head groyne and bank revetment together can arrest bank erosion and reclaim land properly. The effectiveness of dredged channel, groynes and bank revetment as observed after model run (T6) is shown in **Fig. 5**. The typical cross section showing silting trend at different discharges is shown in **Fig. 6**. The bed topography at the end of model run is shown in **Fig. 7.**

Tremendous turbulence with eddy and vortex was developed at the existing Sureshwar bank protection area that requires special attention/strengthening. Bank erosion at the downstream of Sureshwar protection was also noticed but in a smaller reach. Bank erosion at the left bank was also prominent in particular with flood discharge.

Fig. 2. Model area before run (T6)

Fig. 3. Flow diversion by the groynes during model run (T6)

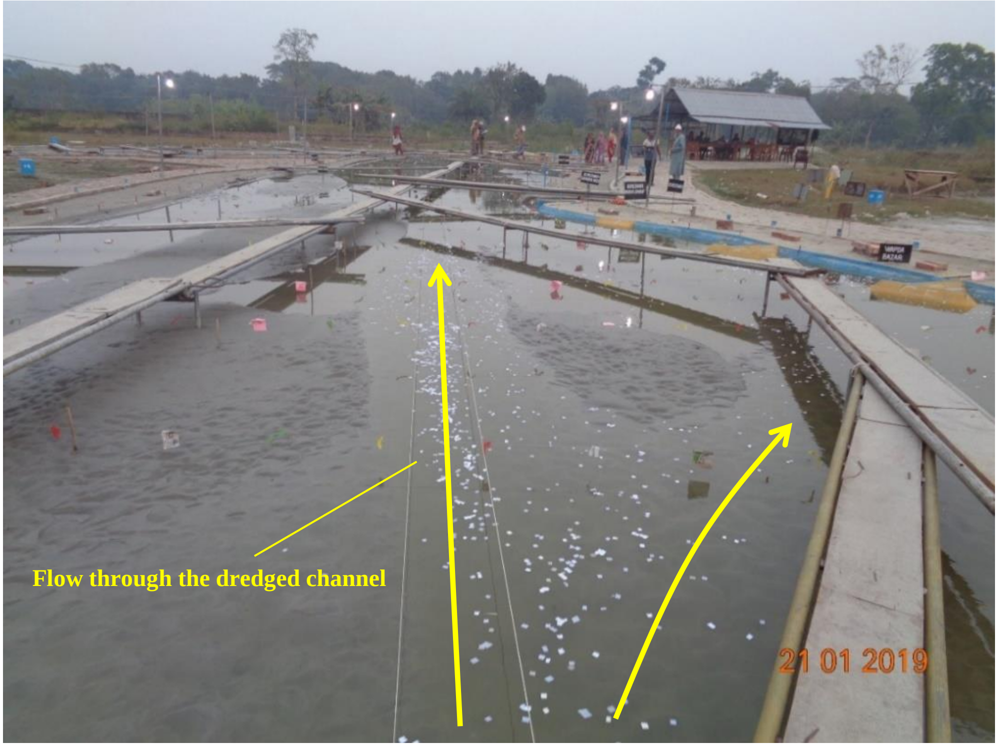


Fig. 4. Flow through the dredged channel at low flow (T6)

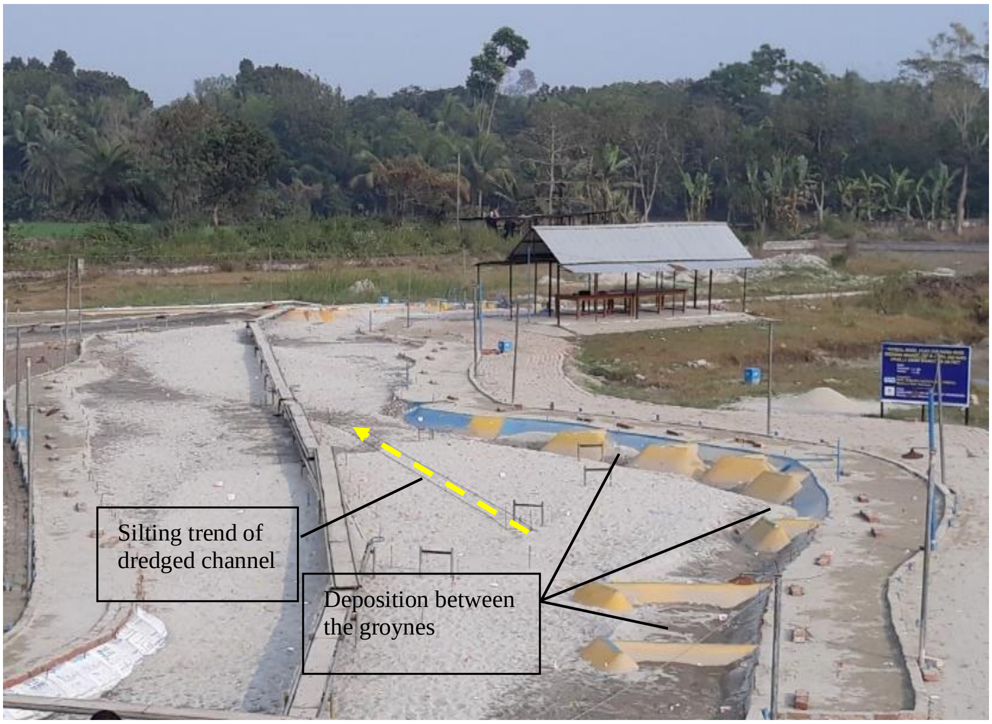
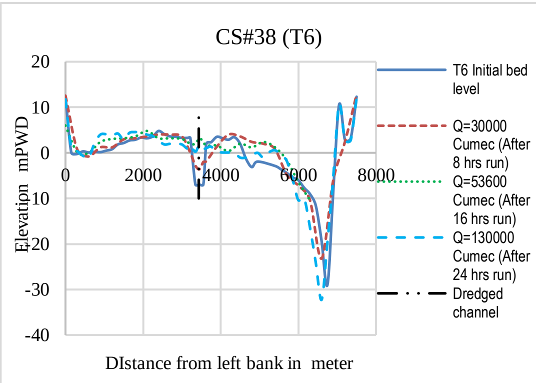


Fig. 5. Effectiveness of dredged channel, groynes & bank revetment as observed after model run (T6)



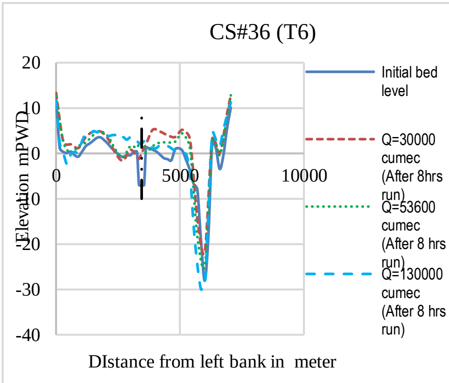


Fig. 6. Typical cross section showing silting trend at different discharges

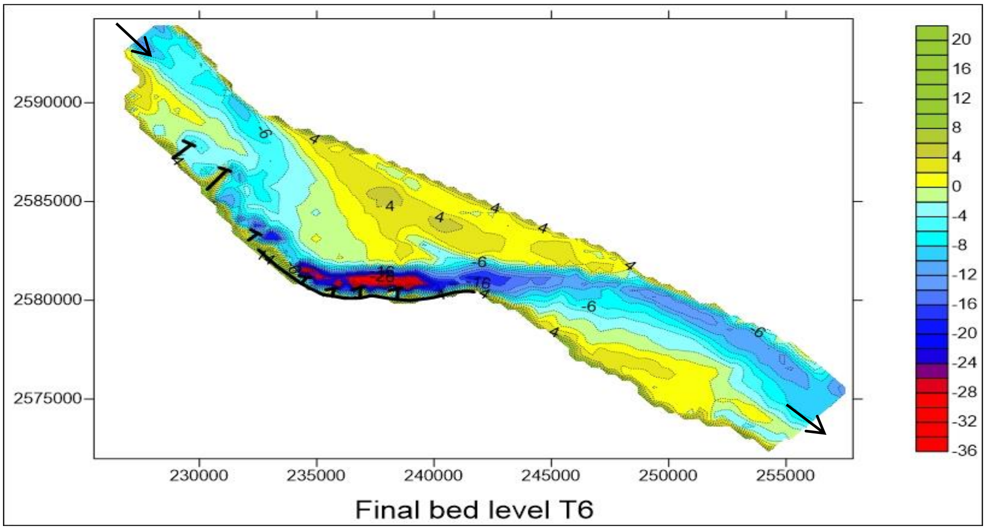


Fig. 7. Bed topography at the end of model run

Conclusion

- It was found from the calibration test that near bank velocity concentration at Naria - Jajira was severe to cause right bank erosion. The flow lines in this test also explains bank erosion will continue at this area if appropriate bank protection measures are not taken immediately.
- Velocity at the left bank in the calibration test was also large enough to cause bank erosion which should be taken into consideration.
- Right bank velocity was found to decrease to some extent due to the proposed dredged channel. Even though the velocity at the right bank was high enough to occur severe bank erosion at Jajira-Naria with 100-year discharge (1,30,000 cumec).
- Velocity as well as discharge was increased along the dredged channel in the beginning but at equilibrium condition of the model test, the dredged channel gradually got silted up. The upstream portion of dredged channel is silted up earlier than the downstream portion.
- The proposed revetment introduced at the right bank is found to working well as found from the model study.
- Near bank velocity measured at R/B for different cross-sections of the river model in test T2 varies 0-3.82m/s.
- Maximum velocity measured around groynes in test T6 (Q=30,000 cumec) is 2.61 m/s at groyne #4.
- Near bank velocity measured at L/B for different cross-sections of the river model in test T6 (Q=53,560 cumec) varies 0.48-1.96m/s.
- Maximum point velocity measured in test T6 (Q=1,30,000 cumec) along different cross-sections of the river model reaches up to about 4.50m/s.
- Near bank velocity measured at L/B for different cross-sections of the river model in test T6 (Q=1,30,000 cumec) varies 1.04-3.54m/s.
- Velocity at some selected points around the proposed revetment in test T6 (Q=1,30,000 cumec) was 0-3.9 m/s but about 4.5 m/s at Sureshwar.
- Maximum velocity measured around groynes in test T6 (Q=1,30,000 cumec) is about 4.75 m/s at groyne #7.

- Maximum scour (qualitatively as the model is distorted) measured around groynes in test T6 (Q=1,30,000 cumec) is 21.04 m (-37.28 mPWD) at groyne #7.
- Deposition occurs between the groynes tested in test T6 which is helpful for land reclamation.
- Flow severely attack the existing revetment at Sureshwar Darbar Sharif and proposed revetment immediate u/s and d/s of it due to the groynes tested in test T6. So due care should be taken before implementing of the groynes in the field.
- The total length of dredged channel is about 11.8 km. Dredged channel is mostly found to be silted up. At the end of the test, the percentage of siltation on average is measured about 70% fully silted (at the upstream) of its length and the rest part partially silted (at the downstream).
- Maximum velocity measured around groynes in test T6 (Q=53,560cumec) was about 3.00 m/s at groyne #6.

Recommendation

The proposed revetment (8.50km & 0.85km respectively at u/s & d/s of existing revetment at Sureshwar) tested in different tests works well and it is recommended to implement in the field. It is necessary to implement the bank revetment in the field immediately for the protection of the problem area and to prevent the bank erosion in the coming year.

Test T6 is the recommended test in this study. The location, alignment, dimension and spacing of groynes tested in this test is recommended to implement in the field as it provides better result. The recommended groynes are expected to divert the flow towards the mid river and to facilitate land reclamation by sedimentation between the groynes.

It is found from the model study that the dredged channel is partially silted up at average annual discharge (30,000 cumec) but at 100-year return period discharge (1,30,000 cumec), it is almost fully silted up. Therefore, if it is intended to maintain the proposed dredged channel, regular maintenance dredging would be required.

The dredging activities can be started from the straight end of dredging alignment as per construction facilities and the hokey shaped portion can be done observing the field condition.

Regular maintenance dredging would be required to make active the Kirtinasha river offtake as some deposition was observed in test T6.

Detailed layout of the recommended revetment, T-head groynes and dredged channel is shown in **Fig. 8**.

Existing bank revetment at Sureshwar requires strengthening since flow concentration, turbulence etc. are severe at that location.

BWDB design office supplied initial dredge section, alignment and location which was optimized by different test runs with the presence of the concerned design and field engineers during tests and accordingly the recommendations are made. However, if it is required to modify the recommended dredging alignment due to the morphological changes in the field, in that case it can be done using the experience of the design engineers and field engineers of BWDB having experience in dredge planning, design and implementation.

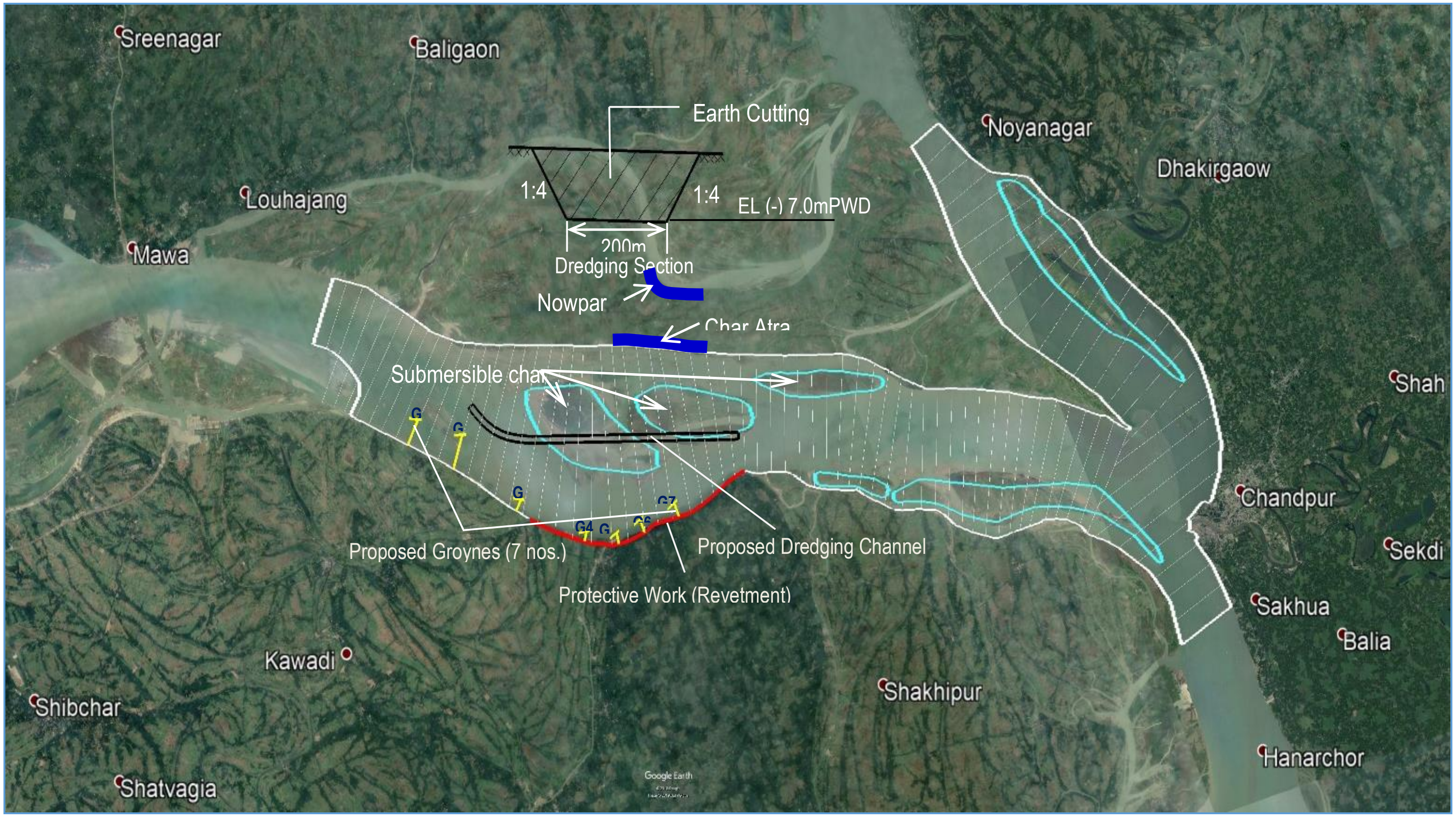


Fig. 8. Detailed layout of the recommended revetment, T-head groynes and dredged channel

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