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LAND RECLAMATION IN CHAR ISLANDS OF BANGLADESH USING MATHEMATICAL MODELLING TOOL: A CASE STUDY ON CHAR MAINKA-MONTAZ AREA

Mohammad Ziaur Rahman[1], Golam Mohiuddin[2] and Zahirul Haque Khan[3]

Abstract

Land reclamation, usually known as reclamation, is the process to create new land from sea or riverbeds. The land reclaimed is known as reclamation ground or landfill. Land accretion and erosion in the Meghna Estuary is a continuous and gradual natural process. Land accretion and erosion in the Meghna Estuary is a continuous and gradual natural process. The net average annual natural accretion is quite significant in the estuary ranging 1200 ha/y to 1900 ha/y (MES, 2001). Natural accretion at the south of Bhola, Hatiya-Nijhum Dwip and Noakhali area is very distinct. In Bangladesh, land reclamation by closure or cross-dam construction in order to accelerate the natural accretion process started in 1956 - 57. However it is seen that physical intervention is needed in order to accelerate the rate and area for this land starving country. Regularly received satellite imageries and other tangible supporting evidences suggest that Bangladesh is about to receive the gift of a huge land mass from its adjoining sea. Already, a substantial land mass has surfaced off the shore of Bangladesh on some of these completely surfaced land masses human habitations are growing while others emerge during the ebbing of the tide and go under water during the tides. The latter types of land formations could be elevated to be permanently joined to the mainland by engineering to step up the land accretion process. Indeed, much of present day Bangladesh including the districts of Faridpur, Barisal, Noakhali, Patuakhali, etc., were formed, over time in this manner. Land masses have already emerged from the sea and more would rise in the future. But the natural process being slow and it can be accelerated by engineering, which is neither prohibitive in cost terms nor complex, in technology terms. The cross-dam is a physical intervention to restrict the natural flow of a stream or river. For Bangladesh, it would require establishment of structures like cross dams to speed up the silt deposition pace in accreted or nearly accreted areas. Under Estuary Development Program (EDP) study the Bay of Bengal Model has been updated using the recent bathymetric and hydrometric data and it has been applied to ascertain the hydrodynamic and morphological processes of the Meghna Estuary at present and to assess land accretion for potential engineering interventions in Char Mainka- Montaz area. In this study, three cross-dams have been considered at the middle of 3 channels: Bestin channel, Mainka channel and Montaz channel. Model results show that a natural accretion process get accelerated as a result of these cross-dams.

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Introduction

Land accretion and erosion in the Meghna Estuary is a continuous and gradual natural process. Huge volume (approximately 1.1 billion ton) of sediment is transported to the Bay of Bengal through the Lower Meghna River and give rise to natural accretion in the shallow water area of Meghna Estuary. To accommodate rapid growing population and to boost up agricultural production more land is needed. In these circumstances it is seen that physical intervention is needed in order to accelerate the rate and area.

In Bangladesh, land reclamation by closure or the cross dam construction in order to promote accretion process started in 1956-57. The first such dam was built over a dying branch of the Meghna River, which is known as Noakhali Cross-dam 1. The length of the cross-dam was 13 km. The second cross dam having 30 km length was constructed in 1964 connecting Char Jabbar Island to the Noakhali mainland. About 1,000 sq. km lands were reclaimed due to the construction of these two cross-dams.

To assess the potential of land reclamation in the Meghna Estuary, Land Reclamation Project (LRP) and Meghna Estuary Study (MES) were carried out. Based on the LRP and the MES findings, Bangladesh Water Development Board made a priority list of 19 potential cross-dam sites for accelerating the natural processes of land accretion. Under Estuary Development Program (EDP) study the Bay of Bengal Model has been updated using the recent bathymetric and hydrometric data and it has been applied to ascertain the hydrodynamic and morphological processes of the Meghna Estuary at present and to assess land accretion for potential engineering interventions in Char Mainka- Montaz area. In this study, three cross-dams have been considered at the middle of 3 channels: Bestin channel, Mainka channel and Montaz channel. Model results show that a natural accretion process in this area under baseline condition and this accretion process get accelerated as a result of these cross-dams. These cross-dams will connect Char Montaz, Char Rustom, Char Bangla and Char Islam with Bhola Island and will reclaim considerable land in the area as well as improve the communication of the Char people without causing any adverse impacts on the surrounding water environment. The potential land accretion is about 493 hectres over a period of seven years as a result of construction of the proposed cross-dams.

Approach and Methodology

The study has mainly focused on the processes of river and estuarine hydraulics and the erosion-deposition pattern of the potential accretion zone and net flow and sediment distribution in the different channels of the Meghna estuary. The modelling study has been devised in combination with data analysis and numerical modelling. The two dimensional (2-D) flexible mesh model (MIKE21 FM) has been applied for hydro-morphological investigation around the study area. Data on recent bathymetry, sediment concentration and hydrometrics of the estuary have been utilized for updating and re-calibrating the existing Bay of Bengal model and establishing baseline conditions.

The Bay of Bengal model domain extends from Chandpur on the Lower Meghna river in the north to about 16^0 Latitude in the Bay of Bengal in the south. There are two open boundaries in the model, one is in the Lower Meghna River at Chandpur (northern boundary) and another one is in the Bay of Bengal (southern boundary). Predicted tide has been used in the southern boundary and observed water level at Chandpur BIWTA Station has been used in the northern boundary.

The riverbank alignment (land boundary) in the northern part of the model has been updated on the basis of the mosaic of IRS P6 LISS III satellite image 2008 provided by EDP. The model applies PWD datum. The southern part of the Bay of Bengal is quite deep and the maximum depth along the southern open boundary is more than 3,000 meter. The computational grid or mesh size decreases (or the resolution increases) towards coastlines and Islands. Figure 1 shows the computational mesh and bathymetry of dedicated Bay of Bengal (BoB) model.

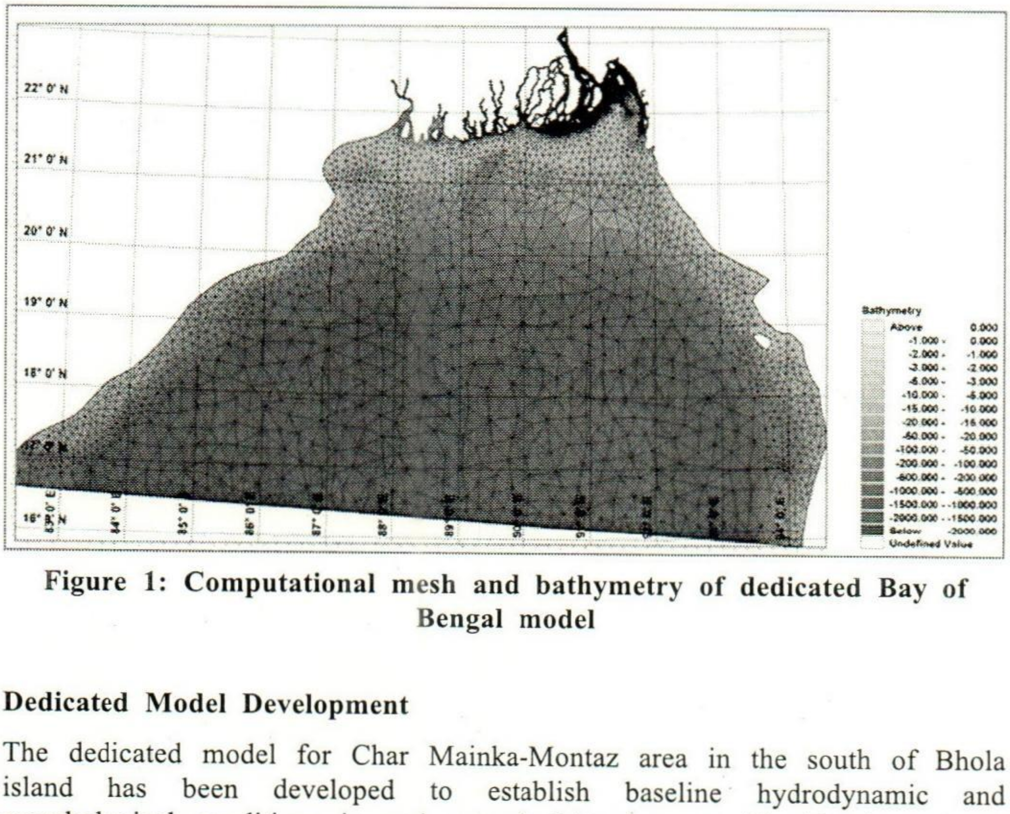

Figure 1: Computational mesh and bathymetry of dedicated Bay of Bengal model

Dedicated Model Development

The dedicated model for Char Mainka-Montaz area in the south of Bhola island has been developed to establish baseline hydrodynamic and morphological conditions in and around this place, to identify the suitable locations for cross-dams, to assess the impact of the proposed cross-dams on hydrodynamic and morphological conditions and to forecast short-term morphological change and land accretion due to different options of cross-dams. In this dedicated model the computational mesh has been made finer at the area of interest with respect to 2D General Model in order to incorporate different options of cross-dam and to assess their impacts at the surrounding area of those cross-dams. Figure 2 shows the computational mesh and bathymetry of Mainka-Montaz area. Other model setup has been kept same as 2D General Model.

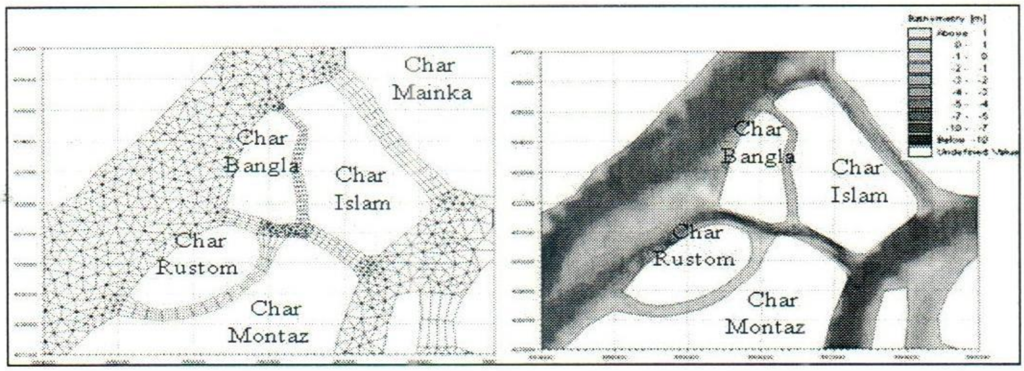

Figure 2: Computational mesh and bathymetry of dedicated model at area of interest

Model Calibration

The dedicated model for Mainka-Montaz area has been calibrated with the monsoon 2009 and dry season 2010 data. Figure 3 shows the locations of measured data which have been used for calibration. The calibration results are shown in figure 4 to 6. All of these figures show reasonable agreement between the measured and simulated discharge in Mainka-Montaz area.

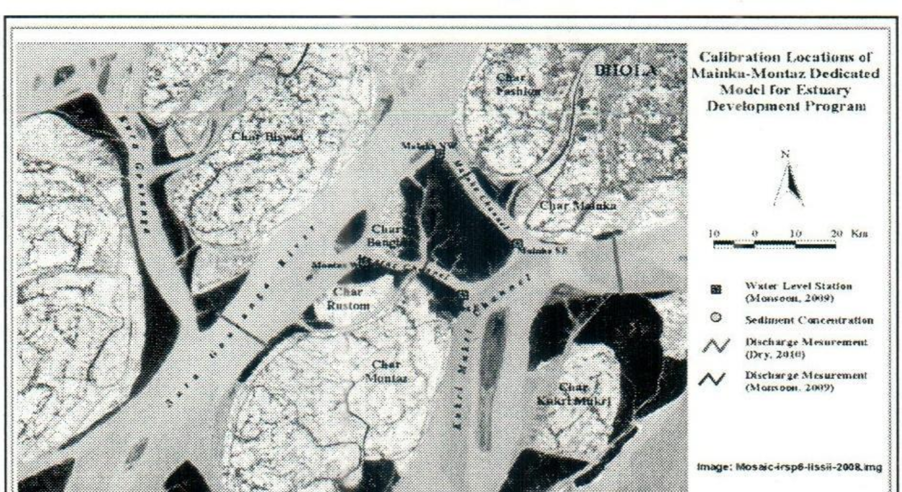

Figure 3: Calibration locations of Mainka-Montaz model

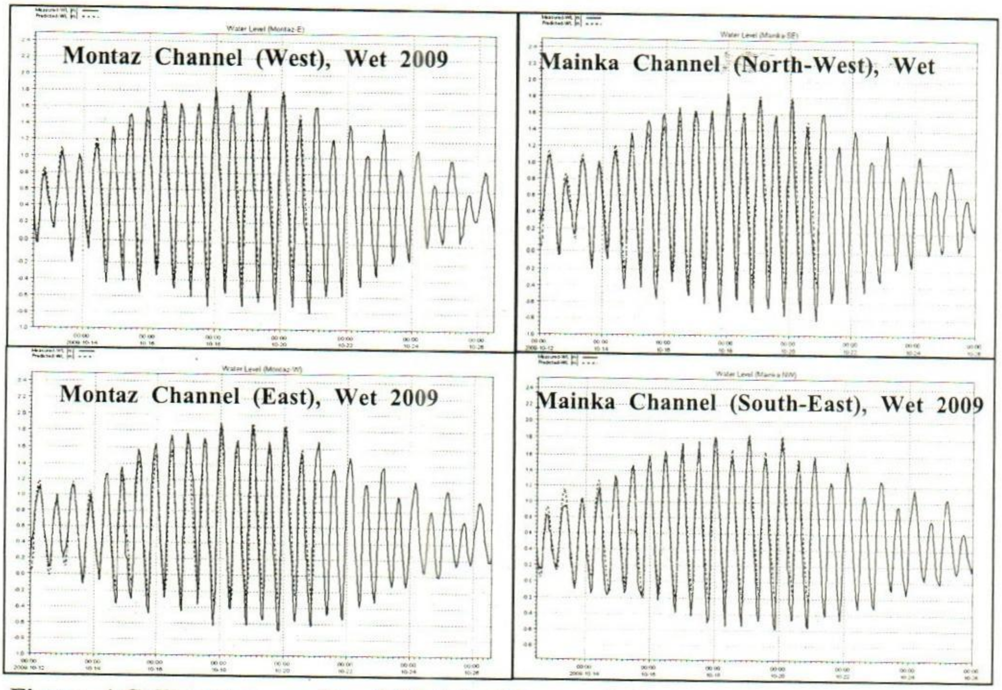

Figure 4:Calibration results of Mainka-Montaz dedicated model on water level

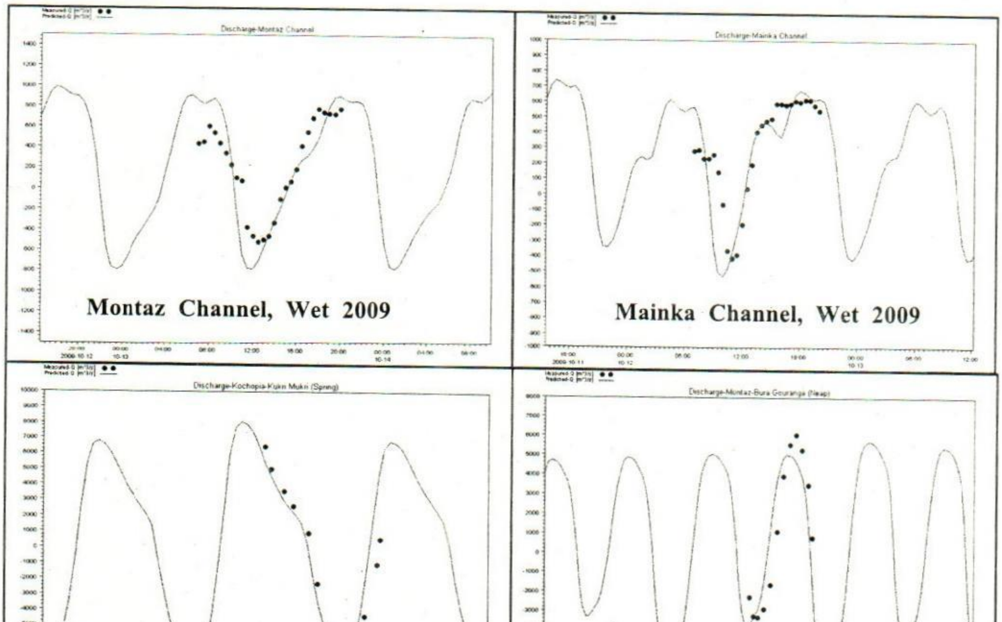

Figure 5: Calibration results of Mainka-Montaz dedicated model on discharge

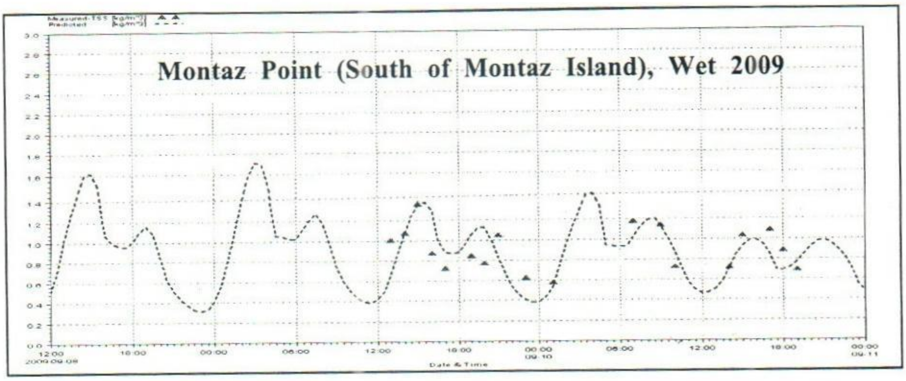

Figure 6: Calibration results of Mainka-Montaz dedicated model on sediment concentration

Establishment of Baseline Condition

The baseline condition of the Char Mainka-Montaz area has been analysed on the basis of local tide, current speed, wave, discharge and sediment transport. In this study, the hydrological year of 2009-10 has been considered as the base period.

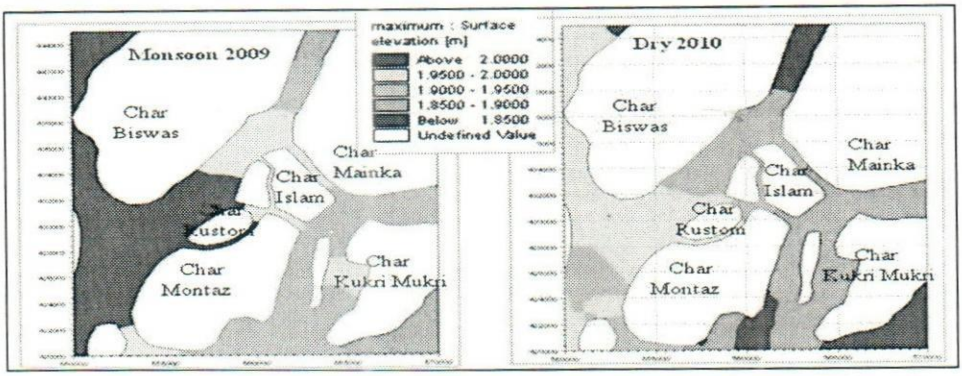

Figure 7: Maximum Water level in Mainka-Montaz area in Base condition

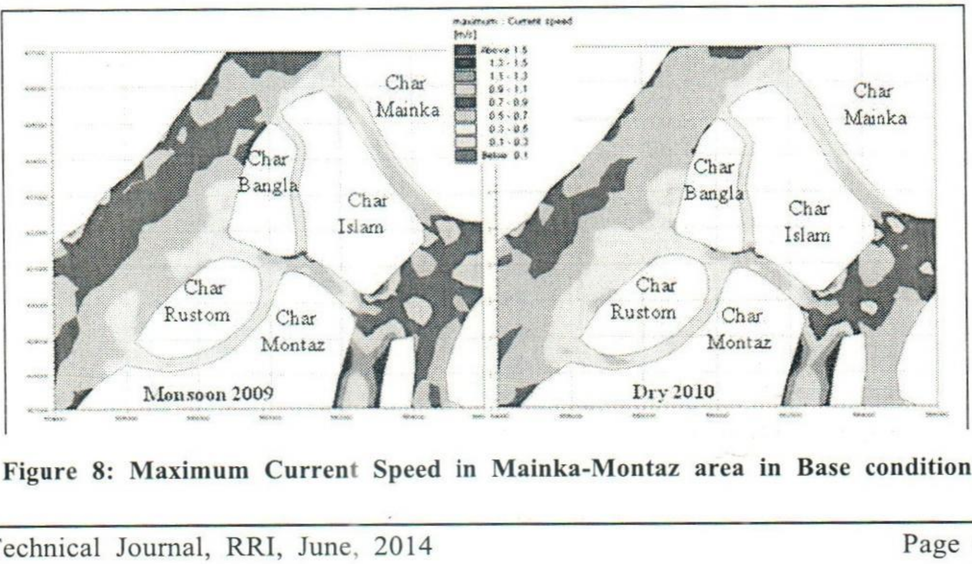

Figure 8: Maximum Current Speed in Mainka-Montaz area in Base condition

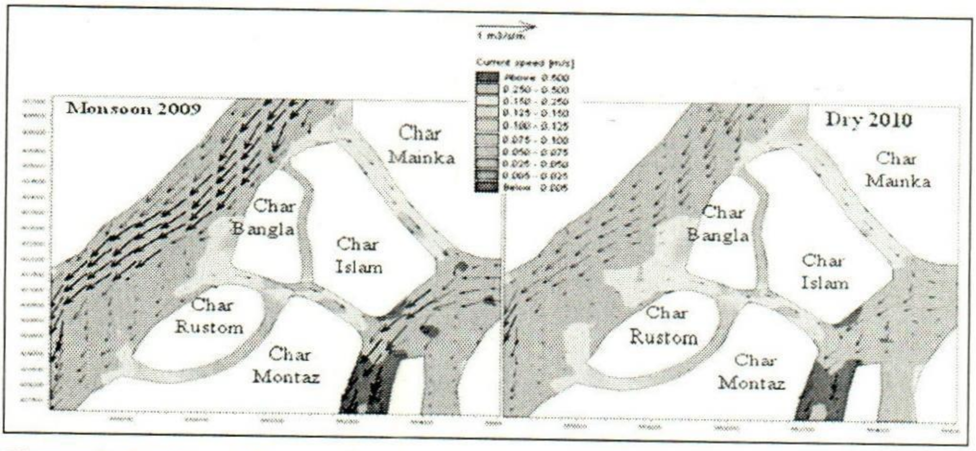

Figure 9: Residual flow and mean current speed in Mainka-Montaz area in Base condition

The area is dominated by tides generated in the Bay of Bengal. The flows from the West Shahbazpur channel and Tetulia Channel have also some influence on the flow inside Mainka and Montaz channels. Water level and current speed in the area have been simulated by the Mainka-Montaz dedicated model and the maximum maps in the wet period are shown in Figure 7 and 8. A residual flow map on the basis of August 2009 model simulations has been shown in Figure 9.

From the figures it is evident that the current speed is very small (0.1-0.3m/s) in Bestin and Bangla channels and moderate in Mainka and Montaz channels (0.3-0.7m/s). The current speed in Bura Gouranga channel varies from 0.5 to 0.9m/s while it varies from 0.5 to 1.5m/s in the Kukri Mukri Channel. The residual current in the Mainka and Montaz channels is from western towards east direction i.e. from Bura Gouranga to Kukri Mukri channel. From the measured data shown in Figure 6 it can be concluded that the sediment concentration in the Montaz point varies between 1.7 kg/m^3 to 1.00 kg/m^3 during spring tide and neap tide respectively.

Selection of Potential Locations of Land Reclamation

After hydrodynamic simulation of five different options in the Mainka-Montaz area one option has been taken for morphologic simulation. This option comprised of three potential cross-dams in the middle of Bestin, Mainka and Montaz channels. These cross-dams will connect Char Montaz, Char Rustom, Char bangla and Char Islam with Bhola Island, reclaim considerable land in the area and improve communication of the char people.

1. Bestin + Mainka closed (Mainka approximately at the middle of the channel)
2. Bestin + Mainka closed (Mainka approximately at the western end of the channel)
3. Bestin + Mainka + Montaz closed (all approximately at the middle of the channel)

4. Bestin + Mainka + Montaz closed (Mainka approximately at the middle of the channel, Montaz at eastern end)
5. Bestin + Mainka + Montaz closed (Mainka approximately at the middle of the channel, Montaz at western end)

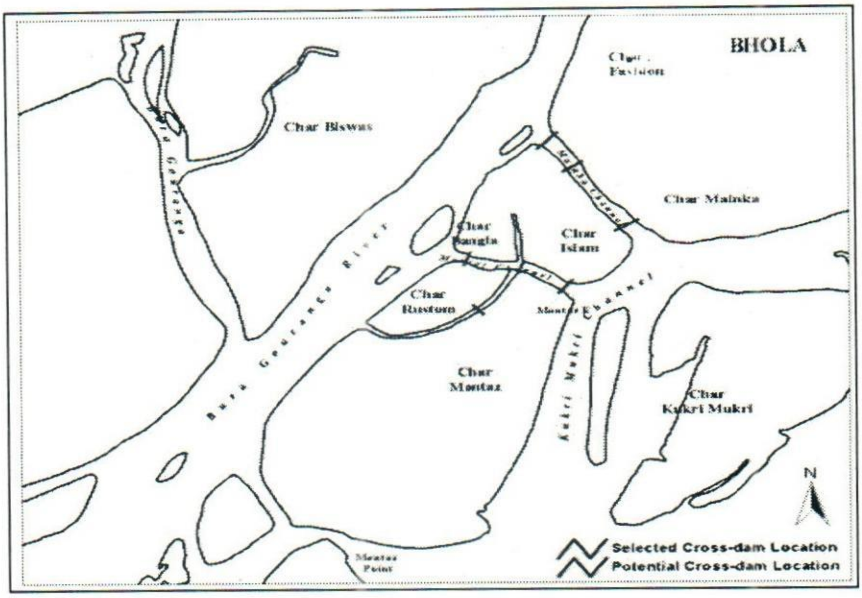

Figure 10: Potential cross dam sites

Hydraulic design parameters of Potential Cross Dams

All the design parameters for potential cross-dams derived from results of the base model developed under present study are presented in the Table 1. The design parameters estimated from the simulation of option-3 are presented in Table 2.

Table 1: Design parameters for the cross-dam in the Mainka and Montaz channel

Design Parameters			Mainka channel		Montaz Channel	
			Dry season	Monsoon season	Dry season	Monsoon season
Maximum water level			1.81	1.97	1.82	1.98
Minimum water level			-0.30	-0.21	-0.56	-0.44
Mean low water spring			-0.22	-0.16	-0.36	-0.32
Mean high water spring			1.46	1.64	1.47	1.63
High water spring			1.81	1.97	1.82	1.98
Low water spring			-0.24	-0.21	-0.56	-0.44
High water neap			0.70	0.85	0.64	0.82
Low water neap			0.21	0.20	0.03	0.16
Mean high water neap			1.01	1.12	0.95	1.09
Mean low water neap			0.03	0.06	-0.04	0.00
Mean water Level			0.58	0.68	0.54	0.64
Maximum Tidal Range			2.11	2.18	2.38	2.42
Maximum Tidal Amplitude			1.05	1.09	1.19	1.21
Velocity (m/s)			0.35-0.65	0.40-0.70	0.32-0.68	0.35-0.80
River width (km)			0.65	0.65	0.50	0.50
Max. Tidal Discharge (m^3/s)	Spring	Flood	670	645	937	1006
		Ebb	778	863	1057	1125
	Neap	Flood	424	526	695	700
		Ebb	445	510	842	886

Table 2: Design parameters after closure

Location of Cross-dam	Maximum depth Average Velocity(m/s) At u/s of X-dam w.r.t. net flow		Maximum depth Average Velocity(m/s) At d/s of X-dam w.r.t. net flow		Head Difference between two sides of cross-dam (m)	
	Dry season	Wet Season	Dry season	Wet Season	Dry season	Wet Season
X-Dam-Mainka	0.07	0.06	0.12	0.11	0.12	0.12
X-Dam-Montaz	0.13	0.13	0.05	0.05	0.13	0.13

Interpretation of Results

Land Accretion Due to Cross-dam

Three cross-dams have been proposed at the middle of Bestin, Mainka and Montaz channels. The construction of these cross dams will accelerate the deposition process in the area. Figure 11 represents the changed bathymetry after 3 years of model simulation without and with the cross-dams. From the figure it is evident that the deposition process will be increased by construction of the cross-dam. Moreover, the land reclamation in the area will also enhance the communication in this char area. Accreted land after 3 and 7 years with and without cross-dam above +1.5 mPWD is shown in Table 3.

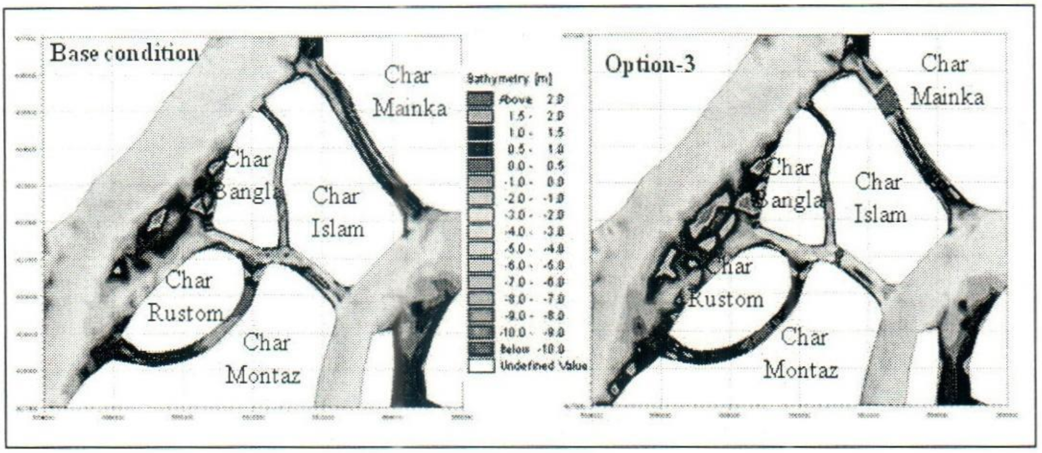

Figure 11: Land accretion after 3 years without (left) and with (right) cross-dams

Table 3: Example Land accretion due to cross-dam (Above +1.5 mPWD)

Year	Future with no cross dams (Ha)	Future with cross dams (Ha)	Benefit due to cross dams (Ha)
3	8	187	179
7	21	493	472

Impact Analysis of the Potential Cross-dams

Impact on Current Speed

The maximum current speed maps plotted in Figure 12 shows that the maximum velocity is reduced by the construction of the cross-dams in the respective channels. But there is no increase in the maximum velocity at the south-west side of the Sandwip island or any other bank in the area.

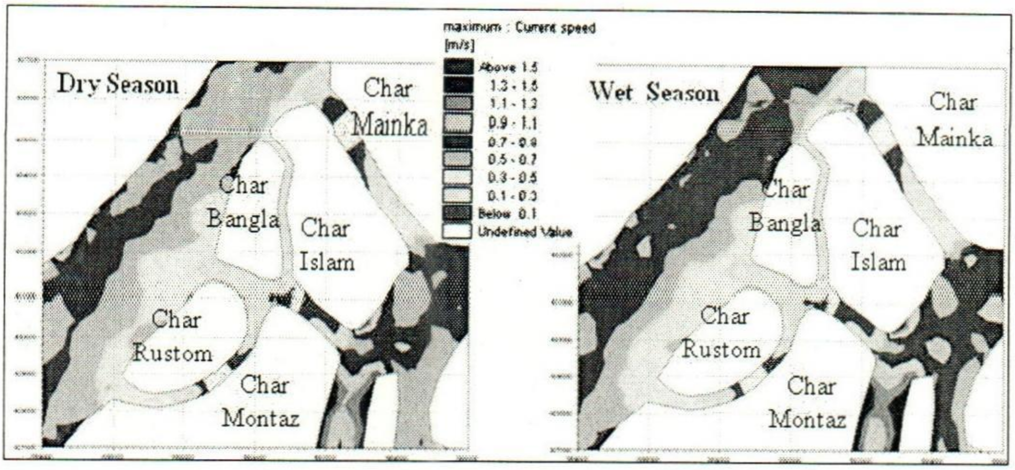

Figure 12: Maximum current speed with cross dams

Impact on Water Level

The 2-D maps of maximum water levels in Mainka-Montaz area for the option during dry period and monsoon period are shown in Figure 13. Figures show that the option causes higher surface elevation at upstream of the cross-dams due to the obstruction of flow.

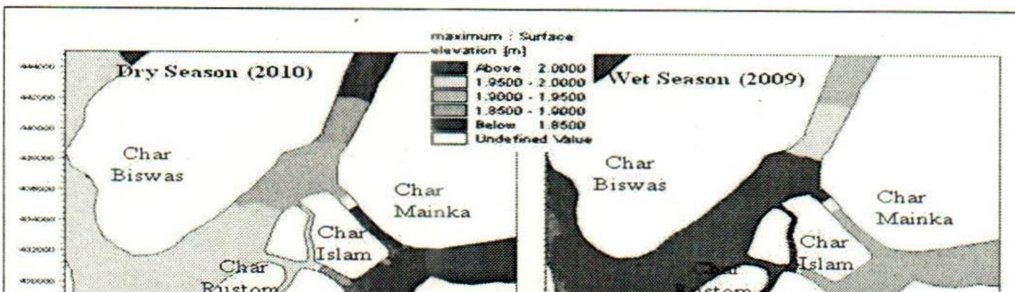

Figure 13: 2-D maps of maximum water level with cross-dams

Impact on Residual Flow

The residual flow (net flow) direction and mean current speed for monsoon 2009 and dry 2010 period with the cross-dams are shown in Figure 14 and Figure 15, respectively. When the three cross-dams are constructed the mean current speed is lowered significantly and the net flow is almost zer inside the channels.

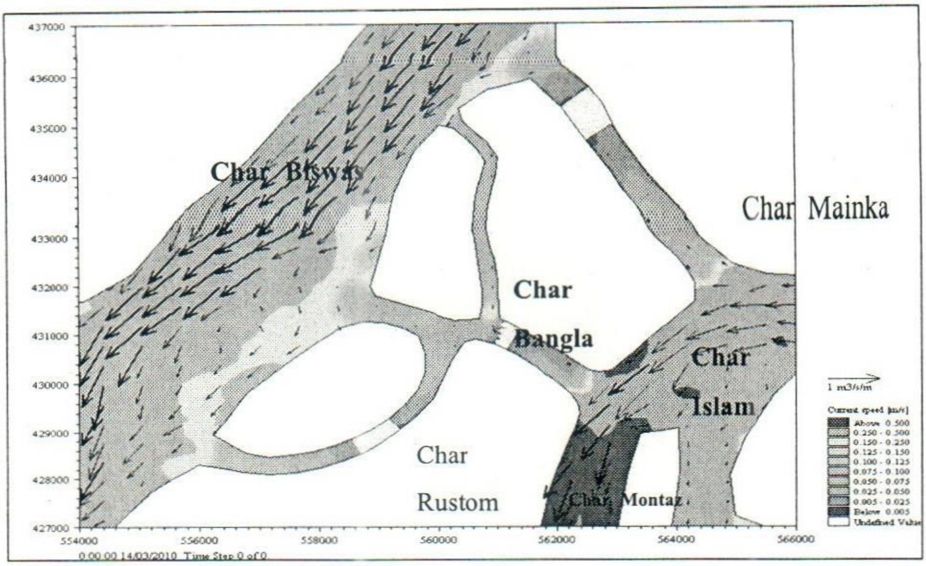

Figure 14: Residual flow and mean current speed during monsoon for option (August 2009)

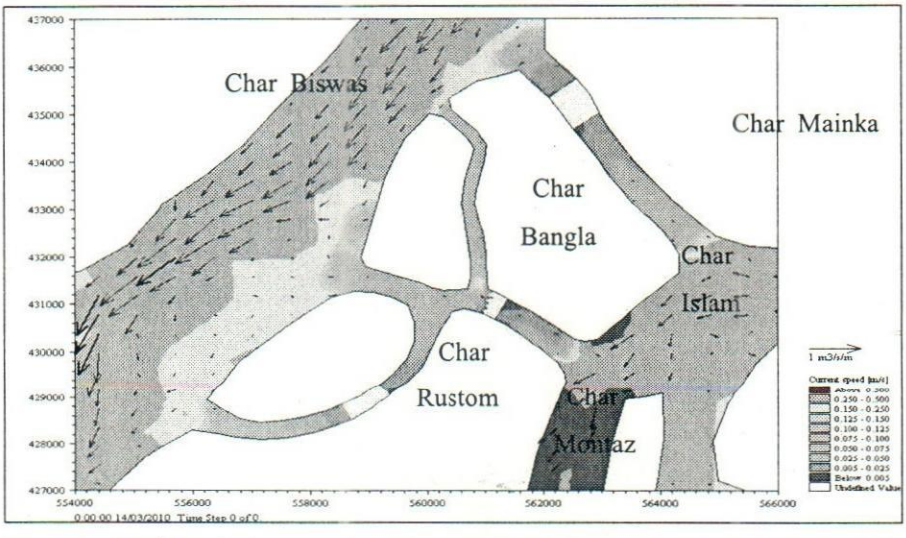

Figure 15: Residual flow and mean current speed during dry period for option (February, 2010)

Impact on Drainage and Erosion

The construction of the three cross dams will reclaim land inside the channels and at the western mouth of Montaz channel. But drainage congestion in the char area may arise due to the deposition in the channels. Proper monitoring works should be taken to prevent drainage congestion in the area. The cross-dams are proposed inside the small channels in the area and they have negligible or no impact on erosion in the adjacent area.

Conclusions

Simulation of morphological model for three consecutive years to assess the morphological changes is very complex and time consuming. In this study, the model has been simulated for some time in dry period and then with updated bathymetry simulation for next period is made. The model has been simulated for three years in this procedure and from the results seven years development has been determined.

Bangladesh is likely to get a positive response for funds as well as technical support from the international community if it can show that it is really keen to reclaim land from the sea for the overpopulated country. But we have to put the endeavor under a systematic policy framework. Holland is one country with unmatched expertise and experience in obtaining land out of the sea. It was in a situation, much worse than Bangladesh. Holland was below the sea level for which high tides and storms in the sea used to completely inundate it. The Dutch engineering solved these problems. They have also permanently reclaimed vast stretches of land from the sea to keep them dry within secure barriers or sea walls.

Acknowledgement

The authors express their gratitude to Institute of Water Modelling (IWM) for the support to use their joint study findings in the paper.

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DEPLETING TREND OF FRESH WATER RESOURCES WITH INCREASE IN WATER USE IS A THREAT IN BOGRA DISTRICT: A CASE STUDY

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Abstract

This study attempted to analyze the trend of fresh water resources and its assorted use under Bogra district using recorded data collected from pertinent organizations. Study found crop area coverage in Bogra district is almost saturated specially in dry period mostly with boro, a high water demand crop comparison with others. Only 7.5% of total annual precipitation falls in dry season in Bogra district with decreasing trend over the recent years, contributed a minute portion of groundwater replenishment. Dry seasonal water level is comparatively low in most of the major channels. Groundwater level is being depleted every year at frightening rate with increase in use of water especially in dry season.

Introduction

In recent years, decline of groundwater table is a threat in north-western region of Bangladesh due to over-abstraction of groundwater. Lowering of groundwater table during dry months is a critical issue to operate shallow tubewell, hand tubewell and dug wells. In addition, many ponds and tanks become derelict due to lowering of groundwater table creating shortage of water for both domestic use and use by the livestock population (WARPO, 2001). The ground water level declined substantially during the last decade causing threat to the sustainability of water use for both of irrigation and domestic purposes in the region and impacting upon other sectors too (Jahan, Mazumder, Islam & Adham, 2010). In the north-western region of Bangladesh, groundwater declines 5-10m in the dry period (Dey et.al, 2012). In greater Rajshahi area, extraction exceeds recharge and groundwater table declined 3 meters between 2004 and 2010 (Luby, 2013).

In the north-west region of Bangladesh, tubewell intensity increased from 6.9 to 36 per square kilometer; deep tubewell become almost double, shallow tubewell reached more than five times higher and irrigated land increased 1.6 times from 1984-85 to 2010-2011 (Dey et.al, 2012). According to Bangladesh Bureau of Statistics, population of Bangladesh increased from 28.93 million in 1901 to 149.77 million in 2011. More use of irrigation water especially groundwater has contributed to manifold increases in crop productivity in Bangladesh. Rice yield for example, increased from 1.0 MT/ha in 1971/72 to 2.8 MT/ha in 2008/09. Much of this increase in yield was due to an increase in the share of rice area especially during the boro season increased from 10% in 1971/72 to 44% in 2006/07. Rajshahi Division reported the highest percentage (27.22%) of households with Hybrid boro cultivation (BBS, 2008).

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This study focused on the specific objective to determine the trend of freshwater resources specially trend of groundwater level of the underlying aquifer system and assessed the water requirement of the Bogra district.

Study Area

The study area is located in 12 upazilas of Bogra district, lies between $24^0 32'$ and $25^0 07'$ north latitudes and between $88^0 58'$ and $88^0 95'$ east longitudes. According to BBS (2011), the total area of the study is 2898.68 sq km.

According to agricultural statistics (BBS, 2011), monthly average maximum and minimum temperature is 33.8^0C in October, 2011 and 10.1^0C in January, 2011. Monthly average relative humidity is about 85% in August, 2011 (maximum) and 66% in March, 2011 (minimum). Topography of the study area varies from 10.34 mPWD in Dhunat to 23.57 mPWD in Shibganj upazila. The Digital Elevation Model (DEM) having topographic variation within the study area is shown in Figure 1.

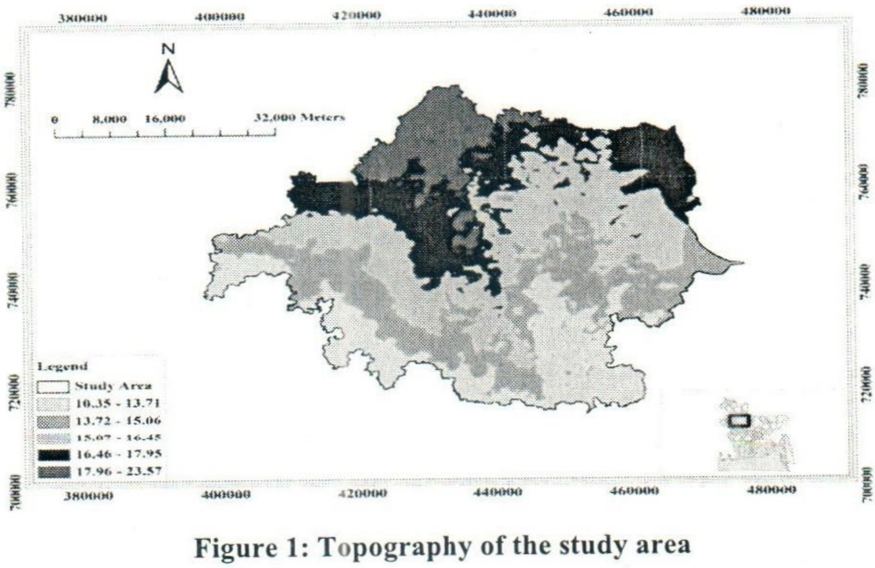


Figure 1: Topography of the study area

Methodology

Secondary data has been collected from Institute of Water Modelling (IWM), Bangladesh Bureau of Statistics (BBS), Bangladesh Agricultural Development Corporation (BADC), Bangladesh Water Development Board (BWDB) & Bangladesh University of Engineering and Technology (BUET) for relevant topics. Groundwater monitoring well data for 30 wells, precipitation data for 8 rainfall stations, river water level and discharge data from major river monitoring stations under BWDB has been collected from IWM, analyzed and used for this study. Crop pattern and crop water demand has been collected from IWM according to 2012 field survey (IWM, 2013). DTW and STWs information has been taken from minor irrigation report (BADC, 2011). Population and agricultural census data has been collected from Bangladesh Bureau of Statistics (BBS, 1991~2011). Data source locations within the study area have been shown in Figure 2.

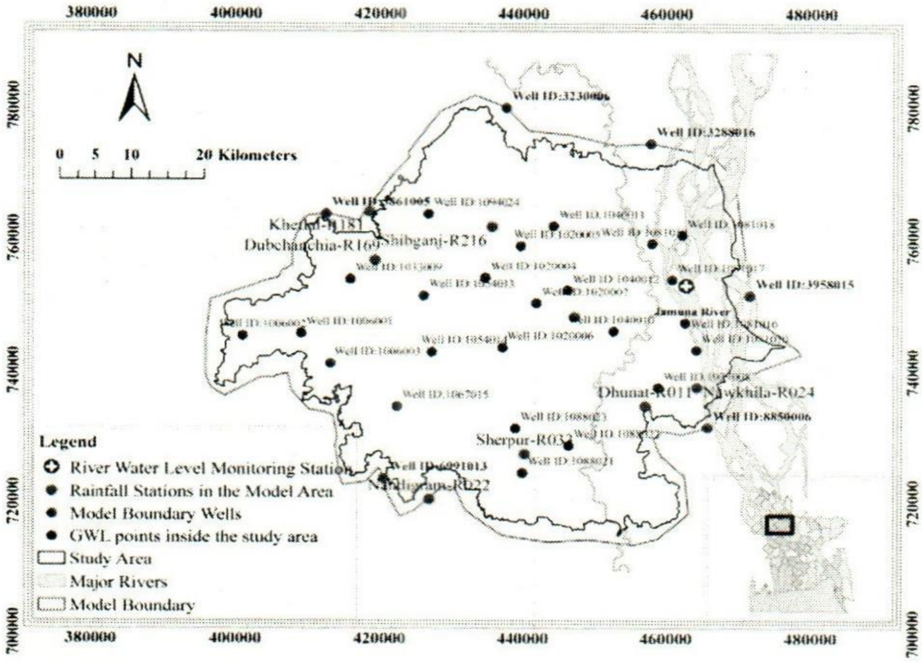


Figure 2: Rainfall, groundwater level, river water level & discharge monitoring stations in/around the Bogra district

Interpretation of Results

Main findings according to study objective on water use sectors due to irrigation and domestic purposes and fresh water resources as precipitation, surface and ground water resources has been discussed in this section.

Water Use Sectors

Water use is the total amount of water used by a household and/or the amount used for a given task and/or the amount for the production of a given quantity of crops. Agricultural water requirement is the dominating part of water use having around 70% of used water.

In the project area, main crops area rice-paddy, jute, wheat, potato and variety of vegetables and they grow in rain fed and irrigated condition. Boro, Wheat, potato and winter vegetables are the main Rabi (November to March) crops, while Kharif-I (April to June) crops are T-Aus & Jute and Kharif-II (July to October) grow HYV Aman, Local variety Aman and rainy season vegetables. The cropping pattern intensities throughout the study area according to 2012 IWM survey are shown in Figure 3. According to demand pattern, this cropping pattern can be categorized into crop calendar for the study area is shown in Table 1. Water demand of wheat, vegetables and mustard are almost same with compare to boro rice and has been categorized into same demand pattern as wheat. Drought and inadequate irrigation facilities are the major limitations to intensive land use and optimum crop production. In the Dry season (Rabi), major contribution of land occupy boro rice is about 53.2% of total study area and about 70% of crop area within Bogra district. Occupied area divided into boro and non boro area. Total crop water requirement has been shown in Table 2.

Table 1: Crop Calendar for the study area

Month	Land I.D.			
	1	2	3	7
July	Fallow	Fallow	Aus	Fallow
August	Fallow	Fallow	Aus	Fallow
	T-Aman	Fallow	T-Aman	T-Aman
September	T-Aman	Fallow	T-Aman	T-Aman
October	T-Aman	Fallow	T-Aman	T-Aman
November	T-Aman	Fallow	T-Aman	T-Aman
December	T-Aman	Fallow	T-Aman	T-Aman
	Fallow	Fallow	Fallow	Wheat
January	Fallow	Fallow	Fallow	Wheat
	HYV-Boro	HYV-Boro	HYV-Boro	Wheat
February	HYV-Boro	HYV-Boro	HYV-Boro	Wheat
March	HYV-Boro	HYV-Boro	HYV-Boro	Wheat
April	HYV-Boro	HYV-Boro	HYV-Boro	Fallow
May	HYV-Boro	HYV-Boro	HYV-Boro	Fallow
	Fallow	Fallow	Aus	Fallow
June	Fallow	Fallow	Aus	Fallow

Table 2: Water requirements for existing crop pattern for the study area

Crops	Crop Area in Sq.km	Water requirement (November to April)		Remarks
		in mm	in Mm^3	
Boro	1,570.10	639	1003	Water Requirement for existing crop pattern = 1203 in Mm^3
Non Boro	684.12	292	200	

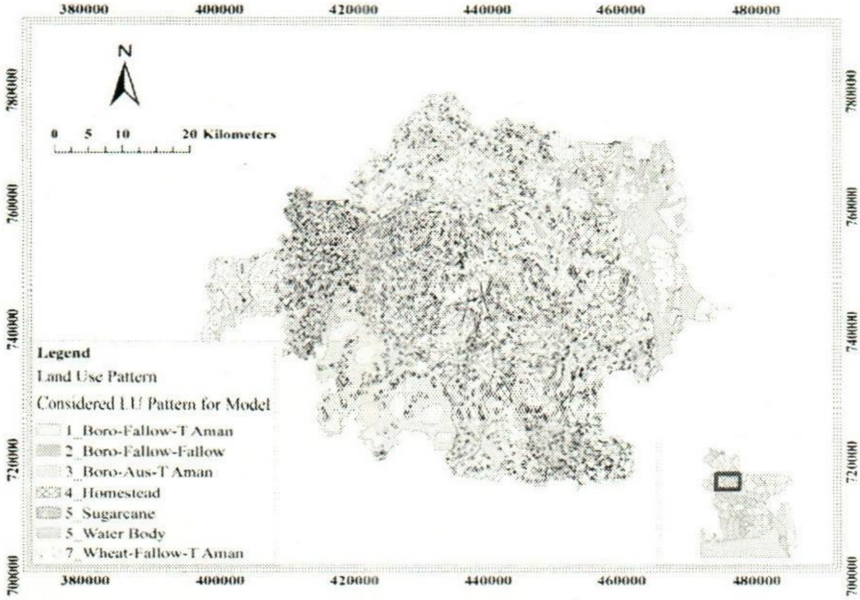


Figure 3: Adopted land use pattern according to crop type throughout the study area (IWM, 2013)

According to Bangladesh Population Census (Bangladesh Bureau of Statistics) of the year 1991, 2001 and 2011 the counted population for rural and urban areas of Bogra district is shown in Figure 4. Shajahanpur upazila has come in focus after 2001 census year.

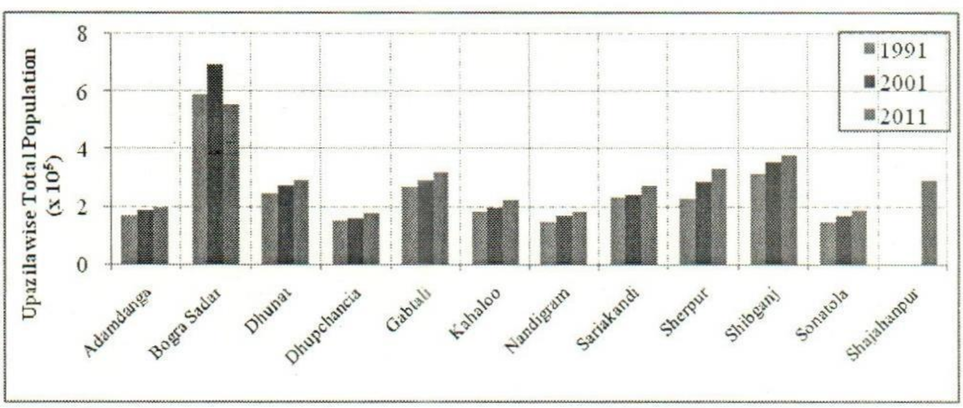


Figure 4: Upazila wise Population in the study area according to Population Census (BBS, 1991~ 2011)

According to population data and considering, per capita water demand has taken as 120 l/c/d for zila town, 100 l/c/d for upazila town and 50 l/c/d for rural population respectively (Ahmed, M.F. & Rahman, M.M., 2003), total domestic water requirement of Bogra zila in 2011 is about $38Mm^3$.

Fresh Water Resources and Trends

Precipitation, Surface water and groundwater are the main fresh water resources to fulfill the water use requirement for agricultural, industrial and domestic purposes is described in this section.

Precipitation is a major control of fresh water availability and it is unevenly distributed everywhere. The yearly average rainfall in the study area is about 1800 mm according to BBS data archive from 1985 to 2011 data. Only 7.5% precipitation falls in the dry months from November to April and rest 92.5% of precipitation falls from May to October in the wet season is shown in Figure 5. The trend of total rainfall in the study area concludes rainfall depletion over time is shown in Figure 6. Deviation of yearly rainfall from the annual average rainfall is shown in Figure 7. The monthly variation of rainfall is shown as a hyetograph in Figure 8 from the average of the total data series from 1985 to 2011.

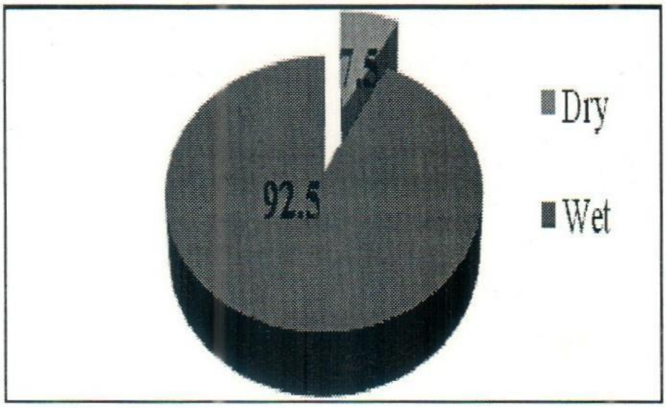


Figure 5: Distribution of rainfall into Dry and Wet Seasons (BBS data)

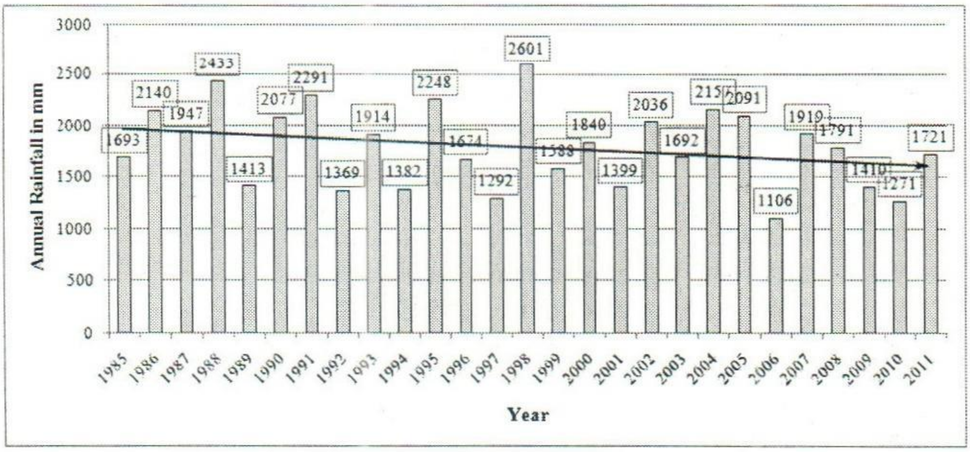


Figure 6: Distribution of annual rainfall in the study area (BBS data)

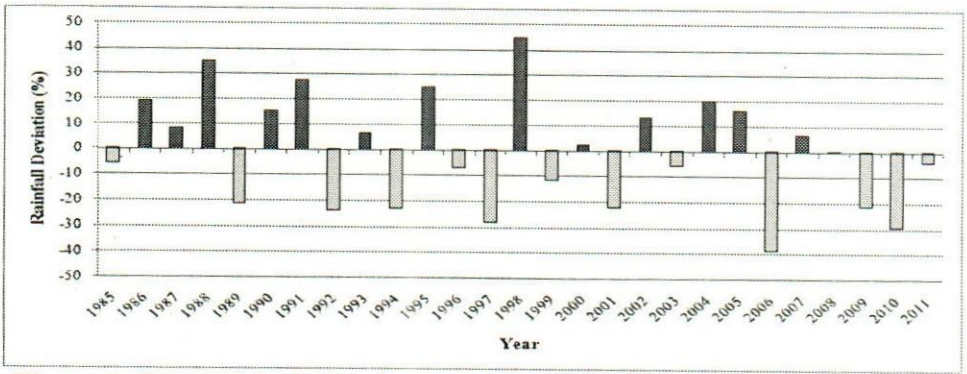


Figure 7: Rainfall deviation from annual normal rainfall in the study area

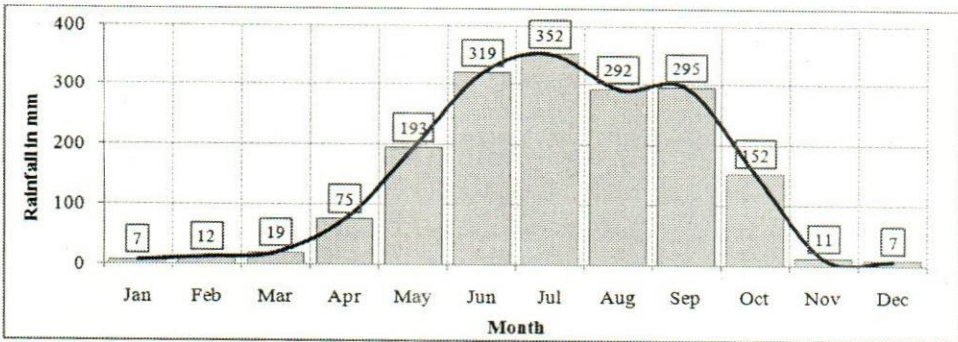


Figure 8: Rainfall distribution into months of the study area as mean of 26 years BBS data

From November to April about 131mm of rainfall is available that gives only 383 Mm^3 of water that is not sufficient to meet total irrigation water requirement. Similarly groundwater recharge for this amount of rainfall is far below amount than water abstraction in dry period.

Some major rivers and a list of beels are the main surface water sources in the study area. Major Rivers passing through the study area are Jamuna, Karatoya, Nagar, Bangali and Ichamati. Among all rivers, Jamuna is contributing major role and dominating surface water resources especially in dry period when other channels are almost dry. River water level hydrograph of River Jamuna with maximum and minimum elevation at Mathurpara station is shown in Figure 9 and Figure 10 that shows dependability analysis of water level. In dry period, 80% water level data has been found equal or less than of 11.81 mPWD from dependability analysis from 1992 to 2012 data using Weibull formula (Subramanya, 1994). Ratio of 80% dependable discharge between dry period and all months from 2005 to 2012 data is about 1: 3.43 ($\leq$ 11,000 m^3/s in dry and 37,700 m^3/s for all data). River water level of Jamuna is depleting over the time at a rate of 5.11 cm/year has determined from the trend line in time series data is shown in Figure 11.

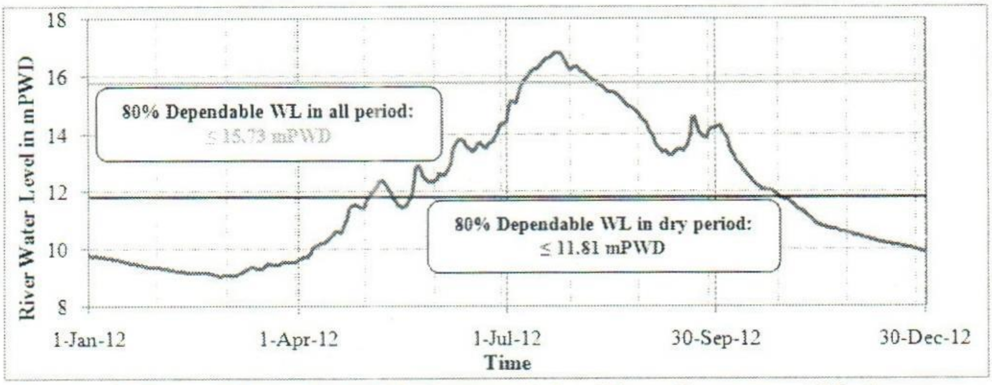


Figure 9: River Water Level Hydrograph for River Jamuna at Mathurpara Station (BWDB data)

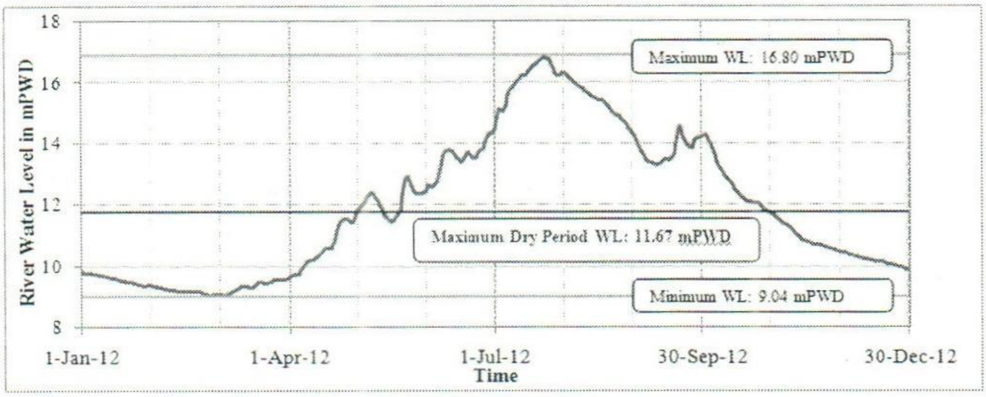


Figure 10: Dependability analysis of water level for Jamuna River at Mathurpara Station

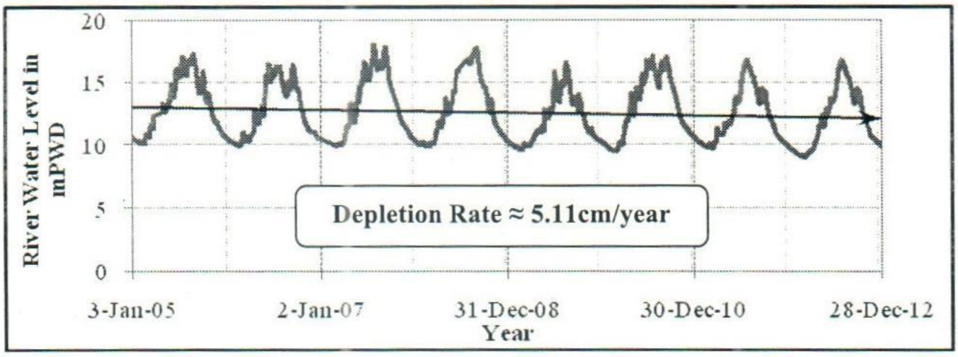
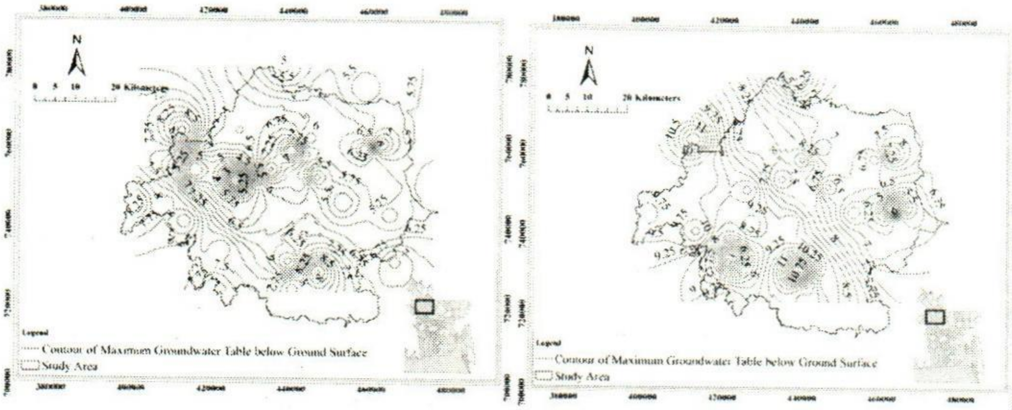


Figure 11: Trend line of water level data at Mathurpara Station on Jamuna River

Groundwater resources is an very important component of the freshwater system and its role is becoming even more prominent as the more accessible surface water resources become increasingly exploited to support increasing populations and development. Comparative contours between year 1985 and 2010 of groundwater table through study area are shown in Figure 12.



(a) Year 1985 **(b) Year 2010**

Figure 12: Contours for Minimum Groundwater Table (below GL) for year 1985 and 2010 (IWM data)

Depletion rate of boundary and inside groundwater wells has been shown in Table 3 and Table 4 respectively. Groundwater depletion rate of the well in Gobindaganj upazila (Well ID: GT3230006) around the study area is shown in Figure 13.

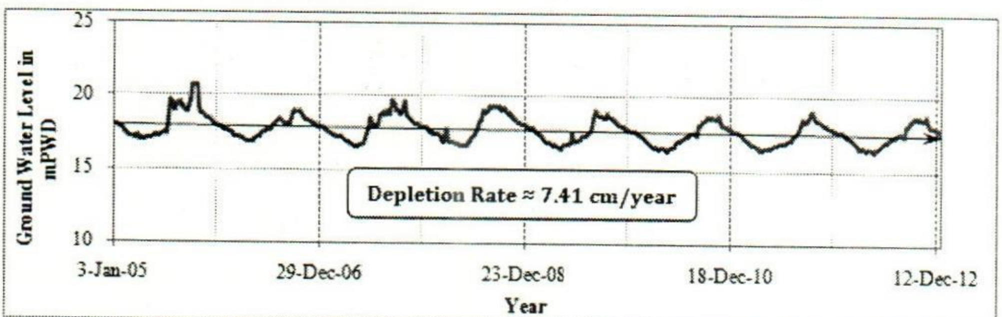


Figure 13: Depletion trend of Groundwater Level for the Well ID: GT3230006

Table 3: Calculated depletion rates for 6 boundary wells in the Bogra District

Ground water Well ID	Depletion Rate of GWL in cm/year from 2005 to 2012 data		
	Trend line from time series data	Tread line from Maximum GWL data	Tread line from Minimum GWL data
3230006	7.41	19.90	9.00
3288016	11.61	4.20	22.60
3958015	6.13	16.10	16.10
8850006	2.01	25.00	1.10
6991013	24.35	79.00	18.00
3861005	27.67	54.60	28.30

Table 4: Depletion trend of Groundwater wells inside the study area

Well ID	1006001	1006002	1006003	1020004	1020005	1027008	1033009	1040010	1040012
Depletion Rate (cm/year)	42.05	39.2	41.68	3.83	18.07	7.74	29.16	0.037	8.03
Well ID	1054014	1067015	1081017	1081018	1081020	1088021	1088022	1088023	1094024
Depletion Rate (cm/year)	42.74	24.35	0	0	0	4.34	17.89	2.66	26.1

Study has been found, groundwater level is depleting over the year from 2005 to 2012 at a alarming rate in some areas; but not uniformly through the study area. Eastern and north-eastern part specially river surrounded region of the study area shows less depletion than western or south-western part. Another thing is that, hydraulic conductivity of the aquifer under eastern and north-eastern part of Bogra district is higher than other part (Figure 14).

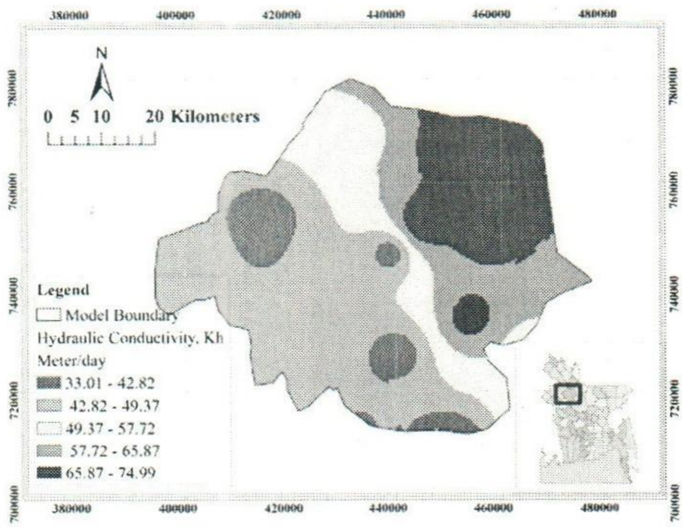


Figure 14: Distribution of horizontal hydraulic conductivity of Bogra district (IWM, 2013)

Conclusions

In dry period, boro is the leading crop occupied most of the cropping area of Bogra district and is almost saturated into total study area. Existing crop water requirement in dry period is about 1203 Mm^3. If boro crop is replaced to other dry crops, crop water requirement could be limited to 659 Mm^3 (considering, same crop area) in dry period. If boro crop is occupied total cropping lands, crop water requirement in dry period will be increased to 1440 Mm^3 (considering, same crop area). About 38 Mm^3 of domestic water might be added with irrigation water as total water requirement.

A part of fresh water resources only 7.5% precipitation falls in dry season. Rainfall in dry months gives only about 383 Mm^3 of fresh water (from 26 years average data). Against huge water requirement more than 2000 DTW and 75,000 STW is now being abstracted groundwater that increases per year at a rate of 3.68% and

3.83% respectively. Surface water level also goes down it's lower limit; even some channels and water bodies dry up in dry period. The groundwater level is depleting at an alarming rate, somewhere up to 43 cm/year. The groundwater depletion rate in north and north-western part of Bogra district is higher than south and south-eastern part.

Recommendations

Now, it is recommended to detail study intensely about how increasing water demand can be confined within tolerable limit of groundwater aquifer in accordance with other fresh water resources. Groundwater model considering future extreme scenarios with probable remedial measures should be run to predict what will happen in future.

Acknowledgement

Author is being expressed his sincere gratitude to Dr. Syed Fakhrul Ameen, Professor, Department of Civil Engineering, BUET and Dr. Mahbuboor Rahman Choudhury, Assistant Professor, Department of Civil Engineering, BUET for their special support to initiate the study, a part of M.Sc thesis; also thanked to Institute of Water Modelling & their experts.

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Tech. J. River Res. Inst. 12(1): 27-31, 2014 (June)

THE BEST CANAL LINING TECHNOLOGY: A REVIEW

Syed Md. Anwaruzzaman[1], Uma Saha[2], Md. Abul Ala Moududi[3] and Md. Zubayerul Islam[4]

Abstract

This paper describes the various aspects of canal lining techniques. Engineers and researchers studied on canal lining to find out cost effective lining techniques and its application to the field. From their achievements, it is found that there are some drawbacks on canal lining technique and its implementation. As canal lining depends on several factors, so a particular method is not suitable for different region. For selecting the lining method it should be needed to consider the factors such as soil types and climate of the region, availability of lining materials and labor. From the comparative study on different canal lining techniques it is envisaged that the best canal lining technique could be selected on the basis of cost effectively and durability of the structure (canal) for a particular irrigation project or any.

Introduction

Irrigation is recognized as a "leading input" for increasing agricultural production, Water loss increases the cost of production. Main loss occurs during supply of water through unlined canal. There is no doubt that about 50% of irrigation water is lost in percolation and seepage. Improvement of irrigation command is greatly depending on the efficiency of water conveyance and distribution system. The water conveyance efficiency in minor irrigation project in Bangladesh is very poor due to high losses of water through the sides and bottom of unlined irrigation canal. In an earth canal soil texture is an important parameter influencing seepage and percolation losses. These losses are also high due to the fact that, in farmers' fields the channels usually are not well designed and nor compacted and graded. Lined canal control seepage, prevent water logging, increase in channel capacity, increase in command area, reduce maintenance cost, eliminate of flood danger etc. Mathematically it can be expressed that if the benefit cost ratio is more than one for a irrigation canal project then the lining is justified. There are several types of lining such as; Cement concrete lining, shot Crete lining, brick or concrete tile lining, asphalted concrete lining, compacted earth lining, soil cement lining. There are several factors responsible for selecting the type of lining technique, such as size of the canal, importance of the canal, channel slope and alignment, climate of the area availability of the material, initial expenditure etc.

Objective of the study

- To critical review the different canal lining method.
- To eventual selection of canal lining technique for a particular type of soil of an irrigation project.

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Literature Review

Several Agricultural Engineers and other relevant personnel studied on irrigation canal lining. Some of the prominent works are illustrated here. Dutta (1981) found that the conveyance losses through the main canals of minor irrigation projects are as high as 60% of the total water diverted. Bishwas *et al.,* (1984) stated that in Bangladesh, average coverage per cusec ($0.0283m^3$/sec) of Deep Tube well (DTW), Shallow Tube well (STW), and Low Lift Pump are 9.06, 6.08, and 7.95 hectares respectively. Low coverage is mainly due to huge water losses in the irrigation conveyance systems.

Dutta (1982) showed that the conveyance losses through the main canal in minor irrigation projects areas (canal length varying from 210-685 meter) is as high as 60% of the total water pumped by a DTW of 0.0586 m3/sec discharging capacity.

Under such cases, adoption of some lining techniques is necessary for minimizing water losses in irrigation canals. Besides, reduction of water losses, lining reduce maintenance and operation costs and many other field hazards like water -logging and salt & alkali concentration in agricultural soils. Lining also reduces canal dimension and right of way costs.

Khair *et al.,* (1985) stated the loss rate through the selected precast canal sections ranged from 0.0053-$0.0055m^3/m^2$/day where as, in the same area the average losses through compacted unlined channel was $0.41m^3/m^2$/day. They also stated that the initial cost of brick lining for $0.0028m^3$/sec canal is Tk. 150.00 per running meter where as, for the same capacity the average initial cost of selected precast sections is about Tk. 50.00 only.

Huda *et al.,* (1993) stated that the cost per running meter of semicircular concrete canal section is Tk.43.88 which is about 25% of brick lining. They also stated that the semicircular canal section has the least rate of seepage amounting to 0.0051 m^3/m^2/day and its water saving was 99% over uncompacted earth canal. Hence the water loss for the introduction of this line canal reach of 32m, reduced from 31.65 to 0.003%, as compared to previous condition.

Khair (1982) reported that nearly one acre ofland can be saved in a DTW irrigation project area if rectangular canal sections are used instead of traditional linings in main and laterals.

Under such conditions, attempts are made to design and developed durable precast canal sections, at a lower initial cost suitable for smaller irrigation canals ($0.028m^3$/sec) in minor irrigation project areas in Bangladesh.

Kinori (1970) stated that soils that may be used as lining materials are;

i. Sand with clay as binding materials (50 to 70% sand, 30 to 40% clay, 0 to 10% silt)
ii. Gravel with binding clay (30 to 40% clay)
iii. Sand with high clay content(30 to 40% sand, 70 to 60% clay);
iv. Loam clay, (30 to 50% clay, 60 to 40% silt, 0 to 10% sand) and
v. Fat clay.

Fat clays are not suitable for canals which are subjected to alternate wetting to drying because of danger of cracking. A well graded sand and gravel within a clay binder is considered best materials for compacted earth lining. The soils of Bangladesh are mostly alluvial in character and clays are at the low lying areas of Bangladesh. Information regarding suitable earth materials in Bangladesh for lining farm canals is lacking. Therefore, there is a scope to conduct a study for the utilization of available suitable earth materials and mixtures of soils and others available stabilizing additives for canal linings. In reality no particular lining appears suitable and satisfactory for use in all locations. The choice of lining to be technique depends on the availability of materials, cost, and skills and on the extent of water tightness and stability desired.

Methods

A number of journals, books, reports are collected and studied thoroughly on canal lining. Theoretical knowledge is gathered and critical reviews are drawn on the basis of cost of construction, water loss, lining materials etc. Finally a conclusion is drawn on the basis of above factors.

Discussions

Several researchers studied on canal lining to provide a cost effective lining techniques. From their achievements, it is revealed that there are some drawbacks on canal lining technique and implementation. It may be due to the small scale or case study. As canal lining depends on several factors, so a particular method is not suitable for different region. For selecting the lining method it should be needed to consider the factors such as soil and climate of the region, availability of lining materials and labour. The salient features of canal lining are shown in the following table.

Table 1: The Salient features of Lined and Unlined Canal

SI. No.	Type of canal lining	Average Seepage rate ($m^3/m^2/day$)	Water saving in percent over		Construction cost per running meter (TK.)	Expec-ted life (year)
			Unlined & uncompacted canal	Unlined & compacted canal		
1	Semicircular concrete canal section (cement:sand:khoa)	0.0051	99.65	98.75	43.88	22
2	Precast rectangular section (cement:sand:khoa)	0.0053	99.64	98.71	50.00	17
3	Precast rectangular concrete canal section (cementsand:surki)	0.0055	99.63	98.66	40.00	16

Sl. No.	Type of canal lining	Average Seepage rate ($m^3/m^2/day$)	Water saving in percent over		Construction cost per running meter (TK.)	Expec-ted life (year)
			Unlined & uncompacted canal	Unlined & compacted canal		
4	Compacted earth canal (alluvial soil)	0.41	72.11	-	15.00	3
5	Brick lining	0.028	98.10	93.12	190.00	15
6	Soil cement lining	0.062	90	85	20.00	5
7	Clay saw dust	0.091	-	-	12.00	1
8	Clayricehusk	0.073	-	-	11.00	1
9	Unlined and Compacted Canal	0.41	72.11	-	8.00	-
10	Unlined and uncompacted Canal	1.47	-	-	5.00	-

From the above table it is observed that cost of construction of brick lining is higher than that of precast canal section with respect to the water saving rate. Moreover, water saving rate is high and eventually it reduces the agricultural production rate.

Conclusions

From the comparative study on different canal lining techniques it is concluded that the best canal lining technique could be selected on the basis of cost effectively and durability of the structure for a particular region which will improve the socio-economic condition of Bangladesh.

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Technical J. River Res. Inst. 12(1): 32-43, 2014 (June)

SUSTAINABLE WATER RESOURCES MANAGEMENT THROUGH MATHEMATICAL MODELLING FOR NORTH WEST REGION OF BANGLADESH

Md. Tarikul Islam[1] and Dr. AFM Afzal Hossain PEng.[2]

Abstract

Sustainable Water resources management in a right way is of prime importance to formulate plans for irrigation, domestic purposes and drought management through optimum utilization of the water resources. In this paper, a case study was carried out for Panchagarh, Thakurgaon, Dinajpur and Joypurhat Districts which is situated in the north-west part of Bangladesh using physically distributed hydrological modelling. With the existing irrigation facilities, it was possible to bring only about 40% of the total cultivable land under irrigation. To bring more potential land under irrigation through sustainable water resources management, an integrated GW-SW model was developed using mathematical modelling software MIKE-11 and MIKE-SHE which was calibrated for the period 2003-2007 and validated for the period 2008-2009. Using model result, groundwater and surface water resources, requirement for present and future demand for various purposes and possible expansion of irrigation coverage for the study area was assessed along with requirement of additional number of Deep Tube Well (DTW). Since surface water availability is very limited during dry season (January to April), irrigation from groundwater sources have been proposed through installation of additional 2847 DTWs of 56 l/s capacity for 17 Upazilas to cover the future demand. As a result irrigation coverage as well as agricultural production would be increased considerably if the project is implemented following the study findings and suggestions. Some area were also been identified which is potential for surface water irrigation. The economic analysis shows minimum EIRR value of 29.85% when all costs increase by 30%. So the study output has positive impact and for sustainable water resources management it is very essential to use the state-of -the art technology.

Introduction

Currently, one-third of the world's population is living in countries and regions of water resources limitation (Bates, *et. al.*, 2008). Because of limited water availability imposing strong restrictions on natural and human systems, the management of water resources has become an increasingly pressing issue in semiarid and arid regions. Generally, when the demand of water has reached the limits that the natural system can provide, water shortage can become a major obstacle to social and economic development for one region (Bronster *et al.*, 2000; Li *et al.*, 2006). Therefore, these issues have forced planners to contemplate and propose ever more comprehensive, complex, and ambitious plans for water resources systems in the semiarid and arid regions (Li *et. al.*, 2008).

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Different studies have documented that groundwater level declined substantially during the last decade causing threat to the sustainability of water use for irrigation in this region and impacting upon other sectors too (Jahan *et. al.*, 2010). Due to lack of proper knowledge, indiscriminate installation of pumps and non-availability of modern technologies, farmers inappropriately lift water without caring ground sources. These impacts upon interlinked sources of water table which is declining alarmingly in many areas of Bangladesh. Although the groundwater dominates the total irrigated area, its sustainability is at risk in terms of quantity in the northwest region (Simonovic 1997; Shahid 2011). Frequent shortage of water in the region has had impacts that can be ranged as economical, social and environmental (Takara and Ikebuchi, 1997; Sajjan *et. al.*, 2002; Dey *et. al.*, 2011).

A recent study shows that groundwater level in some areas falls between 5-10 m in dry season and most of the tubewells fail to lift sufficient water (Dey and Ali 2010). Researchers and policymakers are advocating sustainable development as the best approach to today's and future water problems (Loucks2000; Cai X *et. al.,* 2001). With groundwater development, fluctuations will amplify; but as long as rainfall is managed to recharge aquifers, and proactive water saving strategies are put in place, a steady and sustainable state can be achieved (IWMI 2010). In mainstream irrigation thinking, groundwater recharge is considered as a by-product of flow irrigation, but in today's world, groundwater recharge needs to be understood on its first emergency for making groundwater sustainable integrating all possible options (IWMI 2010).

Hydrologic model was a useful tool for water resources management (Sahoo *et. al.*, 2006). Previously, many lumped hydrologic models were developed to investigate watershed hydrology. With a low data requirement, these lumped catchment models could reflect runoff dynamics and water balance in water resource management systems. However, the lumped models assumed the study watershed as a spatially homogeneous region, and the spatial heterogeneity of the climate variable and land surface was not considered (Bronster *et. al.*, 2000).

Consequently, several distributed and semidistributed hydrological models were developed in response to the aforementioned challenges (Apul *et. al.*, 2005). For example, Refsgaard (1997) integrated MIKE SHE, MIKE 11, MIKE 21, and DAISY to study the environmental assessment in connection with the Gabcikovo hydropower scheme. Sahoo *et. al.*, (2006) used the physically distributed hydrological modeling system (MIKE SHE) to study the watershed response to storm events within the Manoa-Palolo stream system on the island of Oahu, Hawaii. IWM (2005, 2006 and 2009) used the physically distributed hydrological modeling system (MIKE SHE & MIKE 11) for the assessment of potential groundwater and surface water resources. The primary advantage of the distributed hydrological models was enabled to reflect the spatial variations for characteristics of watershed (e.g., rainfall, topography, soil type, and land use) (Refsgaard, 1997). However, higher data requirement became a main obstacle on extensively applying these models to practical problems

Both the Poverty Reduction Strategy (PRS) and Millennium Development Goal (MDG) of the Government of Bangladesh attached priority to increase agricultural production. In this backdrop, Barind Multipurpose Development Authority (BMDA) has undertaken a programme entitled "Deep Tube Well Installation Programme, Unit-2, Thakurgaon" covering 28 Upazilas of Dinajpur, Thakurgaon, Panchagarh and Joypurhat districts having gross area of 7,63,100 ha and cultivable area of 5,75,152 ha. Under this project IWM (2009) carried out the study. With the existing irrigation facilities, it was possible to bring only about 40% of the total cultivable area under irrigation.

The study area (Figure 1) is a drought prone area lies approximately between 88°5.52'E to 89°18.47'E longitudes and 24°49.5'N to 26°39.59'N latitudes. The yearly rainfall varies from 1800 mm to 2600 mm, relative humidity varies from 46% to 83% and topography varies from 94 mPWD to 15 mPWD.

The objective of this paper is to review the state of art on sustainable water resources management approaches for irrigation and ultimately growing food production by bringing more area under irrigation coverage without allowing environmental degradation.

Approach and Methodology

Sustainable water resources management for irrigation is not only an objective on the farm level but also an overall goal at the regional level, which means in general at the basin scale. Due to the competition of all water users of a river basin, especially in water scarce regions, a comprehensive approach is needed regarding not only agricultural purposes but also domestic, industrial, and ecological aspects.

In order to achieve the study objectives, IWM (2009) developed an integrated gw-sw model for the study area. The models developed under this study are based on MIKE-11 (DHI, 1999) for the simulation of flows in rivers, irrigation systems, channels and other water bodies and MIKE-SHE (DHI, 1999) for groundwater model. All the major river systems and updated topographic features were included in surface water (sw) model setup while hydro-geological setting, aquifer properties, DEM, land use pattern, irrigation abstractions were incorporated in groundwater (gw) model. Both the models were coupled through MIKE SHE. The coupled model was calibrated for period of 2003 to 2007 and validated for the period of 2008-2009. The validated model was used to simulate various options and to assess the resources.

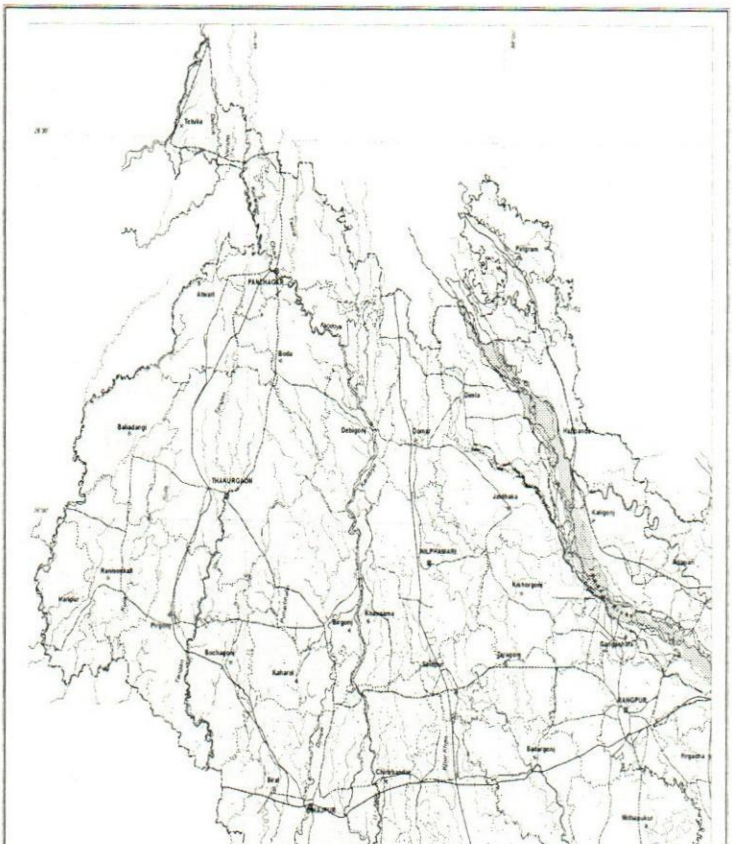

Figure 1: Location Map of the Study Area

Surface water resources at some key locations were assessed based on average year dry period minimum flow. However, for sustainability of a river, it is not feasible to utilize all available resources. As such, useable resource has been considered as 70% of the available resources and 30% as "in-stream flow requirement" in the river (IWM, 2006). Groundwater resources were assessed considering yield criteria of 7m depth from ground surface, so that STW & HTW remain active and potential recharge was assessed upto the depth which recharge fully during monsoon due to rain. Useable recharge was considered as 75% of the potential recharge (IWM, 2006) to account for various uncertainties inherent in different assumptions and natural loss. Irrigation zones were demarcated on the basis of water availability, groundwater level fluctuation, functionality of suction mode pumps, safe yield, extent of irrigation coverage, extent of drainage congestion etc. Environmental impacts, social acceptance and economic viability of the project were also assessed. The approach of the study is shown in a schematic plan illustrated below (Figure 2).

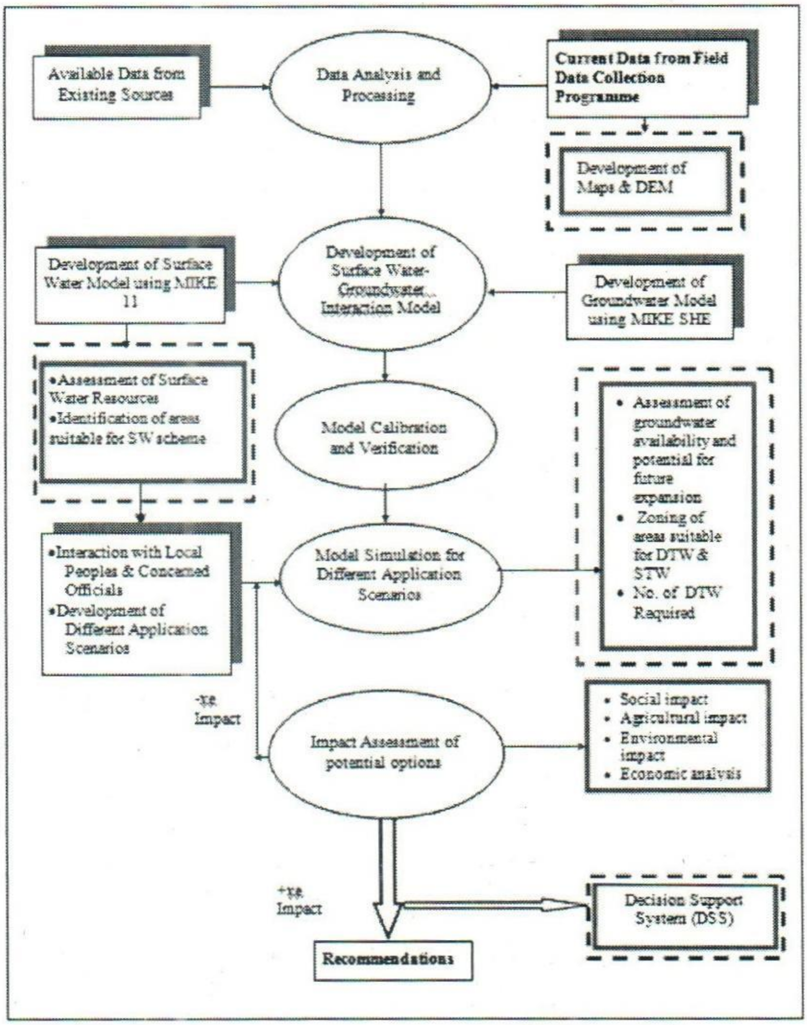

Figure 2: Schematic Diagrams of the Study

Results and Discussions

Assessment of Groundwater Resources

Upazila-wise groundwater resource for average hydrological year was assessed based on recharge characteristics, potential recharge and safe yield criteria. The end of April is the end of irrigation period when the lowest water table generally occurs, after that water table starts rising due to recharge from rainfall. Recharge is the net storage in saturated and unsaturated zone. The components that influence the groundwater storage after April are mainly rainfall, runoff, overland flow, overland storage, drain to river, evapo-transpiration, boundary inflow and outflow. All the factors were considered during estimation of potential recharge through water balance for each Upazila. The water balance for Akkelpur Upazila (MIKE SHE output) is shown in Figure 3; Potential Recharge = 601 mm (for SZ-Storage change) – 20 mm (for UZ-Storage change) = 581 mm. Upazilawise potential resources and usable resources is given in Table 1.

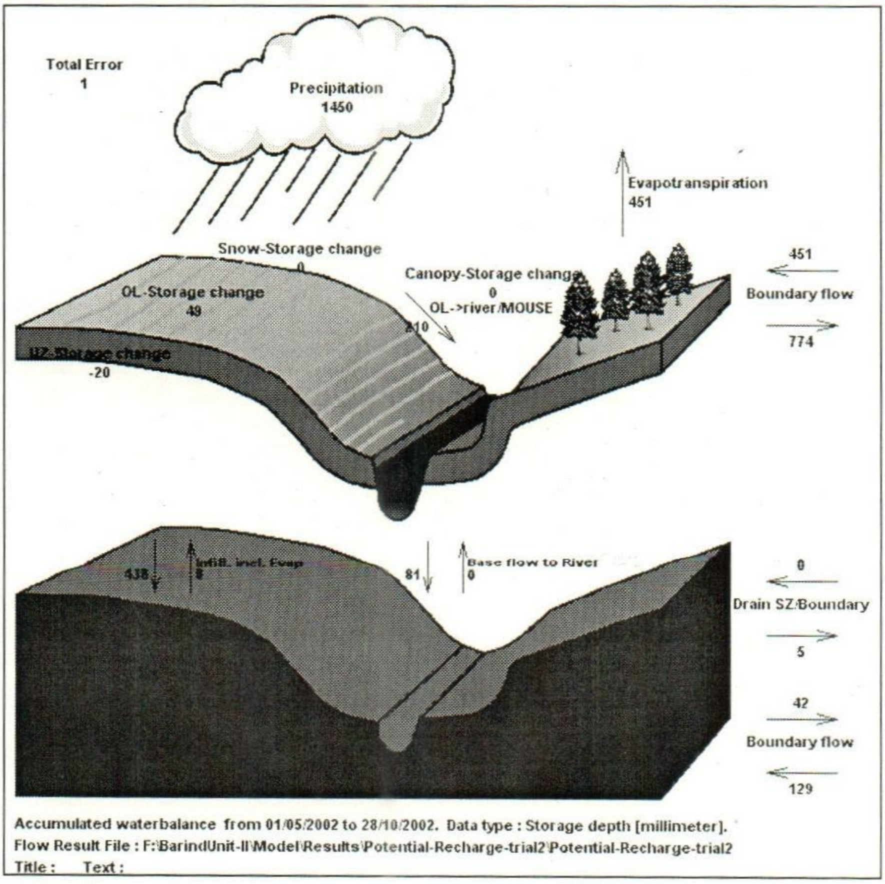

Figure 3: Water Balance Components of Akkelpur Upazila: May -October

Potential for further Irrigation Expansion

The model was simulated for the following three scenarios:

Option 0: Base Condition i.e. Existing Situation with 50% dependable rainfall

- Hydrological condition for the design year 2002 (avg. condition).
- Existing all features i.e. existing crop coverage, irrigation demands etc.

Option 1: Future Option with design year

- Hydrological condition for the year 2002 (avg. condition).
- Crop coverage for future condition; Upazila-wise 80% crop coverage of Boro for the cultivable area except few upazila e.g. Panchagarh, Tentulia.

Option 2: Future Option with extreme dry year

- Hydrological condition for the year 1994 (extreme dry condition).
- Crop coverage for future condition; Upazila-wise 80% crop coverage of Boro for the cultivable area except few upazila e.g. Panchagarh, Tentulia.

From the analysis of different options, it was revealed that, there is scope for irrigation expansion in some Upazilas under average hydrological condition (Table 2). A model simulation was carried out in for future dry season irrigation expansion by increasing Boro crop upto 80% in Upazilas where possible. However, Boro area in Panchagarh Sadar and Tentulia Upazilla has been proposed as 40% and 35% respectively due to natural constraints for development in those Upazilas. Further irrigation development for Joypurhat district has not been considered as the irrigated area of each Upazilla of that district is already found as more than 80%.

Table 1: Upazilawise Potential and Usable Groundwater Resources

Sl. No.	Districts	Upazila	Potential Recharge		Usable Resources	
			mm	Mm^3	mm	Mm^3
1	Panchagarh	Panchagarh Sadar	825	310	619	233
2		Tentulia	994	191	746	143
3	Dinajpur	Biral	715	257	536	193
4		Birampur	672	143	504	107
5		Chirirbandar	766	244	575	183
6		Dinajpur Sadar	729	256	547	192
7		Fulbari	776	181	582	136
8		Ghoraghat	695	105	521	79
9		Hakimpur	575	58	431	44
10		Khansama	668	119	501	89
11		Nawabganj	511	162	383	122
12		Parbatipur	605	241	454	181
13	Joypurhat	Akkelpur	581	85	436	64
14		Joypurhat Sadar	453	112	340	84
15		Kalai	799	131	599	98
16		Khetlal	677	97	508	73
17		Panchbibi	635	174	476	131

Table 2: Potential for Future Irrigation Expansion

Sl. No.	Upazila	Available Resources (Mm^3)		Net Boro Requirement (Mm^3)	Remaining Resources(Mm^3)		Increase of Irrigated area
		Potential	Usable		Potential	Usable	
1	Panchagarh Sadar	310	233	63	247	170	26%
2	Tentulia	191	143	40	151	103	11%
3	Biral	257	193	97	160	96	32%
4	Birampur	143	107	100	43	7	No Change
5	Chirirbandar	244	183	91	154	93	9%
6	Dinajpur Sadar	256	192	127	128	64	No Change
7	Fulbari	181	136	93	88	43	15%
8	Ghoraghat	105	79	64	41	14	16%
9	Hakimpur	58	44	32	26	12	No change
10	Khansama	119	89	48	71	41	22%
11	Nawabganj	162	121	108	54	13	22%
12	Parbatipur	241	181	101	139	79	10 %
13	Akkelpur	85	64	61	23	2	No Change
14	Joypurhat Sadar	112	84	105	7	--	No Change
15	Kalai	131	98	95	36	3	No Change
16	Khetlal	97	73	88	9	--	No Change
17	Panchbibi	174	130	137	37	--	No Change

Estimation of Tubewell Requirement

The number of DTWs and STWs required to satisfy the water demand for proposed irrigation expansion was estimated for shallow and deep. From the point of Farmers' preference on DTW because of lower irrigation cost than STW, DTWs are proposed in Shallow zone area. Upazila wise required no. of additional DTW is given in Table 3.

Table 3: Upazilawise Required DTWs

Upazila	Zone	Usable Resource (Mm^3)	SIWR (Mm^3)	DTW_Req	Existing no of		Proposed
					STW	DTW	DTW
Panchagarh Sadar	Shallow	233	32	180	3718	20	160
Tentulia	Shallow	143	16	177	1028	13	164
Biral	Shallow	151	39	0	10533	138	148
	Deep	42	16	286			
Birampur	Shallow	65	17	108	1883	298	185
	Deep	43	21	375			
Chirirbandar	Shallow	159	34	0	7400	155	22
	Deep	24	10	179			
Dinajpur Sadar	Shallow	160	45	0	12593	135	94
	Deep	32	13	232			
Fulbari	Shallow	59	14	0	5674	214	342
	Deep	77	31	556			
Ghoraghat	Shallow	18	6	0	1253	184	191
	Deep	61	21	375			

Upazila	Zone	Usable Resource (Mm^3)	SIWR (Mm^3)	DTW_Req	Existing no of		Proposed
					STW	DTW	DTW
Hakimpur	Shallow	25	8	80	594	186	19
	Deep	19	7	125			
Khansama	Shallow	89	23	25	3664	78	5
Nawabganj	Shallow	44	14	0	3462	282	254
	Deep	77	30	536			
Parbatipur	Shallow	110	24	0	8844	271	247
	Deep	71	29	518			
Akkelpur	Shallow	0		0	894	216	266
	Deep	64	27	482			
Joypurhat Sadar	Shallow	49	21	30	3275	232	66
	Deep	34	15	268			
Kalai	Shallow	5	2	0	1345	340	279
	Deep	93	35	619			
Khetlal	Shallow	4	2	0	691	318	164
	Deep	69	27	482			
Panchbibi	Shallow	40	13	0	3518	384	241
	Deep	90	35	625			

Note 1: DTW Capacity 2 cusec, average running hour 12 hrs, command area 22-28 ha and STW Capacity 0.5 cusec, average running hour 5 hrs, command area 2.65 ha

Note 2: Existing no. of DTW is based on DAE report (2008), except for Nawabganj which is taken from IWM field survey and existing no. of STW is based on DAE report (2008).

Economic Analysis

The economic analysis for the capital cost of the project is estimated to be Tk. 117,537.50 million and annual O&M cost is Tk. 8,569.59 million for consideration 100% of existing STW. It is estimated that annual paddy production will be increased to 1.80 million MT from existing. Due to the project, employment opportunities will be created for 17.1 million man-days during implementation and 38.6 million man-days of farm labour annually during operation. The risk analysis/sensitivity analysis shows minimum EIRR value of 29.85% when all costs increase by 30%.

Conclusions

The study was conducted to assess the availability of water resources and formulate strategy for its optimum utilisation as well as identification the required number of DTW to cover the additional cultivable land under irrigation. The set of conclusions based on different components of the study are as follows:

- Present dry period irrigation practice in the study area is mostly based on groundwater source because of limited availability of surface water. Groundwater abstraction comes through DTW and STW mode of technologies. A major portion of present irrigation coverage comes through STWs.

- Groundwater resources for the entire study area for the yield criteria of potential and usable are 2865 Mm^3 and 2149 Mm^3 respectively. Whereas the present irrigation requirement has been estimated as 1451 Mm^3. So that in the study area as a whole there is no shortage of water to meet the present water demand rather there is surplus water
- The surplus water can be used for irrigation expansion in some Upazilas where boro coverage is less than 80%. However, when usable recharge is considered, little shortage of water is observed in Joypurhat Sadar, Khetlal and Panchbibi. This is due to the fact that Boro coverage is already more than 80% in those Upazilas which implies to higher water requirement and crop diversification of less water consuming should be inspired in those Upazilas.
- Model simulation reveals that groundwater level goes below 6m from the surface partly or fully in most of the Upazilas of the southern part of the study area which mostly covers Dinajpur and Joypurhat districts.
- Considering usable recharge and future irrigation requirement for Boro, there is shortage of water in some Upazilas like Nawabganj, Joypurhat Sadar, Khetlal and Panchbibi where present Boro coverage is already more than 80%. Hence Boro coverage in those Upazilas should not have further expansion and crop diversification as well as surface water irrigation should be promoted to reduce groundwater irrigation.
- Five rubber dams at Kaliganj, Laxminarayni and Kashimpur on Karatoya, Mohanpur on Atrai and Masnandapur on Tangon, have been proposed to store water for providing dry period irrigation in 4,924 ha and supplementary irrigation in 25,540 ha along the respective river reaches. Non-perennial rivers, Punarbhaba, Tulshiganga and Kulik have been proposed to utilise only for wet period supplementary irrigation to 21,198 ha by constructing water control structures at Gouripur on Punarbhaba, Sonamukhi on Tulshiganga and Rauthnagar on Kulik.
- The project is expected to be sustainable if it is implemented following the study finding and suggestions which has been done using the state-of-the art technology.

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GROUNDWATER UTILIZATION IN IRRIGATION BUSINESS: A FIELD STUDY IN GHATAIL THANA OF TANGAIL DISTRICT

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Abstract

Groundwater is the main source of irrigation as well as drinking water of Bangladesh and one of the key factors making Bangladesh self-sufficient in food production. Importance of groundwater irrigation increased with the introduction of high yield variety (HYV) seeds in late sixties to meet the food demand of a growing population. Field surveys were carried out with a view to examine the availability of groundwater for Shallow Tube well (STW) irrigation business in Dighalkandi union (covering five mouza) under Ghatail Thana of Tangail District. The study revealed that the study area is suitable for STW irrigation business due to aquifer characteristics & groundwater availability at present situation. Increasing tendency of electric driven STW and declining trend of static groundwater table are observed in this study. The lithological stratum of subsurface has been discussed and some policy implications for the sustainable groundwater development as well as STW irrigation business have been recommended in this paper.

Introduction

Bangladesh is a home of 150 million people and the most densely populated country in the world suffers from food deficiency. Food security is the major challenge of our country since liberation for increasing food grain production to meet up the increasing demand. Intensive land use is a pre-requisite to overcome this demand. Boro rice alone contributed the highest of total rice production in Bangladesh during the period of 2007 (BER, 2007). Boro rice produced in Rabi season (dry season) and irrigation is used for its growing. There are two major sources of irrigation water in Bangladesh - surface and ground water. In 1970s, irrigation was mainly dependent on surface water, but during the last two decades the area irrigated by groundwater increased significantly.

Generally, farmers of our country install Shallow Tube Well (STW) to irrigate their own land especially for Boro paddy. Simultaneously they sell pumped water commercially to other farmers to earn money like other businesses. Even, without having any land farmer irrigating other's farmer land and doing irrigation business like other businesses. Following the process of STW irrigation development a form of business has been established in Bangladesh.

There is great potential of agricultural expansion by applying and increasing suitable technologies to utilize water resources. The economy of Tangail, as well as Bangladesh, depends on the proper harnessing of the water resources.

The growth of groundwater irrigation in the country, especially supplied by private STWs purchased by individuals for cash or credit at unsubsidized prices led to the

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emergence of irrigation water markets. Here, farmers without wells benefit and attain access to irrigation through purchase of groundwater from farmers owning wells.

Sustaining the groundwater markets and a growing groundwater-based irrigation economy is critical, for food security and for lifting millions of poor farmers out of poverty. Hence, understanding the facts of groundwater irrigation in contemporary Bangladesh and devising ways of supporting a sustainable development is critical. In changing situation over time some existing issues (the geographic and physical setting) regarding ground water irrigation extracting by STWs is essential to study in order to improve the irrigation business in Bangladesh. Keeping in view of above-cited issues the following objectives were considered to conduct this study.

The specific objectives of the study are:

i. To examine the availability of groundwater for STW irrigation business;
ii. Evaluation of the groundwater resource, with consideration of surface water interaction; and
iii. To draw policy guidelines for sustainability of groundwater irrigation in future.

Methodology

Selection of Study Areas

The study area was chosen from an intensively irrigated area in the north-east Bangladesh, which falls under the Indo-Gangetic basin region. Boro paddy is the main crop of this area, which is cultivated rabi season by groundwater irrigation. The major irrigation system of the area mainly depends on STW technology.

Data Sampling Techniques, Sample Size and Data Collection Procedure

Both STW owners and irrigators (water buyers) have been chosen as samples for this study. Complete survey technique was used for the STW owners. A total of 71 STW owners were found in the area operating their STW business. Out of 71 STW owners, data were collected from 64. Two-stage stratified random sampling technique was used to collect data from the irrigators (water buyers). Firstly, 20 STW owners were selected randomly from all and then five water buyers were also randomly chosen from each of the selected 20 STWs. During the selection of 20 STWs, equal weight was given to Diesel Operated STWs (DOS) and Electricity Operated STWs (EOS). So a total of 164 samples (64 STW owners, 50 DOS water buyers and 50 EOS water buyers) were used for this study.

The primary data were collected through two sets of interview schedule from STW owners and water buyers in January and February 2008. The locations of STWs were collected in the field by using Global Positioning System (GPS) and plot them on the mauza maps by identifying each plot. Four bore logs of BWDB present in the area were used for subsurface lithological and aquifer information. Data on groundwater level were collected from two observation wells of BWDB which are very close to the study area. The secondary data were collected from Bangladesh Bureau of Statistics (BBS, 2006), Bangladesh Economic Review (BER, 2007), Bangladesh Water Development Board (BWDB, 2007), Center for Environmental and Geographic Information Services (CEGIS, 2005) and Department of Land Records and Survey (DLRS, 2000).

Data Analysis, Results and Discussions

Location of the Study Area

The study area is within Dighalkandi union, which is under Ghatail Upazila of Tangail district in Bangladesh. The area is situated between latitude 24^0 25' 17.5" longitude 89^0 57' 26.84" and longitude 89^0 59' 10.10" latitude 24^0 24' 08.55". The study area comprises five Mauzas of Dighalkandi union (Figure 1). The Mauzas are Saitapara, Kurmushi, Kagmari Beltail, Kaijalipur and Shekh Shimul. The Mauza is the revenue boundary and it is the smallest land boundary unit available in Bangladesh. This Mauza map is actually cadastral map and it is similar to villages in terms of area. The total study area is about 376 hectares (Muaza GIS database, 2008).

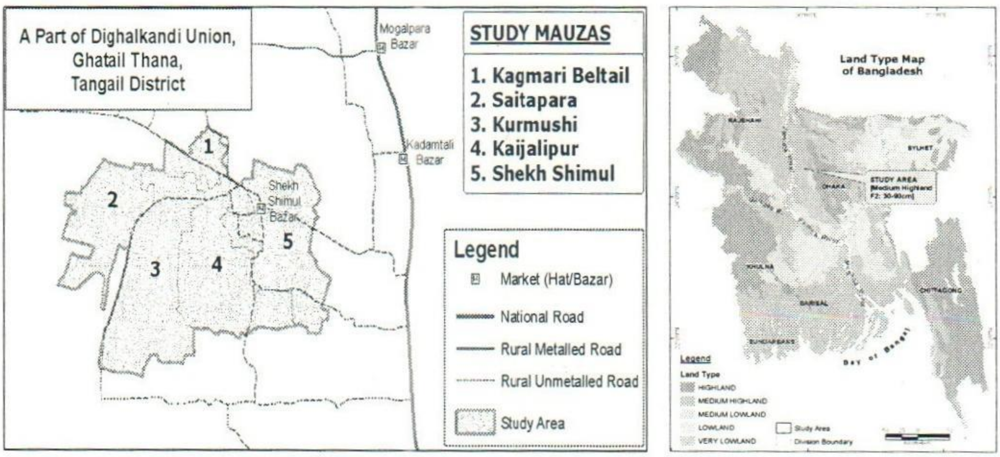


Figure 1: Showing the location of the study area and land type map of Bangladesh (Source: NWRD, 1997)

Land Type

Bangladesh is a flood prone country and it is divided into five land type based on the flood inundation level in normal flood years. The categories are presented in Table 1. The Land Type map of Bangladesh (Figure 1) shows that the study area is under Medium Highland (F2) category, which means that the area inundates at least 30 to 90 cm by flood water every year, even in normal flood year.

Table 1: Land types of Bangladesh based on flood inundation on normal years

Land Type	Description	Inundation Level (cm)
F1	Highland	0 – 30
F2	Medium Highland	30 – 90
F3	Medium Lowland	90 – 180
F4	Lowland	180 – 300
F5	Very Lowland	> 300

Source: NWRD Database, CEGIS, 1997

Flooding

RADARSAT ScanSAR 100m Wide Beam satellite image of the 3rd August 2007 is used to map the flood extent of the area. The flood map (Figure 2) shows that the study area was inundated by flood on the day, except the settlements and its adjoining higher elevated areas. The flooded areas are actually the agricultural lands where *Boro* rice is planted during dry season by groundwater irrigation.

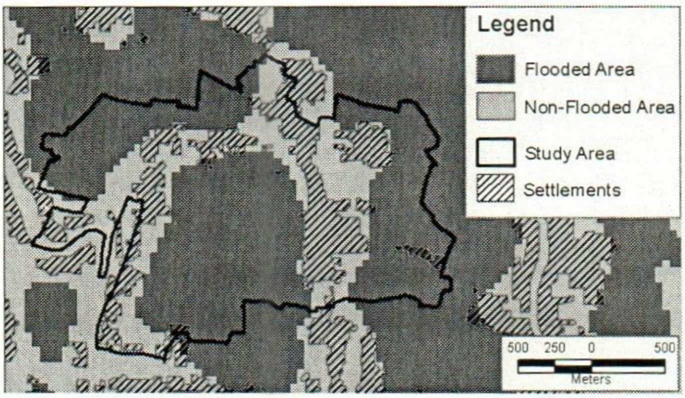
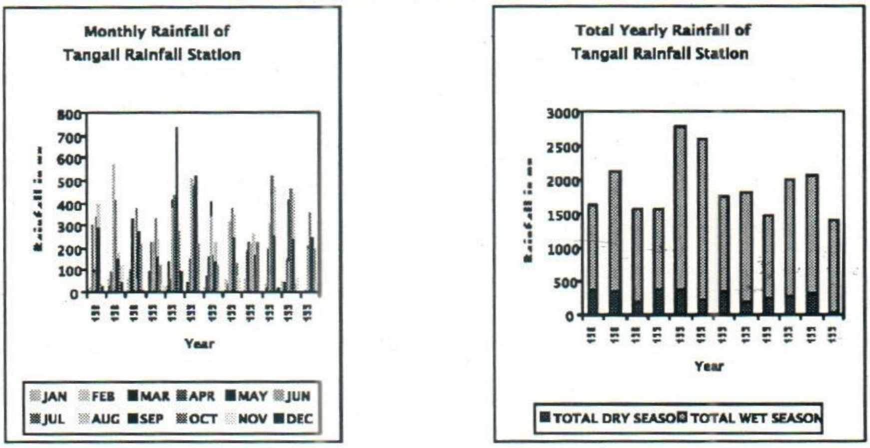


Figure 2: Flooded and non-flooded area mapped from ScanSAR Wide Beam 100m Radar image of the 3rd August 2007 (Source: CEGIS, 2007)

Rainfall

The yearly total average rainfall of the area is 1896mm, which was calculated from 12 years from 1987 to 1999 of the Tangail rainfall station. Recent data were not available for the area. The data also shows that the dry season (November to April) yearly average is 275mm, whereas the rainy season (May to October) yearly average is 1621mm. The maximum monthly rainfall recorded over the 12 years period in the station is 730mm on the month of September in 1991. Monthly rainfall (Figure 3a) and total yearly rainfall of Tangail rainfall station is shown in Figure 3b. The yearly total rainfall ranges from 1401 mm (in 1999) to 2792 (in 1991). The yearly rainfall indicates that the rainfall is sufficient enough for the area and is a good source of groundwater recharge.



(a) Monthly rainfall in Tangail (b) Yearly total rainfall in Tangail

Figure 3: Rainfall information of Tangail rainfall station (Source: NWRD Database, CEGIS, 2005)

Spatial Distribution of STWs and Energy Use

The STW business dynamics is changing in the study area like any other business. Therefore, changing phenomenon of tubewell locations and corresponding command areas also took place. There was 68 tubewells in 1997 (Mandal, 1997) in the study area. Among these only two were Deep Tubewells (DTW), one in Kaijalipur and the other one in Saitapara mauza. In 2006-07 irrigation period, the total number of tubewells rose to 71 with only one DTW was working in Kaijalipur mauza. The spatial distribution of the tubewells of 1997 and 2007 are shown in Figure 4a and distribution of diesel and electricity operated tubewells of 2007 are shown in Figure 4b. Among the 68 tubewells in 1997, 52 tubewells (76%) were run by diesel and the remaining 16 (24%) by electricity (Table 2). Whereas in 2006-07 irrigation period, only 31 tubewells (44%) were run by diesel and 40 (56%) by electricity (Table 2). It is evident that electricity driven tubewells are increasing. Table 2 shows that EOS was increased by 32% during the last 10 years.

Table 2: Mauza-wise number of tubewell and their energy use

Mauza	Number of STW		Energy Use			
	1997	2007	1997		2007	
			Diesel	Electricity	Diesel	Electricity
Kagmari Beltail	3	4	2	1	2	2
Kaijalipur	11	11	4	7	0	11
Kurmushi	27	21	25	2	10	11
Saitapara	12	17	12	0	14	3
Shekh Shimul	15	18	9	6	5	13
TOTAL	**68**	**71**	**52**	**16**	**31**	**40**

Source: Field survey January, 2008

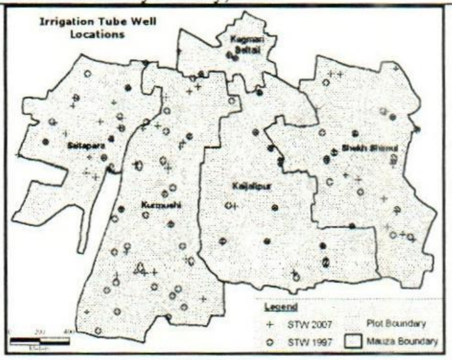


Figure 4a: Shallow Tubewell (STW) locations of 1997 and 2007

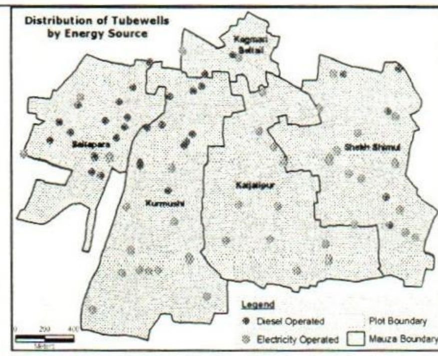


Figure 4b: Distribution of diesel and electricity operated tubewells in 2007 of the area

Figure 4b shows that the diesel operated tubewells are mainly concentrated in the northwestern part of the study area i.e. mainly Saitapara and northern part of Kurmushi mauzas. This is probably because these villages are far away from the existing electricity transformers or point of connection.

Topography

The total study area covers about 376 hectares. The area has rivers, sandbars, wetlands, settlements with homestead vegetations, agricultural lands, etc. The following table 3 presents different topographical features & their coverage areas of the study area.

Table 3: Different types of topographic features of the study area

Feature	Area (Hectare)	Percentage
Agricultural Land	288	77
Settlement	73	19
Waterbodies	15	4

Source: NWRD Database, CEGIS, 2005

Field survey revealed that more than 95% of the agricultural land is under irrigation during Boro season (dry season). The settlement class includes household, surrounding homestead vegetations, few surrounding homestead agricultural lands, etc. There are three wetland areas (*Beel*) present in and around the study area named as Kaicha, Naksala and Salenka Beel.

Lithology and Aquifer

Unit bore-log data is widely used to understand and interpret the subsurface lithology. Bangladesh Water Development Board (BWDB) has completed a groundwater survey in 1996 for all over the country as a whole and during the project they have collected unit bore-log data. The BWDB unit bore-log data is used for this study.

No unit bore-log is available inside the study area. Thus four unit bore-log data were selected from the surrounding areas that could represent the study area. The bore-logs consist of several small lithological variations. All the lithology classes of the area are generalized and grouped into ten major classes i.e. silt, very fine sand, fine to very fine sand, fine sand, fine to medium sand, medium sand, coarse sand,

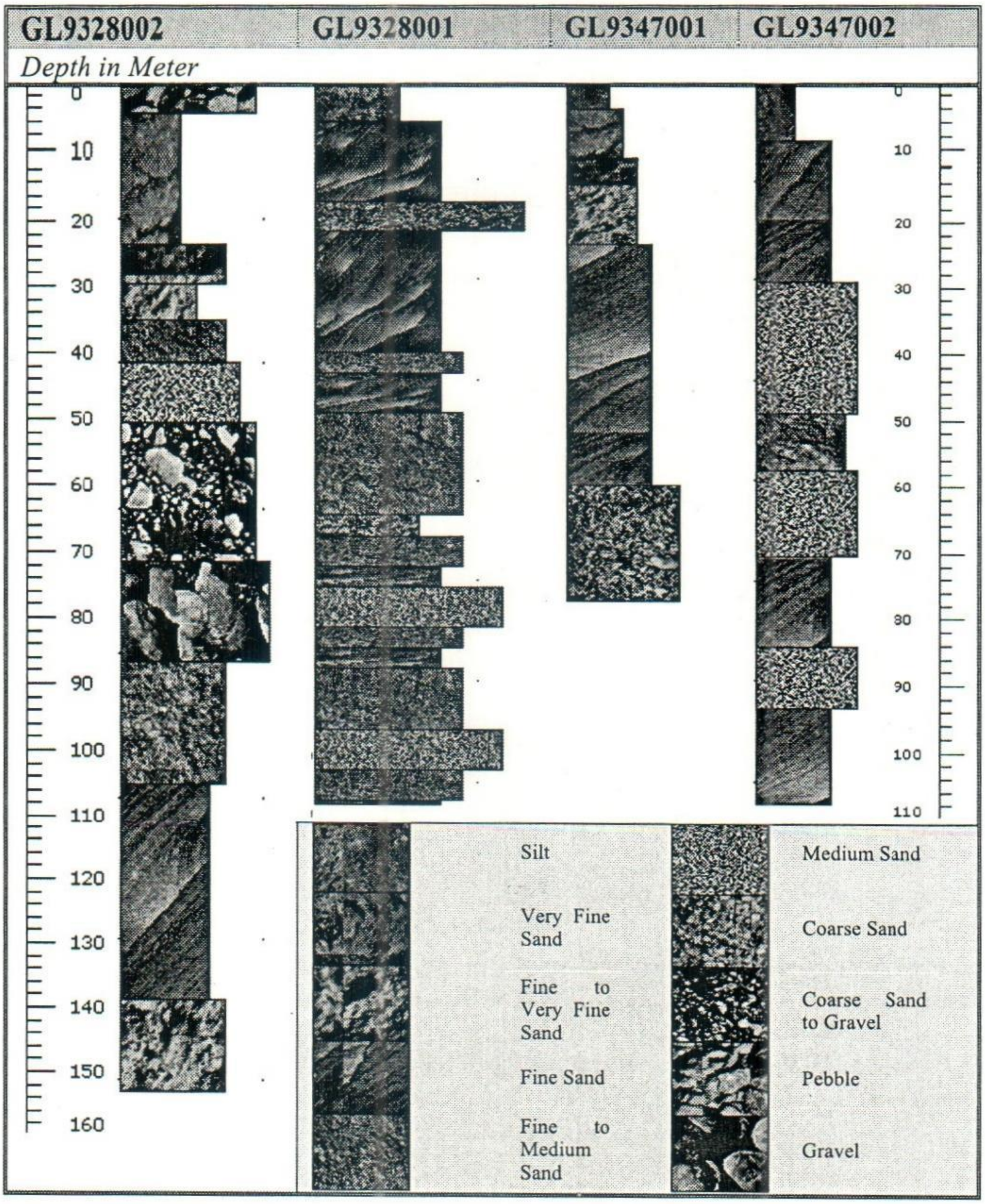


Figure 5: Generalized lithological stratum of four bore logs around the area (Source: BWDB, 1996)

coarse sand to gravel, pebble and gravel. Each of the lithology class is actually a mixture of different percentages of grain sizes. For generalization, the grain size that has the highest percentage of presence is considered as that class. Generalized lithological stratums for four bore-logs are shown in figure 5.

The lithological characteristics (up to 153 meter depth) of the study area represents that the subsurface is an unconfined aquifer, which is a good source of groundwater reservoir. It is the upper water bearing zone i.e. the upper (shallow) aquifer of the area (Zahid, 2006).

Groundwater Recharge and Suitability of Groundwater Irrigation

The subsurface of the area is governed by different grades of sand and sand is a good reservoir to store water and there is no confined layer present. Thus it is an unconfined aquifer. Each year the groundwater table of the study area comes to its normal water level by regaining water from different sources (Figure 6 & 7). Several perennial surface waterbodies [the *Kalidaha* River and *Beldaha Gang* (River), ponds and three Beels (wetlands) areas], plenty of rainfall (yearly average rainfall 1896mm), every year flooding [Medium Highland (F2) category], seepages of irrigation water are good sources of groundwater recharge in the study area. Above-mentioned sources can percolate through the sandy lithology and can contribute to recharge the groundwater table of the unconfined aquifer of the area. So, the source of groundwater is available in the study area and groundwater irrigation is suitable in the study area.

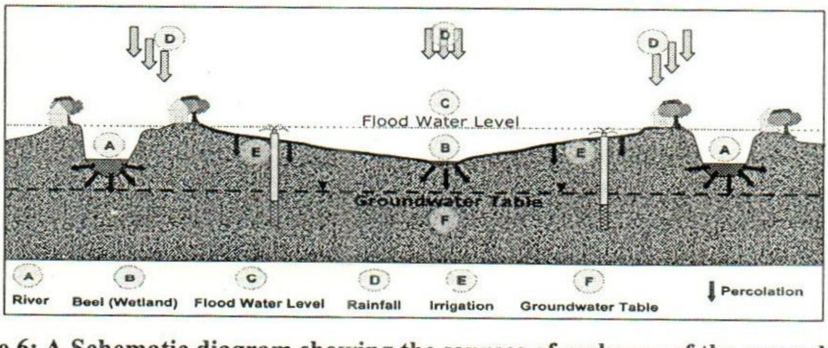


Figure 6: A Schematic diagram showing the sources of recharge of the groundwater table of the study area

Groundwater Table Hydrograph

There is no observation well in the study area to monitor and record the static groundwater table. Two nearby observation wells are present, which are maintained by Bangladesh Water Development Board and thus used for this study, which are under Ghatail Upazila.

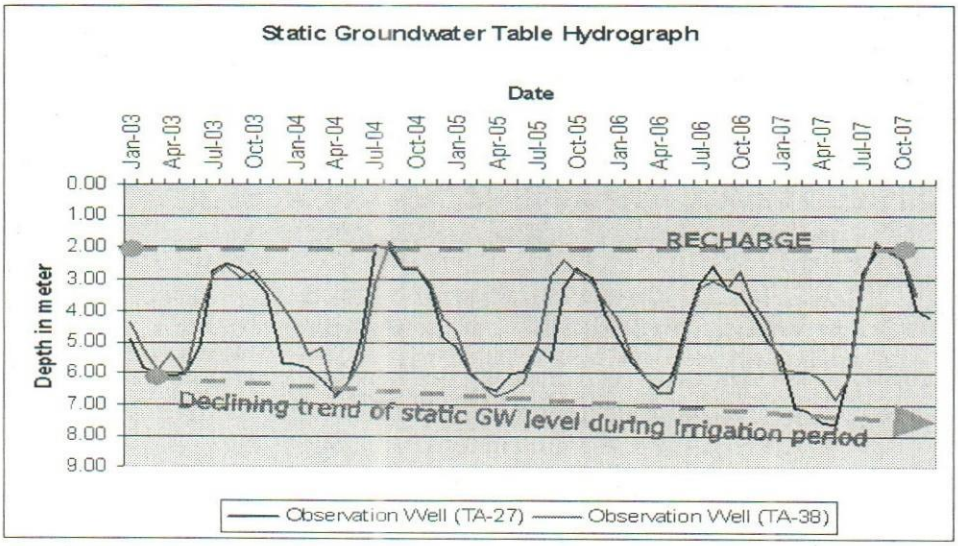
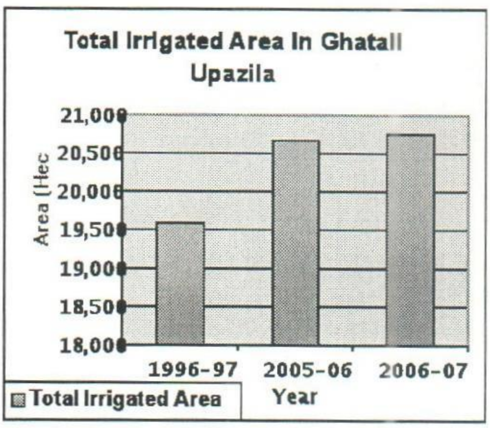


Figure 7: Static groundwater table hydrograph in Ghatail Upazila, Tangail (BWDB, 2007)

The hydrograph of the observation wells for last five years (2003 to 2007) shows that the static groundwater table maximum drops about 4 to 5 meters every year during the Boro season (March and April) due to the tubewell pumping for irrigation and it comes back to its original position during rainy season (Figure 7). This is the period (March and April) of the flowering stage of *Boro* crop when it requires maximum water and thus highest pumping occurs at that time. The normal depth of static groundwater table in the area is 2 to 2.5 m depth from the surface. The declining trend of the static groundwater table (Figure 7) is observed during the irrigation period is probably due to the increasing trend of irrigated area and also increasing number of electric driven tubewells (Figure 8) of the Ghatail upazila.



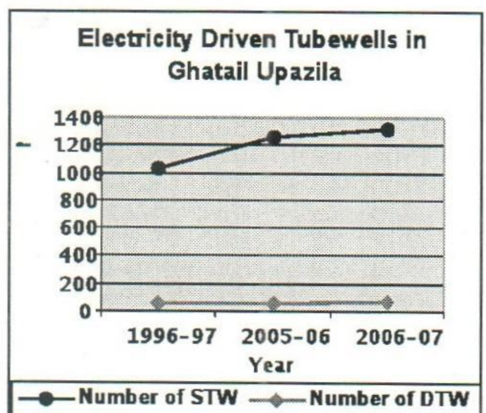


Figure 8: Increasing trend of irrigated area and electric tubewells (BADC, 2007)

Conclusions and Recommendations

Conclusions

The following conclusions are summarized from the study of irrigation business:

a) The study area is suitable for irrigation business due to geographical location, topography, lithology, aquifer characteristics and groundwater availability.

b) Increasing tendency of electricity driven STW is found in the study areas.

c) The declining trend of the static groundwater table is observed during the irrigation period each year is probably due to the increasing trend of irrigated area and also increasing number of electric driven tubewells.

Policy Implications (Recommendations)

The following recommendations are made to improve the quality of STW irrigation Business in Bangladesh:

Though the study area is suitable for irrigation business at present situation, but only 4% of water bodies (which is limited) are decreasing with increasing of settlement and groundwater utilization is increasing with the introduction of high yield variety (HYV) seeds to meet the food demand of a growing population. That's why government should take necessary steps (such as well spacing, utilization of groundwater considering safe yield, preservation of existing water bodies etc.) through integrated participatory approach to maintain for sustainable uses of groundwater resources.

Acknowledgement

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PHYSICAL MODELLING FOR REFINEMENT AND VERIFICATION OF HYDRAULIC DESIGN OF GORAI OFF-TAKE INTERVENTIONS

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Abstract

Gorai off-take interventions have been planned under Ganges Barrage Project to divert a part of Ganges flow through the Gorai during dry season. Scale model investigation is an effective tool in providing necessary support for finalization of hydraulic design of such interventions. An interactive and collaborative modelling approach can be followed in this regard involving the designers which gives them confidence in the future performance of the project. The main objective of physical morphological modelling is refinement and verification of hydraulic design of the off-take structure and associated works recommanded from MIKE21C numerical model investigations. Geometric scales of the model have been set at 1:300 and 1:60 for length and depth respectively. Since scale effects are inherent to this type of sand bed model an assessment of scale effects is made to highlight the limitations and possibilities of the model. The off-take structure consists of 15 bays of 18m width, navigation lock (20m wide), fish pass (20m wide) and hydropower plant (36m wide). The associated works include guide bunds and flow divider. The efficacy of the proposed off-take strucrure has been verified in terms of river approach to the off-take structure, erosion/sedimentation at the off-take, discharge in the Gorai as a percentage of total Ganges flow, flow patern and scour depth near the structures etc. for dominant discharge. The model is calibrated first for the same discharge and guide bunds, off-take structure and flow divider have been introduced in the model sequentially to investigate their performance individually and combinedly compared to the base condition. During every test sediment is fed at the inflow section and the model run is continued until dynamic equlibrium condition is reached. It is observed from the test results that the introducion of the guide bunds alone as per design causes somewhat reduction in the average bed level at the off-take and also reduces conveyance of the Gorai channel compared to the base condition. On the other hand, proposed flow divider causes an increase in the conveyance and further reduction in the average bed level at the off-take. It apopears that proposed layout of off-take structure, guide bunds and flow divider combinedly produces favourable river approach to the off-take structure. However, refinement in the alignment of the right guide bund is needed to reduce frontal flow attack at its upstream head and to ensure smooth entry of flow into the Gorai.

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Introduction

The Gorai River is an important source of fresh water supply to the South-West Region of Bangladesh. It is the only remaining major spill channel of the Ganges river flowing through the South-West Region. For the last 20 years, or so, the dry season (January-May) discharges in the Gorai river have discresed resulting in an increased saline water intrusion with negative environmental impact on the coastal area of South-West Region and Khulna. The Sundarbans area is also believed to be adversely affected. It is feared that the decresed dry season flows will lead to the siltation at the offtaking point of the Gorai channel to an extent that the river may be permanently disconnected from the Ganges. This would deprive the region of important source of fresh water supply.

Under such circumstances , a barrage across the Ganges River is envisaged as a prerequisite to ensure proper use of the Ganges water for the benefit of the people living in the Ganges Dependent Area (GDA) and to make up for the negative impacts of the continuous withdrawal of water upstream. With this end in view, Bangladesh Water Development Board (BWDB), Ministry of Water Resources has taken up " Feasibility Study and Detailed Engineering for Ganges Barrage Project." River Research Institute (RRI), Bangladesh has provided sub-consultancy services for carrying out physical model investigation to support the feasibility study for Ganges Barrage Project. Six numbers of physical model studies carried out separately at RRI under this assignment. Gorai off-take structure model is one of them and the model is designed and constructed as a distorted morphological model covering about 15 km stretch of the Ganges extending from 10km upstream of the Gorai off-take to 5km downstream of the same. The model also covers 5.5km stretch of the Gorai starting from the off-take. A part width of the Ganges corresponding to 61% of its dominant discharge and full width of the Ganges has been reproduced in the model. The model has been constructed in an open-air bed of 125m × 60m. The objectives of the study are to investigate the effects of the proposed interventions at Gorai off-take on existing flow pattern, bed erosion/accretion at the off-take and upstream and downstream of the off-take structure and discharge distribution in the Gorai and also to determine the depth and extent of local scour near the structures qualitatively. The model has been constructed with geometric scales of L_r =300 and h_r =60. In a morphological physical model with bed of prototype sand it is almost impossible to fulfil all the scale conditions related to three governing processes namely flow, sediment transport and bed topography simultaneously. Therefore, compromise among different scale conditions has been sought. Scale factor for different basic and derived parameters including depth should come out from calibration of the model. The exact scale of these parameters has, therefore, been determined during the calibration of the model based on the ability of the model to reproduce the channel morphology as observed in the prototype. Scale effects are inherent to this type of model and therefore, the model has limitations in reproducing local details quantitatively. Since the model is intended to verify the efficacy of the proposed interventions at the Gorai off-take guide bunds, off-take structure and flow divider have been introduced in the model sequentially as per design to investigate their individual and combined effects on existing flow pattern and river morphology at and around the off-take. The design of the proposed interventions has been prepared based on MIKE21C numerical model outcomes and tested in the physical model for finalization of their hydraulic design parameters.

Methodology

Model Set-up

The model setup includes model scale, model discharge, model grid, bathymetry and bank line, water recirculation system, water level gauging stations, tailgates and existing and proposed structures. Based on the field survey data a model grid consisting of sufficient number of grid points both inside and outside of the model area has been installed to construct the bank line, bathymetry and engineering interventions correctly according to their exact positions. The bathymetry of the model has been reproduced according to the bathymetric survey data of monsoon 2010. The model layout is shown in Figure 1.

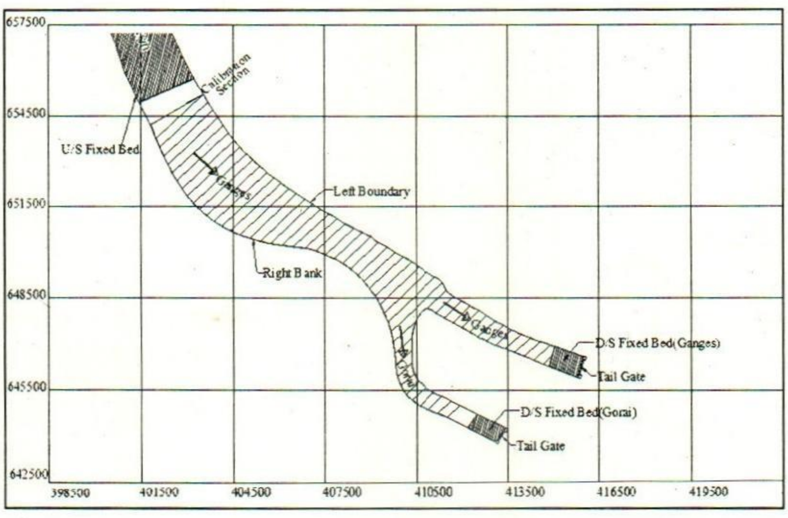

Figure 1: Layout of the model

Model Design

In the design of the 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 flow, sediment transport and bed topography. For scaling and design of the morphological model different scale conditions for flow and sediment transport have been taken into account.

The design of the model has been made after analyzing the field data. A constant discharge (domunant discharge) has been considered for the design as this discharge is representative for major morphological developments. In a sand bed model conflicting scale conditions arise in order to reproduce flow and sediment transport conditions simultaneously. A compromise is, therefore, sought that has led to scale effects. The main conclusions for the model to reproduce morphological phenomena and bed topography are:

- the velocity scale should be in between ideal velocity scale ($V_r^2 = C_r^2 D_r \Delta_r$) and velocity scale according to Froude condition.
- a supplementary slope should be added.
- sediment should be supplied at the upstream end of the model ensuring that the bed level immediately downstream of the model inflow section neither gets scoured nor aggraded.

Model and prototype roughness play a vital role in selecting the different scale factors. Based on the previous experience, a Chezy value of 29 $m^{1/2}/s$ has been considered as model roughness for scaling. On the other hand, the prototype roughness has been considered as 88 $m^{1/2}/s$.

Model Calibration

Calibration of the model has been conducted in existing condition of the river (without any proposed interventions) and for dominant discharge to ensure that the model is able to reproduce the flow condition, morphological behaviour and sediment transport in the field. The main focus of the model calibration has been concentrated on three governing processes namely flow, sediment transport and changes in bed topography.

The measurements during the calibration include water levels, bed levels, point velocities, float tracks and discharges. Calibration of water surface slope aims to achieve a dynamic equilibrium in water surface slope in the model. Calibration of discharge distribution aims to achieve a more or less constant discharge through the Gorai as a percentage of total discharge of the Ganges upstream. Calibration of sediment transport depends on the reliable field data. During calibration test sediment was 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 0.00005 m^3/s for dominant discharge. It is to be noted here that some deviations around this value is caused by bed forms that migrate through the model. The sediment transport could not be compared as the sediment transport measurements in the prototype for the dominant discharge is not available. Moreover, the sediment transport in the model is very difficult to compare with the prototype measurements as the model has some scale effects in reproduction of the magnitude and direction of sediment transport. The scales of the different basic and derived parameters have been determined based on the calibration test results. It is to be noted here that scales of water depth and velocity vary at different locations of the model signifying the presence of scale effects. However, the range of variation is less. During the calibration test the model is allowed to adjust its water surface slope and bed slope. The equilibrium water level slope is 0.0011. Calibration of morphology has been concentrated on achieving an equilibrium condition in the model bed configuration similar to the initial bathymetry (monsoon 2010) of the model. Table 1 presents some characteristic parameters of the dominating processes for the overall morphological model after calibration.

Assessment of Scale Effects

On the basis of Table 1 the scale of the following parameters are determined as:

$n_A = 1.20$, where, A is co-efficient in the expression for lateral slope
$n_{Ks} = 1.25$, where, K_s is correction co-efficient in the expression of bed slope
$n_{Fs} = 0.50$, where Fs is equal to CI_1/I_2
$n_{h/R} = 0.193$, where h/R is the ratio between h (water depth) and R (radius of curvature)
$n_{D/h} = 0.0145$, where D/h is the ratio between D (average grain size) and h (water depth)

In order to satisfy roughness condition the model Chezy value should be 2.27 times less than the prototype Chezy value. However, according to Table 1 the scale factor for Chezy value is 3.03. It has an effect on the flow lines. The flow lines in the model are more curved. Since in river models the banks give additional guidance, hence this effect might be less serious. Higher roughness may also attract more flow in between active measures like groynes. In order to satisfy the roughness condition for bed topography the scale factor for Chezy value should be 3.53, which is not much higher than the obtained scale factor. According to Table 1 initiation and ceasing of motion is on scale of about 25. However, for equal erosion and deposition it should be reproduced on the vertical scale (58). Hence there is a scale effect and too much sedimentation is likely to occur on the char. The shape of the concentration vertical is determined by suspension number (u_*/w). For accurate reproduction of concentration vertical in the model, the scale of the suspension parameter should be on scale 1. It can be seen from Table 1 that the scale of suspension parameter is about 2.23. It means that the quantities of sediment transported as suspended sediment in the model are smaller than that in prototype. Visual observations in the model also suggest the same. This can also be explained by the fact that the value of Shields parameter in the model is low.

Table 1: Characteristics parameters of the calibrated model

Parameter	Unit	Prototype	Model	Scale factor
Basic Parameters				
Discharge (Q)	m^3/s	25000	0.287	87108
Width (W)	m	1140	3.8	300
Cross-sectional area (A)	m^2	12209	0.7	17414
Depth (D)	m	10.71	0.18	58
Velocity (V)	m/s	2.05	0.41	5.0
Slope (i)	-	0.00005	0.0011	0.045
D_{50}	m	0.00015	0.00018	0.83
Sediment transport (S)	m^3/s	7.70	0.00005	154000
Sediment density	(kg/m^3)	2650.0	2650.0	1
Relative density	-	1.65	1.65	1
Derived Parameters				
Chezy (C)	$m^{1/2}/s$	88	29	3.03
Froude Number (F_r)	-	0.2	0.3	0.67
Shear velocity (u_*)	m/s	0.072	0.04	1.62
Fall Velocity (w)	m/s	0.016	0.022	0.73
Friction factor (f)	-	0.0025	0.0237	1.11
Sheilds parameter (θ)	-	2.16	0.68	3.18
Rouse (suspension) number ($u_{*/w}$)	-	4.52	2.03	2.23
Non-dimensional sediment discharge (Φ)	-	273.63	1.605	170.45
Non-dimensional particle parameter (D_*)	-	3.428	4.114	0.83
Critical velocity (V_{cr})	m/s	0.41	0.27	1.52
Critical Shields (θc)	-	0.07	0.057	1.23
Critical depth	m	0.39	0.0156	25.0
Weigh function of influence of bed slope ($f\theta$)	-	0.46	0.92	0.5
Adaptation length for flow (λ_w)	m	3552	6.64	535.0
Adaptation length of sediment redistribution (λ_s)	m	6777.0	7.36	920.4
Interaction parameter (IP)	-	1.91	1.145	1.67
1D bed form celerity (C_{bw})	m/s	0.00315	0.000357	8.82
1D morphological time scale	days	55.0	1.62	34.0

The direction of the sediment transport is an important issue at off-takes. In models with dominant suspended sediment transport, the direction of the suspended load should be simulated on scale. No scale effects are present when the following condition is fulfilled: $n_{Fs}n_{h/R} = 1$, where n stands for scale ratio.

The scale of h/R is about 0.193. For Cp = 88 $m^{1/2}/s$ and Cm = 29 $m^{1/2}/s$ as function of the values of suspension number in prototype and in model the scale factor of Fs is about 0.47. The product of the scale factors of Fs and h/R is about 0.091. It means the lateral transport due to helical flow is about 11 times stronger in the model than in the prototype. The consequence of the scale effect identified here is that in the model more sediment enters into the Gorai compared to the prototype situation. In a distorted model such scale effects are unavoidable. The sediment entering the Gorai is the combination of the lateral transport and the convective transport by the main flow. Because of this combination, it is not easily possible to correct this scale effect numerically.The scale rule for the axisymmetric lateral slope for the case of a combination of bed load and suspended load is:

$n_{sin\beta} = n_{Ks} n_A n_{f(\theta)} n_{h/R}$The slope should be reproduced on the scale of the distortion, hence the following scale condition holds: $n_{Ks} n_A n_{f(\theta)} = 1$

Based on the derived values of n_{Ks}, n_A *and* $n_{f(\theta)}$, the value of $n_{Ks} n_A n_{f(0)}$ becomes about 0.75. It is therefore clear that there are scale effects in reproduction of bed topography in the model which explains the steeper slope in the model. The lateral slope is reproduced at a scale of 0.1 instead of 0.19. Hence the lateral slopes for axisymmetric conditions are about 10 times steeper in the model compared to the prototype condition. About half of this corresponds to a scale effect. This is in particular due to scale effect in $f(\theta)$ due to difference in h/D value in the model and prototype.

Test Procedures

Four application tests have been conducted in the model that include base run (Test T1) for dominant discharge. These tests aim at assessment of likely future hydro-morphological developments in the study reach particularly at and around the Gorai off-take with and without intervention conditions, individual and combined performance of the proposed interventions and appropriateness of their placement, dimension and orientation. Test T2 is conducted with only guide bunds in place as per design whereas in Test T3 off-take structure has been introduced in addition to the guide bunds. Test T4, on the other hand, has been conducted with guide bunds, off-take structure and flow divider in place.Layouts of the interventions have been determined first by numerical model investigations. During the tests bed levels have been measured at selected cross-sections with particular care at the beginning of the upstream mobile bed section and water levels have been obtained from point gauges that are installed along the model bank and within the river channel at regular intervals to observe the developments in the bed profiles and water surface profiles. Sediment supply is done manually at the upstream inflow section. The quantity to be fed is determined by close observation of sediment transport phenomena in the upstream fixed bed part of the model. Sediment supply rates and distributions are recorded for the duration of each test and at the end sediment feeding rate is computed. The measurements made in the model include water levels, bed levels, velocities and flow fields. A scale from a reference wire is used to measure bed levels manually. An A-OTT current meter is used to measure the flow velocity.

Test Results and Discussions

Base Condition

The calibrated bathymetry has been considered as initial bathymetry for base run (Test T1). The boundary conditions for this model run have been derived from the results of MIKE 21C model. The model run has been continued so long as dynamic equilibrium condition is attained. Necessary measurements have been taken both in development stage and in equilibrium condition. The results have been discussed hereafter.

The velocity field in the model in base condition and at (quasi) equilibrium state is shown in Figure 2. In the upstream of Talbaria high velocity flow channel occurs along the left boundary of the model and at Talbaria flow concentration occurs near the right bank. Therefrom flow continues towards the downstream and flow separation occurs up to the downstream tip of the bifurcation point between the Ganges and the Gorai. The maximum velocity at Talbaria is about 2.62m/s and occurs at about 350m away from the right bank. Just downstream, of Talbaria, flow separation occurs and a part of the Ganges flow enters into the Gorai. The cross-sectional average flow velocity found at the Gorai mouth is 1.16 m/s. The discharge measurement has been conducted several times during the test run. It is found from the calculated discharges (with little variations) that the discharge through the Gorai is about 10% of the total Ganges discharge in base condition. The equilibrium water level in the model is found to be 0.0011 for the Ganges reach and for the Ganges-Gorai reach, it is about 0.0012. After model run in base condition the bed levels have been recorded (Figure 3). Based on the measurements the bed levels in the model have been determined in model values. Prototype values have also been calculated where necessary. It is observed from the difference of initial and after run bed level that deposition has occurred at the Gorai mouth particularly on the attached char (Figure 3) along the left bank. The char deposition may be attributed to the scale effects. However, bed scour is observed at the Gorai mouth along the left bank. It is to be mentioned here that since the scale effect is present in both base and intervention conditions the effect of the different structural measures compared to the base condition can be fairly assessed qualitatively.

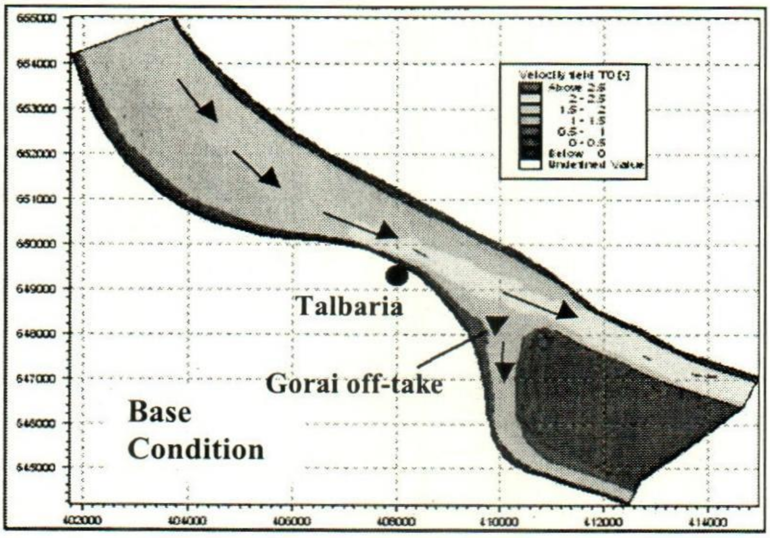

Figure 2: Velocity field in base condition (T1)

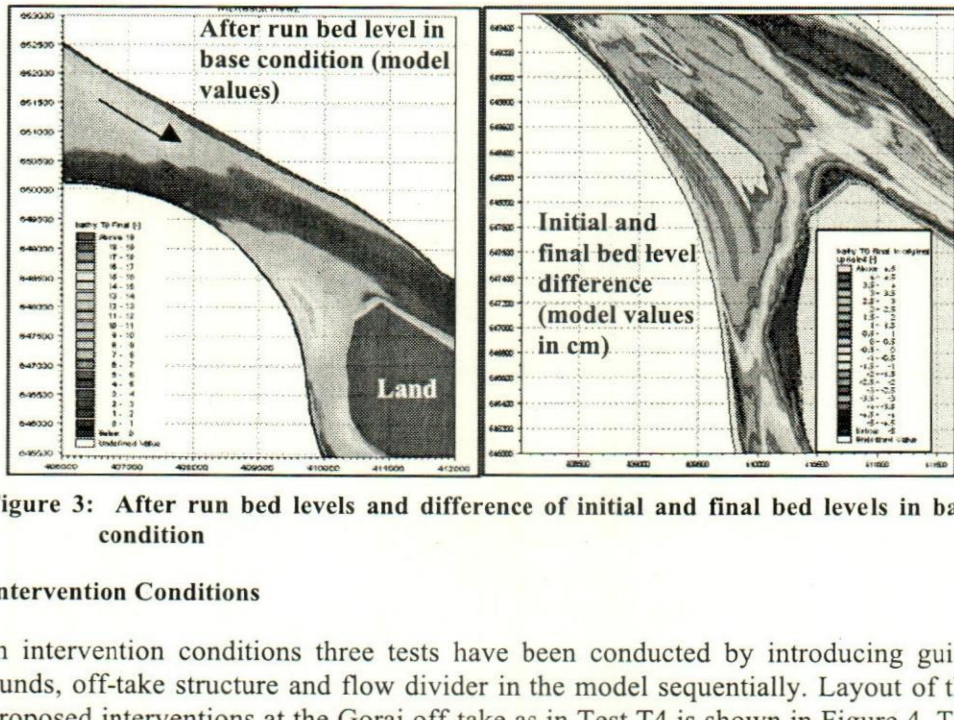

Figure 3: After run bed levels and difference of initial and final bed levels in base condition

Intervention Conditions

In intervention conditions three tests have been conducted by introducing guide bunds, off-take structure and flow divider in the model sequentially. Layout of the proposed interventions at the Gorai off-take as in Test T4 is shown in Figure 4. The tests are conducted for dominant dischage and model run is continued until dynamic equilibrium condition is attained. The velocity fields in the study reach in three different test conditions appear in Figure 5.

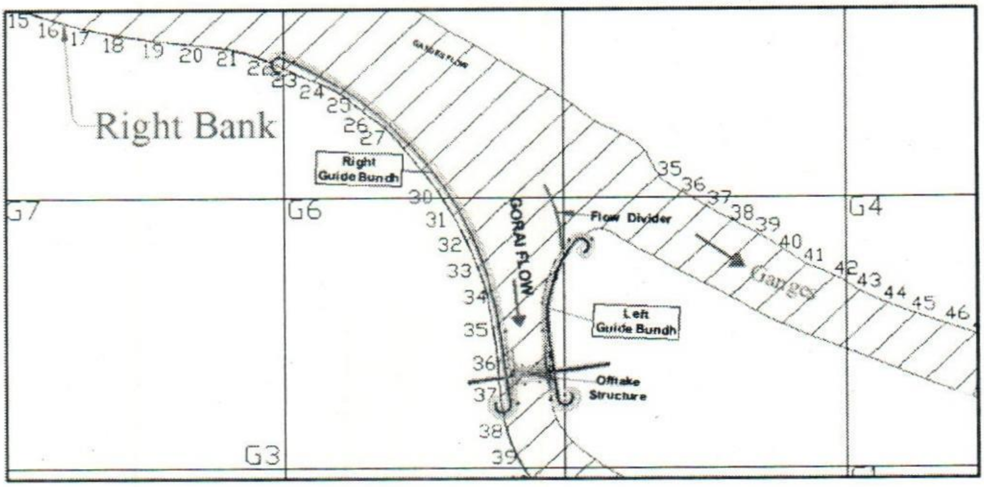

Figure 4: Layout of the proposed interventions at the Gorai off-take as in Test T4

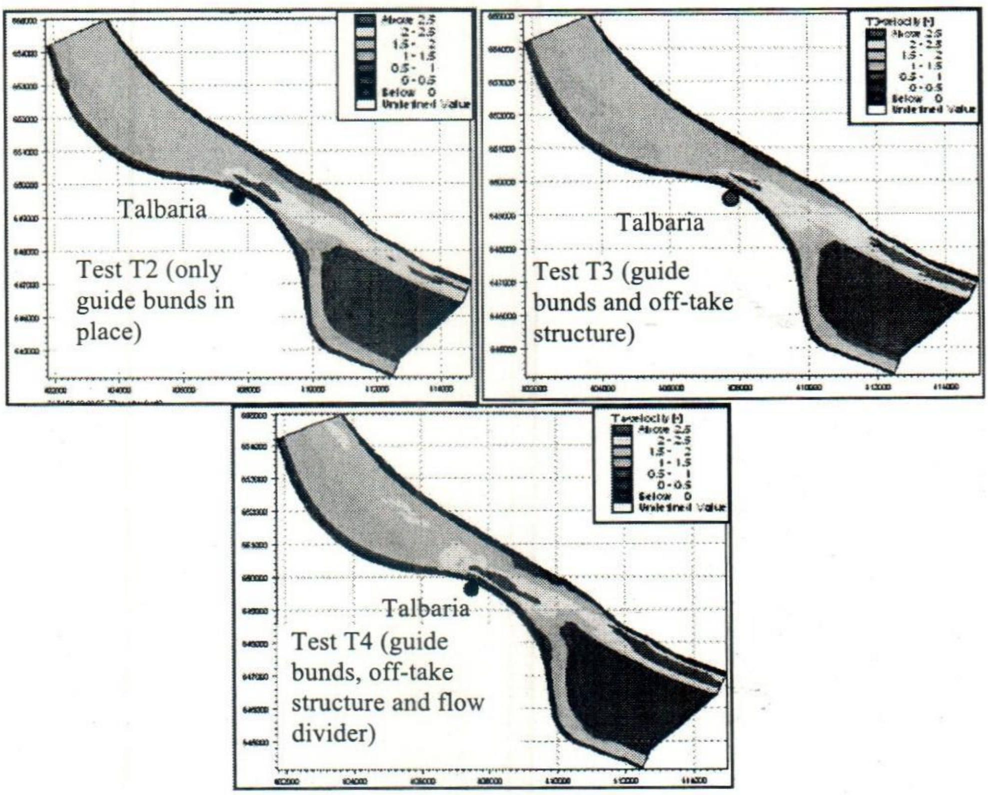

Figure 5: Velocity fields in the study reach in three different test conditions

It is observed from the flow fields that for all test conditions flow concentration occurs near the right bank at Talbaria as in base condition. However, flow concentration and maxium velocity there are the highest for flow divider in place.

The maximum velocity at the upstream termination of the right guide bund is found to be as high as 3.3m/s. The cross-sectional average velocity at the Gorai mouth in base condition (1.16m/s) experiences a decrease after introduction of guide bunds (1.09m/s) and off-take structure due to reduction in the conveyance of the Gorai channel. The flow divider, on the other hand, causes an increase in this velocity (1.24m/s) compared to the base condition. The discharge distribution (Q_{Gorai}/Q_{Ganges}) in the Gorai in base and intervention condiions is shown in Table 2.

Table 2: Dischage distribution in the Gorai in base and intervention conditions

Tests	Q_{Gorai}/Q_{Ganges}
T1 (Base run)	10
T2 (with guide bunds in place)	7
T3 (with guide bunds and off-take structure in place)	7.5
T4 (with guide bunds, off-take structure and flow divider in place)	9

It is observed that a large scour hole is formed (Figure 6) around the head of the flow divider that attracts Ganges flow and a part of this flow finds its way into the Gorai. The flow divider also improves the velocity distribution along the cross-section in the upstream of the off-take structure in terms of maximum to minimum velocity ratio. The afflux at the off-take structuure for dominant discharge is found to be about 10cm with and without flow divider in place. The afflux at flow divider is, however, about 40cm. The after run bed configuaration in Test T4 is shown in Figure 7.

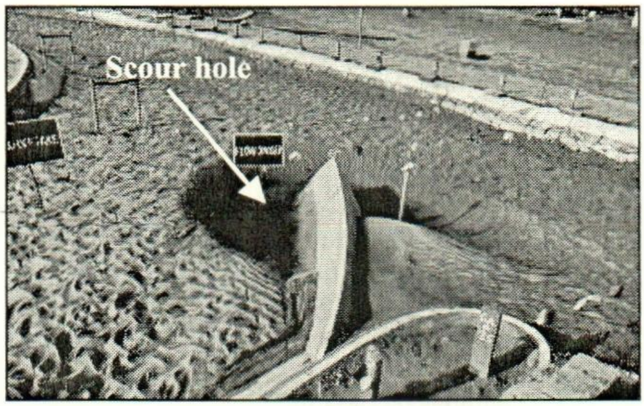

Figure 6: Local scour situation around the head of the flow divider

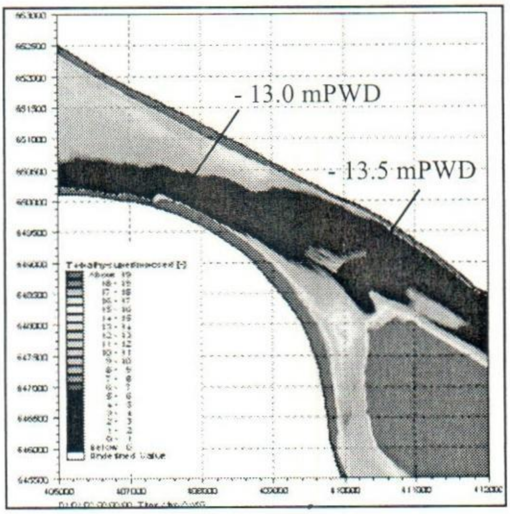

Figure 7: After run equilibrium bed levels at and around the off-take and near the proposed structures

The minimum bed level near the flow divider and at the upstream termination of the right guide bund is found to be -13.5mPWD and -13.0mPWD respectively. However, there might have influence of scale effects in reproduction of depth and extent of local scour. Noticeable deposition is observed at the downstream of flow divider along the right bank of the Ganges. It is observed from the comparison of bed levels at the Gorai mouth and in the immediate upstream and downstream of the off-take structure that the average bed level has remained almost the same as in Test T3 situation at the Gorai mouth. However, downstream of the Gorai mouth the average bed level is found to have decreased in various amounts (ranging from 17cm to 55cm in prototype values) compared to the Test T3 situation.

Due to model distortion and the associated scale effects, the observed scour depths are not directly comparable to prototype values. The results found in the model are more qualitative than quantitative. Scale effects are inherent in such type of distorted morphological model that use fine sand as bed material; however, it is notable that the impact of these scale effects will be similar in the tests conducted for different components of Gorai Off-take structure. These qualitative results can be used in combination with results from the associated numerical model studies. Detailed (undistorted) sectional model is recommended to assist in developing a final design for different components of Gorai Off-take structure.

Conclusions and Recommendations

Conclusions

The followings are some of the conclusions drawn from the study:

- With proposed interventions at Gorai off-take in place the distribution in the Gorai is 9%, which is 1% less than that in base condition. Without flow divider the discharge distribution is 7.5%.

- The proposed guide bunds reduce the width of the Gorai channel from the Gorai mouth to the downstream termination of the guide bunds. As a result, average bed level at this river stretch goes down to some extent (of varying amount). At the same location, option of placing flow divider causes further reduction in the average bed level.
- With proposed interventions in place large scour is observed at the upstream head of right guide bund and flow divider. The observed minimum bed level is -13.0mpWD and -13.5mPWD respectively.
- With flow divider in place, large deposition is observed in the downstream of the divider along the right bank of the Ganges.
- For submerged unregulated flow condition the afflux at the off-take structure is found to be about 0.10 m for average discharge and the afflux at the flow divider is 0.40 m.
- The maximum velocity at the immediate downstream of the upstream termination of right guide bund is about 3.3 m/s.
- The proposed interventions result in favorable river approach to the off-take structure.

Recommendations

- The hydraulic design parameters obtained from this model study can be utilized in design of proposed structures at the Gorai off-take.
- The proposed flow divider appears to be effective in increasing flow distribution into the Gorai.
- Under water constructional aspects of the off-take structure and associated works has not been investigated in this model study.
- If any unexpected developments are observed during the construction and operation phases of the off-take structure and associated works, it might be addressed by further physical model testing.

Acknowledgements

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LABORATORY STUDY ON CHANNEL ROUTING

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Abstract

This study was conducted to simulate channel routing in a mini artificial channel in the hydraulic laboratory of the Department of Irrigation and Water Management in Bangladesh Agricultural University, Mymensingh. A trapezoidal channel of 1760 cm long lined with cement-sand mortar having bottom width of 2 cm, side slope 0.5:1 and an average depth of 8.5 cm was constructed with a longitudinal slope of 0.0011 in soil bed on a metallic tray. Flood flows were simulated as inflow to the channel by supplying water at a varying rate by a pump via a flow measuring device. The outflow from the channel was also passed through a measuring device. Inflow and outflow rates were simultaneously measured at 30 sec intervals. Three trials of routing were conducted with different peaks of inflow hydrographs. Muskingum model was then fitted to the inflow-outflow hydrographs obtained in each of the trials. Routing effects of the channel were evident in the plots of inflow-outflow hydrographs. In each of the three trials, the flood wave was found to be attenuated and lagged. This study also revealed that attenuation was not related to peak value of inflow hydrograph. However, the values of x and K of the Muskingum model increased with the increase of inflow peak while the lag time had an inverse relationship with the peak of inflow hydrograph.

Introduction

Flood routing is the process of calculating downstream flows from upstream data in a channel or outflows from an inflow hydrograph in a reservoir. The former is known to be channel routing while the latter the reservoir routing. As discharge in a channel increases, stage also increases and with it the volume of water in temporary storage in the channel. During the falling portion of a flood an equal volume of water must be released from storage. As a result, a flood wave moving down a channel appears to have its time base lengthened and (if volume remains constant) its crest lowered. The flood wave is said to be attenuated. Wave movement in natural channels traditionally has been treated in design and prediction by applying hydrologic routing procedures. Such procedures solve the continuity equation (or storage equation) for an extended reach of the river, usually bounded by selected gaging points. The hydraulic characteristics of the channel and, in some cases, also dynamic effects are taken into account.

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Routing analysis estimates the effect of a channel reach on an inflow hydrograph. Techniques of channel routing are useful in instances where known hydrographic data are available at a point other than the point of interest. These techniques are applied to such problems as real-time flood forecasting, flood protection, hydraulic structure design, dam-breach analyses, modeling of watershed hydrology, peak flow estimation, and floodplain and flood insurance rates studies (USACE 1994, Henderson 1966). This is also useful in those instances where the channel profile or plan is changed in such a way as to alter the natural velocity or channel storage characteristics.

The principles of routing as applied to a reservoir involve computation of the effect of the reservoir on the shape of a flood wave. As a flood hydrograph approaches and passes through a reservoir or detention facility, the characteristics of unsteady flow become significant. Inflow and outflow rates and water storage characteristics are taken into account in routing a flood hydrograph through the storage facility.

Reservoir or detention pond storage routing also applies when outflow depends only upon the volume of flood storage. Techniques of reservoir routing are used in design of detention basins for flood control and in design of overflow spillways of conservation reservoirs.

Hydraulic storage occurs not only in channels and reservoirs but also as water flowing over the ground surface. Hence, storage is effective at the very inception of the flood wave, and routing techniques may be used to compute the hydrograph which will result from a specified pattern of rainfall excess.

Objectives of the study

The general objective of this study was to simulate routing behavior of a channel in the laboratory. The specific objectives were:

a. To apply the Muskingum Method to the observed data and estimate the values of x and K
b. To investigate whether the peak of the inflow hydrograph has any effect on-
    i. the parameters (K and x) of the Muskingum model.
    ii. attenuation and translation of the peak.

Review of Literature

This chapter outlines some methods of channel routing as available in literature.

Hydraulic Method

The classic hydraulic solution to the flood-routing problem was first proposed in 1848 by Sarre de Saint-Venant (Chow 1959). His approach solves both the continuity and momentum equations for a different volume of one-dimensional flow, where the forces on the control volume are limited to the effect of gravity, pressure variation, and friction or roughness of the channel walls. Mass is conserved in the solution and the effect of acceleration within the control volume and momentum flux across the upstream and downstream faces are considered (Henderson 1966).

The governing equations, known as the Saint Venant equations are given by

Continuity: $\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$..(2.1) !

Momentum"

$$S_f = S_o - \frac{dy}{dx} - \left(\frac{u}{g}\right)\frac{du}{dx} - \left(\frac{1}{g}\right)\frac{du}{dt} \qquad (2.2)$$

Where Q is the discharge, A the cross-sectional area, x the distance measured along direction of flow, t the time, S_f the friction slope, S_o the bed slope, y the depth of water above bed, and u the longitudinal velocity of flow.

Muskingum method

The Muskingum method is one of the best known and most widely adopted hydrologic methods for flood routing. It was introduced by McCarthy in 1938 for use by the Army Corps of Engineers in managing the Muskingum River basin in Ohio (Chow 1959, Henderson 1966, Roberson *et al.*, 1988). Its original formulation was strictly empirical, with two coefficients that operated jointly to control translation and attenuation. It was recognized that one of the coefficients, often termed as K, was related to the travel time or translation of the wave through the channel, whereas the second coefficient, represented by x, had the greatest impact on attenuation. The range of x coefficient was considered to be 0.0 to 0.5. Graphical techniques were devised to estimate it from calibration data (Roberson *et al.* 1988). Because the original Muskingum method was empirical, it was limited for use to cases where calibration data existed.

Kinematic routing

It was developed by French hydraulicians Kleitz and Graeff in the late nineteenth century, as reported by Montes (1998). The model was elaborated upon by Lighthill and Waltham (1955). It proceeds from joint consideration of the one-dimensional continuity equation and from the kinematic form, of the momentum equation. In theory, kinematic routing should not produce any attenuation in the peak flow, but the shape of the hydrograph will change because the kinematic celerity at higher flow depths is greater than at lower depths. This leads to a steepening of the rising limb of the wave profile. If allowed to continue long enough, the steepening becomes as sharp as to create a surge wave or moving hydraulic jump, known as "kinematic shock."

Kalinin-Milyukov method

This method was developed by Kalinin-Milyukov (1958) in the Soviet Union, which introduced the concept of characteristic reach length, upon which Pemmal's modem understanding of Muskingum-Cunge is based. The characteristic reach is a conceptual length of channel for which a one-to-one relationship can be established between the depth in the midpoint and the discharge at the downstream end, at least for small deviations from steady-state flow. The final form of the Kalinin-Milyukov equations is distinct from Muskingum-Cunge, utilizing various exponential relationships for coefficients. Authoritative presentation of the method is given by Miller and Cunge (1975) and Montes (1998).

Hydrologic Models - HECA

HEC-1 is a computer model that packages many common hydrologic processes together in one modeling environment (USACE, 1998). The original model was developed by the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers and was first released in 1968. Various refinements were issued 1970, 1973, 1981, 1990 and 1998. HEC-l includes procedures to input precipitation patterns and amounts, estimate net losses, convert precipitation to runoff, combine hydrographs from multiple tributaries and route flows using a variety of methods, including Muskingum-Cunge, modified Puls for river routing (hereinafter referred to as Cascading Reservoirs), and kinematic wave method. The program also contains models for flood damage. The HEC-I model describes a watershed as a series of independent subareas or processes, and within a subarea calculations are made based on average or lumped values of the parameters. In 2000, HEC released HEC-HMS (Hydrologic Modeling System), which is the successor to HEC-1. Most of the same computational methods were included in HEC-HMS, but with more advanced interface and programming (USACE, 2000).

USGS FEQ model

The FEQ (Full Equations) model was developed by the U.S. Geological Survey, in cooperation with the Illinois Department of Natural Resources. It was developed in part to handle the flat streams with broad floodplains that were characteristic of the Chicago area. It solves a modified form of the one-dimensional St. Venant equations for a sequence of open channels and can also handle empirical relationships for control structures such as bridges, culverts, dams, spillways, drop structures, etc. Channel characteristics are pre-processed into a series of tables, and an implicit numerical solution calculated (Franz and Melching, 1997).

Materials and Methods

Theoretical Considerations

As flood wave moves downstream through a river channel, both the peak flow rate and overall shape of the flood wave change. These changes can be measured by plotting hydrographs at different stations downstream. The essence of the flood routing problem is to predict the downstream hydrograph, using an input hydrograph upstream and information about the reach through which the wave travels. The two changes of primary interest are attenuation and translation of the floodwave. "*Attenuation*" is the relative decrease in the magnitude of peak discharge and "*Translation*" is the delay in time of peak discharge, based on travel time of the moving downstream. Fig. 3.1 depicts the inflow and outflow hydrographs for a typical flood routing problem in a river channel (USACE 1994, Bedient and Huber 1992).

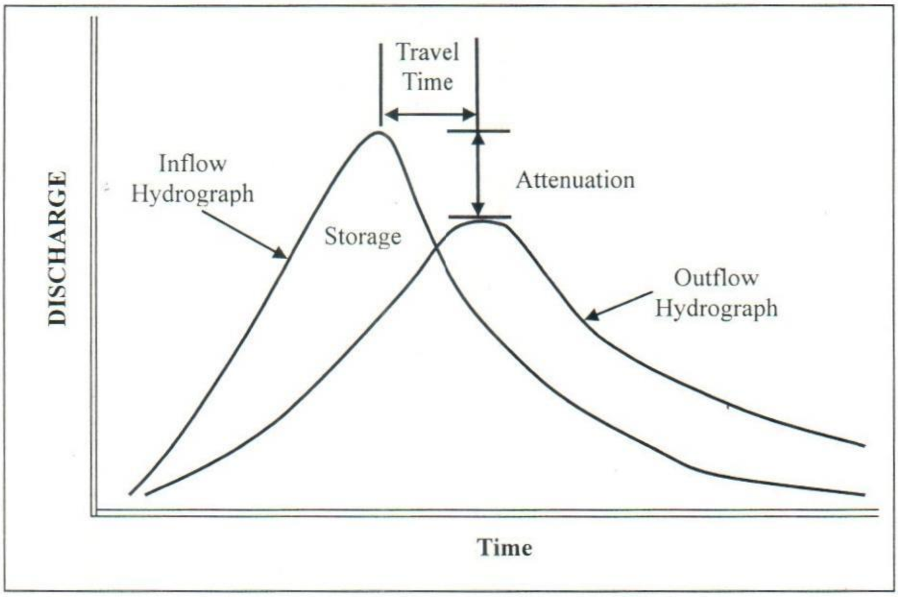


Fig. 3.1 Typical Hydrographs for Flood Routing Problem in River Channels.

Muskingum Method

The Muskingum channel routing method uses the continuity equation and a relationship between storage and inflow and outflow. The continuity equation in discrete form can be written as:

$$\frac{I_1 + I_2}{2}\Delta t - \frac{O_1 + O_2}{2}\Delta t = S_1 - S_2 \quad \text{.................................(3.1)}$$

Where, I_1 and I_2 are inflows (L ^{3}/T); O_1 and O_2 are outflows (L^3/T); #t is the time interval (T); and S_1 and S_2 are the values of reach storage (L^3) at time 1 and time 2 respectively.

In Muskingum method, storage in the channel reach is assumed to be linearly related to weighted sum of inflow and outflow and is generally expressed as:

$$S= K\,[x\,I + (1-x)\,O] \quad \text{...................... (3.2)}$$

Where, *K* is the storage time constant for the reach and x the weighting factor which varies between 0 and 0.5 for a given river section. Linsley *et al.* (1982) described a graphical procedure to determine *K* and *x* from available upstream and downstream hydrographs. The best value of x is that which causes the plot of *S* vs. *K [x I + (1-x) O]* most nearly as a single-valued curve. The Muskingum method assumes that this curve is a straight line with slope *K*.

Determination of K and x

Values of K and x for Muskingum routing are commonly estimated using K = travel time in the reach and the average value of x= 0.2. If both the inflow and outflow hydrograph records are available for one or more floods, the routing process is easily reversed to provide better values of K and x for the reach. To illustrate, instantaneous S versus [x I + (1- x) 0] are first graphed for several selected values of x. The accepted value of x is that which gives the best linear plot (the narrowest loop). After plotting, the value for K is determined as the slope of the narrowest loop.

Detailed Methodology

This study was under taken in the hydraulic laboratory of the Department of Irrigation and Water Management (IWM), in Bangladesh Agricultural University, Mymensingh for the period of 15 August 2010 to 20 April 2011.

Construction of Open Channel

An artificial trapezoidal channel of 1760 cm length was constructed in sand bed on a raised platform. The sand bed was 460 cm long, 122 cm wide and 15 cm deep placed on a horizontal metallic tray 75 cm above the floor. In order to increase the length of the channel it was constructed in a zig-zag pattern on the sand bed. The channel was dug by a metallic cutter of trapezoidal shape having bottom width of 2 cm and side slope of 0.5:1(H:V).The three cutting sides of the cutter, the bottom and the two sides, were sharpened from the inner side.

At first a channel was dug using the cutter in the sand bed with a slightly larger dimension than the intended finished size of the channel. Then cement-sand mortar was used to provide a lining on the channel bed and sides. The finished channel had a bottom width of 2 cm and side slope of 0.5:1. The bed slope of the channel was maintained at 0.0011 with an average depth of 8.5 cm. In order to prevent excessive lowering of flow depth at the downstream reach due to drawdown effect, a rectangular notch of 3 cm width was placed at the tail end to control the flow.

Preparation of Water Supply and Outflow Measuring Device

A cylindrical metallic drum was used to supply water to the channel. A pipe was fitted to an orifice situated just above the bottom of the drum to serve as an outlet. The drum was set on a circular stand to increase its height so that the outlet could supply water into the channel. Water was delivered to the drum by a pump from the underground reservoir of the laboratory. The outflow from the channel was measured by a vertical cylinder having an orifice same as water supply drum. A transparent plastic tube with a scale was set vertically on the outer surface of both inflow and outflow measuring device to observe water level inside the cylinder. A small centrifugal pump operated by a 0.5 HP motor was used to supply water from the underground reservoir to the drum. In order to provide a variable supply to the drum, a valve and a by-pass flow device were fitted to the delivery pipe of the pump. When the supply rate was reduced by the control valve, excess flow passed through the by-pass pipe.

Calibration of Water Supply Drum and Outflow Measuring Cylinder

The devices were calibrated to prepare rating curves. At first water was pumped into the drum at a low rate. When the water level in the drum stabilized, outflow from it was measured by volumetric method. The flow rate and corresponding water level were recorded. The pumping rate was then increased that raised the water level in the drum, as well as the outflow rate, to increase. When the water stage and the outflow became constant, the flow rate was measured and data recorded. In this way, the pumping rate was increased step by step to the highest capacity of the pump. At each step, the flow rate was measured. The stage and discharge data, so generated, were used to prepare a stage-discharge relation curve for the water supply drum. This curve was used to determine the drum outflow (inflow to the channel) from the stage reading during subsequent studies. In the same way, a stage-discharge relation curve was prepared for the cylinder used for the channel outflow measurement.

Simulation of Flood Hydrograph

It was intended to provide a varying supply into the channel so that it increased with time, attained a peak and then receded gradually as observed in a natural river during a flood flow. In order to do so, initially water was pumped into the drum at a low rate. After the outflow rate from the drum (inflow to the channel) and the channel flow became steady, the control valve of the delivery pipe was opened fully which increased the supply rate to its maximum level. This caused the stage in the drum to rise and the flow rate to increase. When the water stage rose a high level (arbitrarily chosen) the supply to the drum was gradually decreased by the flow control valve. This resulted in a gradual decline of the water stage in the tank (also the flow rate). During this entire period of variable flow, water stages in both the supply drum and outflow measuring cylinder were simultaneously recorded at an interval of 30 sec. These values were converted to inflow and outflow rates of the channel using the stage-discharge relation curves. This exercise was repeated thrice and in each time the Peak of the inflow hydrograph was changed. Thus, three sets of channel inflow-outflow hydrographs were generated.

Results and Discussions

The aim of this work was to observe the routing effect in the constructed mini channel in the laboratory. To serve the purpose, necessary data were collected for analysis. Table 4.1 and Table 4.2 show the calibration data of water supply drum and outflow measuring cylinder, respectively.

Table 4.1 Calibration Data of Water Supply Drum		
OBS. No.	Stage H (cm)	Average Discharge Q (cm^3/s)
1	8.6	18.34
2	9.4	70.68
3	11.6	145.46
4	14.5	210.24
5	18.0	280.43
6	22.6	396.53
7	26.4	436.63
8	27.8	460.82
9	35.3	526.50

Table 4.2 Calibration Data of Outflow Measuring Cylinder		
OBS. No.	Stage H (cm)	Average Discharge Q (cm^3/s)
1	5.6	12.21
2	6.5	77.96
3	8.4	156.31
4	10.7	224.90
5	15.3	324.91
6	22.7	448.28
7	24.0	466.00
8	25.6	489.96
9	28.5	529.41

Fig. 4.1 and 4.2 present the stage-discharge relationship of the water supply drum and outflow measuring cylinder, respectively. These show that the discharge increases nonlinearly with the increase of stage. However, the rate of increase of discharge increases with the increase of stage. These curves used to get the inflow and outflow rates for the recorded water stages.

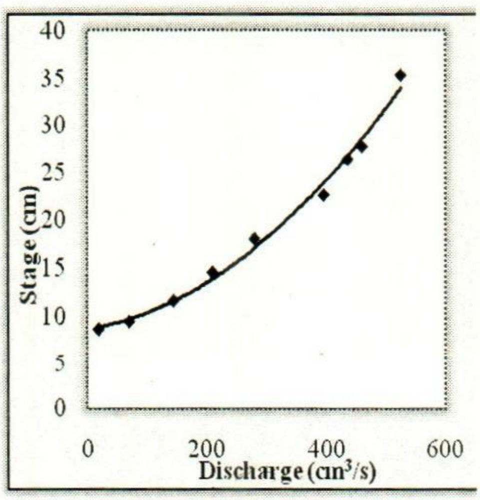


Fig. 4.1 Rating Curve of Water Supply Drum

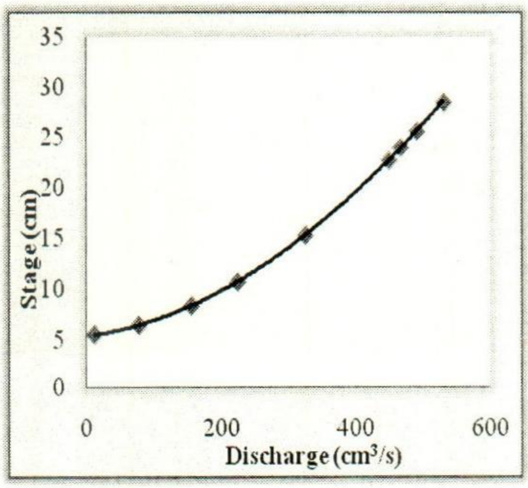


Fig. 4.2 Rating Curve of Outflow Measuring Cylinder

Table 4.3 presents the of inflow and outflow hydrograph data of the channel for three different trials. The table shows that, the highest inflow discharges are 512 cm^3/s, 470 cm^3/s and 428 cm^3/s for trials 1, 2 and 3, respectively.

Table 4.3 Hydrograph data of the channel

Time (sec)	Trial-1		Trial-2		Trial-3	
	Inflow (cm^3/s)	Outflow (cm^3/s)	Inflow (cm^3/s)	Outflow (cm^3/s)	Inflow (cm^3/s)	Outflow (cm^3/s)
00	20	10	60	50	25	13
30	110	10	210	50	150	13
60	222	20	322	70	257	20
90	330	50	380	120	315	65
120	385	125	405	265	345	130
150	415	255	434	330	370	212
180	435	340	450	370	390	295
210	455	390	462	400	410	337
240	468	420	470	415	420	370
270	480	445	470	430	428	392
300	488	460	468	444	427	400
330	495	475	460	455	417	408
360	505	480	457	460	395	410
390	512	485	438	460	365	407
420	510	496	430	460	317	402
450	504	497	415	455	285	392
480	490	492	400	450	236	365
510	472	485	375	440	195	324
540	450	475	340	425	160	280
570	437	460	310	400	130	196
600	415	455	262	385	95	160
630	390	445	210	350	75	105
660	360	430	150	325	50	90
690	320	420	125	305	35	75
720	282	405	90	250	30	50
750	240	376	65	175	24	40
780	210	348	50	105	22	30
810	170	332	30	85	20	25
840	145	300	25	70	-	-
870	115	265	25	60	-	-
900	100	232	20	50		
930	75	190	-	-	-	-
960	60	155				
990	50	110	-	-	-	-
1020	40	72				
1050	30	55	-	-	-	-
1080	22	40				

The inflow-outflow hydrographs for three trials are plotted using these data in Fig 4.3 - 4.5. The routing effect of the channel in modifying the flood wave is clearly evident in these figures, as expected. In each of the trials, the peak of the flood wave is attenuated in magnitude and translated in time.

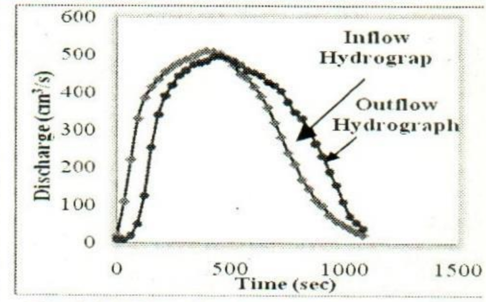


Fig. 4.3 Inflow-Outflow hydrographs for Trial-1

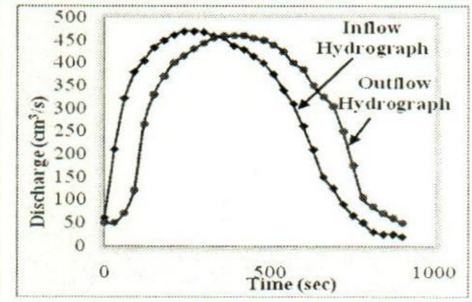


Fig. 4.4 Inflow-Outflow hydrographs for Trial-2

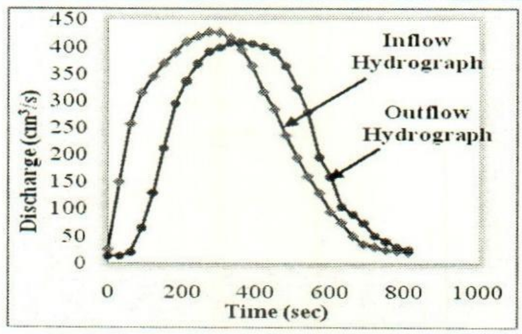
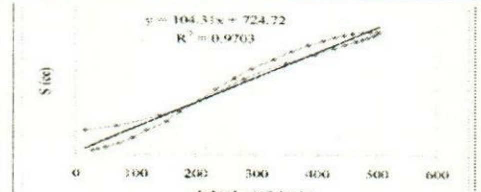


Fig. 4.5 Inflow-Outflow hydrographs for Trial-3

Fig. 4.6 $ 4.8 present the plots of S versus [x I + (1- x) O] values for the hydrograph data of three trials. For each trial, different values of x were assumed and the values of [x I + (1- x) O] were calculated. Then the values of S were plotted against the values of [x I + (1- x) O] and the best values of x was one which produced the narrowest loop. A straight line was fitted through the loop and the value of K was determined from the slope of the line.



!Fig. 4.6 S vs. [x I + (1- x) O] curve for Trial-1 (x= 0.6 and K=104 sec)

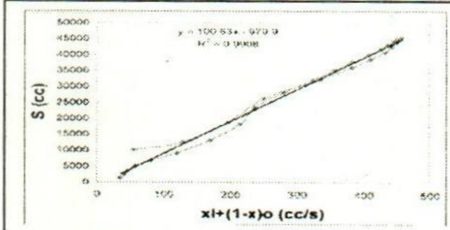


Fig. 4.7 S vs. [x I + (1- x) O] curve for Trial-2 (x= 0.5 and K=100 sec)

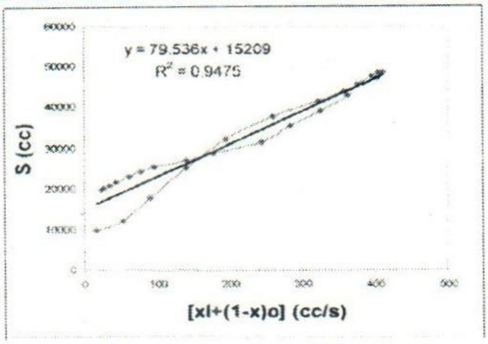


Fig. 4.7 S vs. [x I + (1- x) O] curve for Trial-3 (x= 0.3 and K=79 sec)

Table 4.4 presents the values of attenuation, translation, x, K and inflow peaks for the trials. It shows that attenuation has no definite relation with the inflow peaks, but translation increases with the decrease of inflow peak values. However, the values of K and x increase with the increase of inflow peaks.

Trial No.	Inflow Peak (cm^3/s)	Attenuation (cm^3/s)	Translation (sec)	K (sec)	x
1.	512	15	60	104	0.6
2.	470	10	90	100	0.5
3.	428	18	120	79	0.3

Conclusions and Recommendations

Conclusions

The study was conducted to simulate flood flows to study channel routing in the laboratory. Routing effects of the channel were clearly evident in the plots of inflow-outflow hydrographs. The peaks of the flood waves were attenuated and translated.

The study revealed that the amount of attenuation was not related to peak value of inflow hydrograph. However, it showed that the values of x and K of the Muskingum Model were positively related to inflow peak while the translation time had an inverse relationship with the peak.

Recommendations

This study was conducted in a single artificial channel of trapezoidal cross-section. Further studies may be conducted to study the effects of different geometric shapes and width-depth ratios of the channel section on routing parameters.

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ANALYTICAL EXPLANATION OF RIVERBANK FAILURE MECHANISMS

Aysha Akter[1]

Abstract

'Failures' both in protected and unprotected riverbank resulted in endemic and recurrent hazard to the stakeholders. In Bangladesh, all the three mighty rivers are experiencing erosion or failures due to composition of the braided stream and unstable bank materials. Using the river induced sand and local peoples' involvement, in recent years 'geobags' (sand filled geotextile bags) become long term protection means. Unfortunately geobag protections often fail to offer the expected protection and ends with the similar results experienced with unprotected riverbank. Due to easy handling geobag protection became a common practice among local and government initiations, so an in-depth knowledge on the failure mechanisms of these protection works are urgent. In this study, the failure mechanisms due to hydrodynamic forces on unprotected and protected typical Bangladesh river bank have been studied. The analytical approaches were reviewed for the unprotected riverbanks, and then an understanding was developed for possible forces acting on geobag protected riverbank. This analytical finding for the geobag protected riverbank is expected to provide useful information on remedial actions to ensure hydraulic stability. Thus this study will also contribute to the other discrete element materials namely riprap and cement concrete blocks.

Introduction

'Failures' in riverbank results disastrous socio-economic effect to the adjacent inhabitants and this is a common issue for all the three mighty rivers in Bangladesh. About 1 million people are directly affected each year and the total monetary loss is estimated to be approximately US$ 500 million a year (Rahman, 2000; Rahman and Manprasert, 2006) which is almost 7% of revenue income in the national budget for Bangladesh. The unprotected riverbank failure progression can be illustrated in three main combinations of bank materials, i.e. (a) In non cohesive bank, failures are due to secondary current and tension crack development. During most of the flood events weak banks of non-cohesive materials collapse easily due to secondary down welling and forms wide shallow channel (Figure 1) (Hoey, Smart *et al.,* 1998). Non-cohesive soils or materials in which relatively deep tension crack developed leading to the planar / slab-type failure with a large block slipping and / or rotating forward (Figure 2). Typically the slab type failure is observed in the Brahmaputra-Jamuna river bank, toe erosion or undercutting at depth leads to the failure in the upper bank above the water surface (Thorne, Russell *et al.,* 1993).(b) Relatively deep-seated namely planar or rotational failure occurs in cohesive banks and the associated tension crack observed some distance back from the bank top (Hoey, Smart *et al.,* 1998).(c) In composite bank, the lower bank is more frequently exposed to flowing water form a cantilever overhang by having upper bank undercut and

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mostly three types of failure occurs (i) Tension failure (Figure 3), (ii) Shear failure and (iii) Beam-type failure (Figure 4).

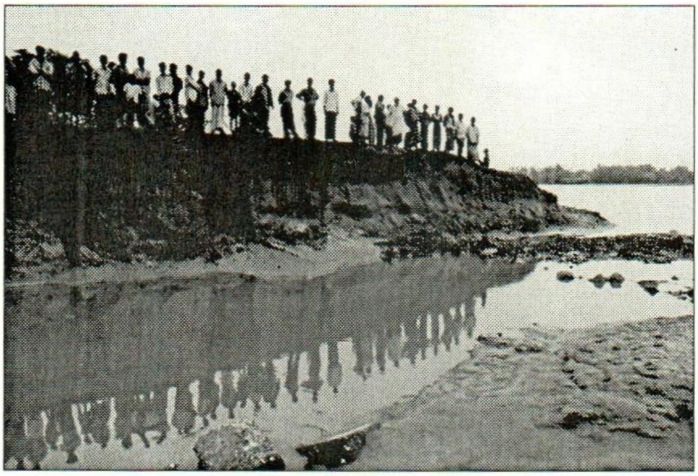
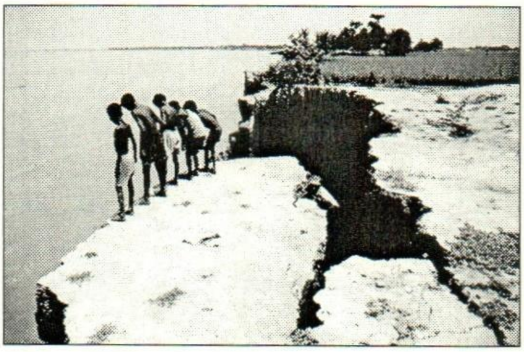
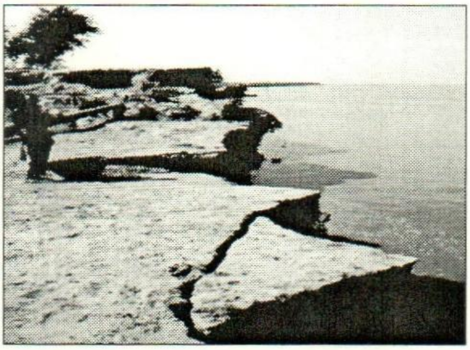


Figure 1: Village washed away, Lalmonirhat (DRIK, 2007)



(a) Dewanganj(DailyStar 2007)



(b)Sharia kandi upazila(DailyStar 2007)

Figure 2(a)(b)Planar/Slab failure observed in Jamuna river bank

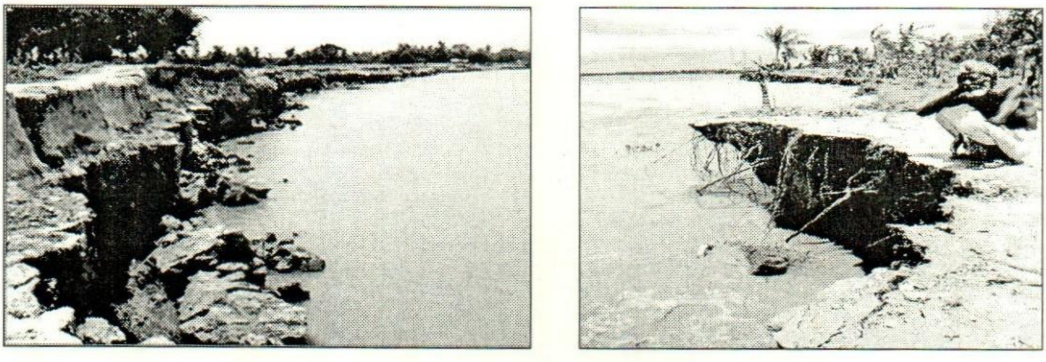
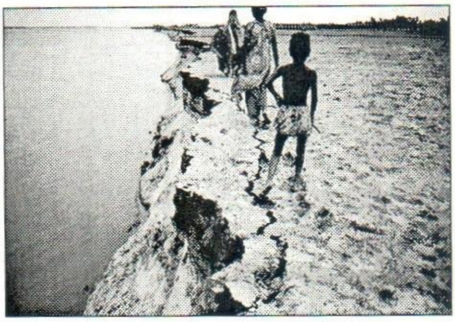


Figure 3:Devouring lands along the western bank of Brahmaputra (DailyStar 2007)



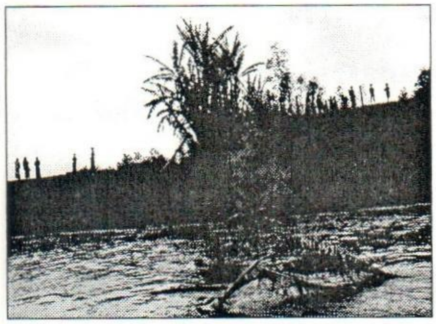


Figure 4: Edge of the island about to be eroded, Chilmari, Jamuna River in 2007

'Riverbank Protection' is treated as the efficient and potential affordable solution against the above mentioned problem of bank erosion considering the geographical position of Bangladesh. However, as the typical construction and subsequent maintenance along the major rivers in Bangladesh often fails due to the shifting nature of the erosive attack locations. Additionally, these rivers also immediately respond to any type of intervention. So, it is required to respond flexibly and to adapt the work until large-scale stability is observed at the area of intervention and this approach is named 'adaptive approach'(JMREM, 2006). Last decade, in 2002, an adaptive riverbank project has started for 7.0 km and 4.4 km of riverbank revetment in Pabna Irrigation and Rural Development Project (PIRDP) and Meghna-Dhonagoda Irrigation Project (MDIP) by using sand filled geotextile bag ('geobags') against the regular erosion of the Jamuna and the Meghna rivers respectively. Field survey during 2009 showed failures in some places in the protected bank (Figure 5).

As the adjacent inhabitants take initiatives to save their livelihood during emergency, the knowledge on failures could provide them well prepared. Thus, the main objective of this paper to review the processes of different types of natural riverbank failures and develop an idea how a geobag protected riverbank experienced failures.

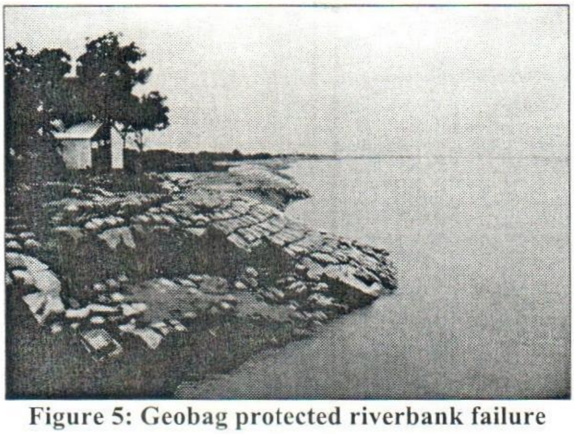


Figure 5: Geobag protected riverbank failure

Unprotected Riverbank Failure Process

According to Hooke (1979) two main processes of bank failures are- the direct corrosion, which is controlled by river flow condition, and the slumping, related to the soil moisture conditions. The rate of bank failure / erosion can be determined by the size of the bank failure block, equal to the width of the bank failure block divided by the total time. The total required times consists of the time required for basal erosion T_s and bank failure T_f (Duan, 2005). The period of bank failure is the time interval between two sequential bank failures and the frequency of bank failure is the reciprocal of the period of bank failure. Then the rate of bank erosion (Duan, 2005) is

$$M = \frac{dB}{dt} \approx \frac{\Delta B}{\Delta T} \text{...} (1)$$

Where,

ΔB = Width of failure block;

ΔT = $T_s + T_f$ = Total time requirement

Soil property and bank geometry play an important role on the stability of the bank with respect to mass failure (Osman and Thorne, 1988; Darby and Thorne, 1994). Most of the riverbank mass failure occurs whenever the shear stresses imposed on the bank exceed the maximum shear strength of the bank material. Soil shear strength expresses proportional to the cohesion c and the angle of friction ϕ (Taylor, 1948; Lamb and Whittman, 1969). So, the increased c and ϕ increases river bank stability. The driving force for bank failure is directly proportional to the specific weight γ , bank height H and slope angle α (Osman and Thorne, 1988). The riverbank stability decreases with their increasing value.

The term factor of safety for stability analysis can be defined as:

$$F_s = \frac{\text{Resisting force of riverbank (i.e. Soil shear strength)}}{\text{Driving force (i.e. Shear stress imposed on the soil)}} = \frac{s}{\tau} \text{.................} (2)$$

When, $F_s = 1$ failure is imminent, while increasing values of F_s i.e., $F_s < 1$ indicate progressively stability.

Non Cohesive Bank

Case I: Without Tension Crack

Plane slip analyses consider the stability of a single slab or wedge (Figure 6).

Resisting Force = $c.AC + W\cos\beta.\tan\phi$
(Hemphill and Bramley, 1989)

Driving Force = $W\sin\beta$

Where,

W = Weight of the failure block = $\frac{\gamma H^2}{2}\left(\cos\beta - \frac{\sin\beta}{\tan\alpha}\right)$

β = The angle that the failure plane makes with the horizontal

H = The slope height

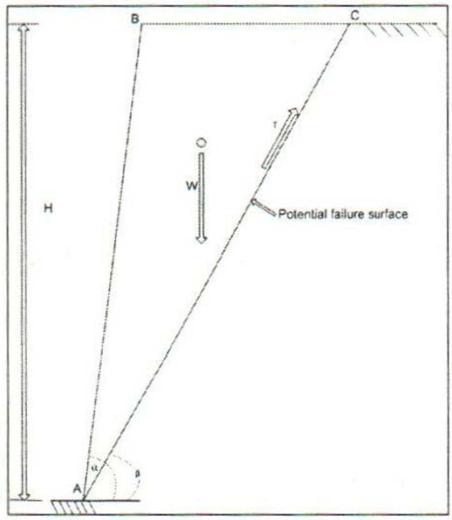


Figure 6: Planar slope failure (without tension cracks)

(Hemphill and Bramley, 1989)

Then,

$$F_s = \frac{2c.\sin\alpha}{\gamma H.\sin(\alpha-\beta)\sin\beta} + \frac{\tan\phi}{\tan\beta} \quad \text{.......................................(3)}$$

For cohesionless soil c is zero, then from Equation 3, $\alpha = \beta$ and

$$F_s = \frac{\tan\phi}{\tan\alpha} \quad \text{...(4)}$$

According to Hemphill and Bramley (Hemphill and Bramley 1989), for stable non cohesive bank the value of ϕ is approximately 30^o to 35^o. For a cohesive soil ϕ is zero and without any tension cracks, the factor of safety against sliding:

$$F_s = \frac{2c.\sin\alpha}{\gamma H.\sin(\alpha-\beta)\sin\beta} \quad \text{..(5)}$$

For cohesive bank material c in Equation 5 represents as undrained strength i.e. C_u

Case II: Tension Crack Consideration

This type of failure observes in Jamuna River normally 0.5 meter to 1.0 meter behind the upper verge (top of the riverbank) of the slope where the upper clay layer frequently shows deep fissures (JMREM, 2006). The self-weight of the block acts as a charge on the sliding sand mass.

Figure 7 shows the planner/ slab failure geometry after erosion of the river bank (noted in Figure 2).

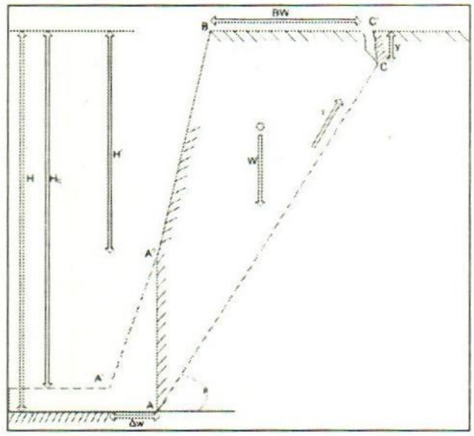


Figure 7: Riverbank after erosion to point of failure

(Osman and Thorne, 1988)

Where,

Δw = Change in riverbed width due to lateral

erosion at bank;

Δz = Degradation depth over time period ΔT;

H = Bank height above the riverbed;

H_o = Initial bank height above the bed;

H' = Bank height above the failure plane;

y = Depth of tension cracking.

Resisting Force $= c.AC + W\cos\beta.\tan\phi = \dfrac{c(H-y)}{\sin\beta} + W\cos\beta.\tan\phi$

Driving Force $= W\sin\beta$

Weight of the failure block $= \dfrac{\gamma}{2}\left(\dfrac{H^2 - y^2}{\tan\beta} - \dfrac{H'^2}{\tan\alpha}\right)$ (Osman and Thorne 1988)

According to Osman and Thorne (1988), the width of the failure block:

$$\Delta B = \left(\frac{H-y}{\tan\beta} - \frac{H'}{\tan\alpha}\right) \qquad (6)$$

Failure plane angle:

$$\beta = \frac{1}{2}\left[\tan^{-1}\left\{\left(\frac{H}{H'}\right)^2\left(1-\frac{y^2}{H^2}\right)\tan\alpha\right\}+\phi\right] \qquad (7)$$

When lateral erosion and bed degradation negligible $H = H'$, and due to absence of tension crack $y = 0$; Osman and Thorne (1988), Taylor (1948), Spangler and Handy (1973) defined the angle between failure plane and the horizontal as:

$$\beta = \frac{1}{2}(\alpha+\phi) \qquad (8)$$

And for a vertical bank (Osman and Thorne 1988),

$$\beta \cong \frac{1}{2}(90+\phi) \qquad (9)$$

For cohesive bank experiencing planar bank failure, Duan(2005) discussed the bank failure probability and the rate of erosion. The probability of bank failure is defined as equal to the probability when the discharge is greater than the critical discharge of bank failure.

Cohesive Bank

Osman and Thorne (1988) presented the Culman-type analysis for the stability of the steep cohesive riverbank (Figure 6). In their analysis bank geometry changes due to toe scour and lateral erosion are not accounted for the tension crack analysis. Darby and Thorne (1994) recommend for the accurate determination of tension crack location, to the calculation of the geometry of riverbank failure blocks. They suggested for the prediction of land loss and bank sediment yield associate with riverbank instability and channel widening. A rational, physically based method to predict the location of tension cracks on the floodplain behind the eroding bank face is also presented. The mass of the failure block (Darby and Thorne, 1994) is expressed as:

$$m = \rho\left\{\left[\frac{x^2\tan\alpha}{2}+x(H-H')\right]-\frac{x^2\tan\beta}{2}\right\} \qquad (10)$$

The tension crack depth y can be defined geometrically (Darby and Thorne 1994) as:

$$y = x(\tan\alpha-\tan\beta)+(H-H') \qquad (11)$$

Composite Bank

According to Thorne and Tovey (1981), a tensile failure across a horizontal plane occurs when the tensile stress due to the weight of the lower part of the block overcomes the tensile strength of the soil. Beam failure is the most common mechanism of cantilever collapse. Thorne and Tovey (1981) indicated a horizontal axis which followed by the rotating block before its failure. Shear failure is the type of failure normally found in the sandy soils of low cohesion and to areas where the weak bank vegetation observes. The failure occurs by the downward displacements of an overhanging block along a vertical plane through entrapping the fissure (Thorne and Tovey, 1981).

Thus the natural riverbank failures can be predicted based on the bank materials, failed block area or volume as well as the active hydrodynamic forces. But for protective riverbanks specifically while there are discrete materials on the bank, this prediction becomes very difficult. In following section there is an attempt to understand how these discrete elements perform while they are in a stack. This should be noted that the concepts of hydrodynamic forces on single geobag has studied earlier by Zhu *et al.*, (2004), unfortunately bags in stack would behave differently in their initiation movement and then this might follow the single geobag phenomena.

Geobag Protected riverbank Failure

The geobag at the river bank will begin to move towards the bed when the downstream and upward directed forces acting on bag overcomes the forces keeping it in contact with the bank. In Figure 8, the geobag (Q) is subjected to the forces due to the gravity (F_g), buoyancy (F_b), lift (F_L), drag (F_D) and a resisting force (F_R).

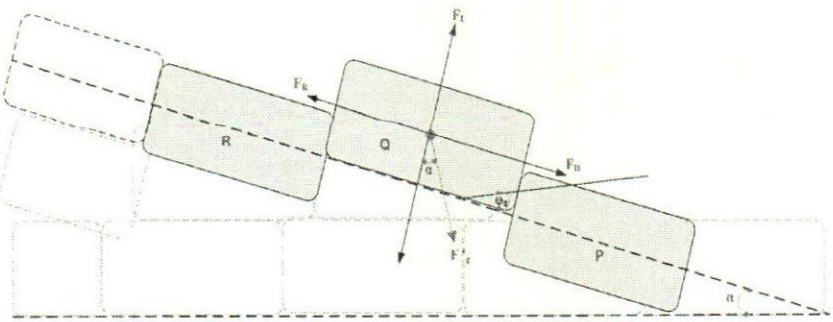


Figure 8: Force balance on geobag Q in stacked bags(modified)

At incipient condition, the geobags start to slide; the force balance equation can be represented in a modified form of the formula derived by Zhu *et al.,* (2004), as:

$$F_R = F_D + F_g' \sin\alpha \qquad (12)$$

The force resisting downstream motion is

$$F_R = f.F_N = f\left(F_g^{'}\cos\alpha - F_L\right)\text{.................................}(13)$$

Where,

f = Coefficient of friction or the coefficient of resistance

The gravitational force on the immersed weight of the sediment filled bag is:

$$F_g^{'} = F_g - F_B = \left(\rho_{bag} - \rho\right)g\left(b\times l\times t\right)\text{.............................}(14)$$

Here, equations(12), (13) and (14) are modified for the geobag in a stack.

As the lift and drag forces depend on the undisturbed flow velocity, Wiberg and Smith (1987) expressed the downstream velocity u in terms of the product of the shear velocity U_* and velocity profile function. The velocity profile comprises of height of the sand filled bag above the riverbed and the bottom roughness parameter. The bottom roughness is replaced by the effective roughness parameter similar to Marsh *et al.,* (2004).Thus can be written for Figure 8 as:

$$u = U_* f\left(\frac{h}{\Delta_{eff}}\right)\text{..}(15)$$

Where,

U_* = Shear velocity

= $\sqrt{\frac{\tau}{\rho}}$ (Shields, 1936)

h = Height above the bed

Δ_{eff} = Effective roughness parameter = $\frac{k_s}{X}$

k_s = D_{50} (meter)

X = Correction factor as given by Chang(1988)

$$f\left(\frac{h}{\Delta_{eff}}\right) \quad = \quad \text{Velocity profile function}$$

The drag force

$$F_D = \frac{1}{2}\rho C_D\left[u^2(h)\right](b \times t) = \frac{1}{2}\rho C_D U_*^2\left[f^2\left(\frac{h}{\Delta_{eff}}\right)\right](b \times t) \qquad (16)$$

Lift force according to Wiberg and Smith (1987),

$$F_L = \frac{1}{2}\rho C_L\left[u_T^2 - u_B^2\right](b \times l) = \frac{1}{2}\rho C_L U_*^2\left[f^2\left(\frac{h_T}{\Delta_{eff}}\right) - f^2\left(\frac{h_B}{\Delta_{eff}}\right)\right](b \times l) \qquad (17)$$

Where,

C_D	=	Drag Coefficient
C_L	=	Lift Coefficient
u_T and u_B	=	Velocity at the top and bottom of the bag respectively
h_T and h_B	=	Height above the bed of the top and bottom of the bag respectively

So, the modified equations (16) and (17) are considering undistributed flow velocity.

Assuming that at the stage of instability, drag and lift force dominates the resultant force. Tsinker (2004) formulated a stability ratio as the drag force plus the lift force divided by the gravity force:

$$\frac{F_D + F_L}{F_g} \approx \frac{\rho {D_{50}}^2 u^2}{g\left(\rho_{bag} - \rho\right){D_{50}}^3} = \frac{u^2}{g\Delta D_{50}} \qquad (18)$$

Where, D_{50}, the median size of geobag (length, l; width, b; and thickness, t) is calculated for the Jamuna - Meghna River Erosion Mitigation Project (NHC 2006) as:

$$D_{50} = (l \times b \times t)^{1/3} \qquad (19)$$

Thus equation (18) expected to provide a useful guidance on the active hydrodynamic forces for geobag riverbank once the field data are available.

Concluding Remarks

The failures in riverbank are controlled by river flow condition and the slumping whether the bank is protected or unprotected. In unprotected riverbank, soil properties as well as soil moisture conditions influence the rate of bank failure / erosion. For geobag protected riverbank the mechanism involved through geobag – riverbank interaction. The geobags at the riverbank will start to move towards the riverbed when the resulting forces fail to keep it in contact with the bank. Thus, the involvement of gravity, buoyancy, lift and drag forces were studied in this study and finally a relationship were developed among the bag velocity and bag median size during bag movement initiation.

For riverbank protection, discrete materials namely cement concrete cube / blocks and riprap also used in Bangladesh. This understanding from geobag movement initiation can also provide useful information on this. However, to involve this relation in practical riverbank protection design criteria, there is a need for intensive studies.

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SUSTAINABILITY OF AN ARTIFICIAL DREDGED CHANNEL ALONG THE BRAIDED JAMUNA RIVER OF BANGLADESH

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Abstract

River bank erosion is a severe problem in Bangladesh especially along the braided Jamuna River considering both the scale and intensity of erosion. BWDB has initiated a 20 km capital dredging on the River Jamuna as pilot basis in order to guide the flow to reduce the risk of failure of Sirajganj Hard Point and right guide bundh of the Bangabandhu Multipurpose Bridge. This study would be useful to assess the role of capital dredging as well as the morphological stability in comprehensive river management plan along the braided Jamuna river of Bangladesh. Geo-referenced satellite image of ASTER and IRS LISS from 2007 to 2012 has been superimposed to assess the sand bar movement. During 2007 to 2012, lateral movement of Sandbar-1 ranges from 800m to 1870m whereas for Sanbar-2 ranges from 560m to 1060m. At the same time, the longitudinal movement of Sandbar-1 ranges from 150m to 600m whereas for Sandbar-2 ranges from 300m to 1400m. It is observed that, the channel is developing very fast along the western bank and flows were diverted towards the river bank. Image analysis indicates that the rate of bar translation is high and it may cause the siltation of the dredged channel. From cross-section comparison at three major locations indicates that the channel near Sirajganj Hardpoint shifts towards East. The average sediment concentration within the dredge area varies between 300~500 mg/l and the siltation rate along the dredged channe lis 60% to 80%. It is also observed that the siltation rate is higher, where the dredging alignment passing through the existing sandbar/char. The present study reveals that the artificial dredged channel of a sand bed braided Jamuna River would not be stable unless and until changing the upstream river morphology as well as hydraulic condition along the dredging alignment.

Introduction

The erosion and shifting of river courses, loss of land, especially along the Jamuna river have long been recognized as a natural problem that affects a sizable population (Elahi, 1991; Chowdhury and Kabir, 1991) in Bangladesh. The secondary current in a bend of a third order channel of braided river is similar to that of the single thread meandering channel (Uddin *et al.,* 2010). The influence of secondary currents on flow and sediment dynamics causes meander shifting through bank erosion and bar formation in typical meandering river [Farzana Mahmud *et al.,* 2011; Thorne, 1991; Leopold and Wolman, 1960; Leopold *et al.,* 1964; Dury, 1965; and Nakagawa and Scott, 1984]. The bank material characteristic along the Jamuna River is fine sand and almost uniform with respect to flow resistance (FAP1, 1993). The overall width of the river exhibits an increasing trend and there is tendency of shifting westwards, especially at the upstream part of the Jamuna river [FAP1, 1993; FAP24, 1996; China-Bangladesh Joint Expert Team, 1991]. To mitigate the

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above problem, historically Bangladesh Water Development Board (BWDB) is being implemented a number of river bank protection structures such as embankments, groins, revetments, spurs, hard points etc (BUET, 2008).

Since last sixties (FAP 1, 1993) BWDB have mixed experiences of failures and successes and each year a big amount of money is required for the maintenance these river training structures. As an alternative BWDB has initiated capital dredging on the River Jamuna as a pilot basis in order to guide the flow away from the west channel into a mid channel to reduce the risk of failure of Sirajganj Hard Point and to guide the flow along the middle of the existing char through the Bangabandhu Bridge to near Dhaleswari Offtake (BWDB-IWM, 2011). Total length of the pilot dredging is about 20 km from upstream of Sirajgonj hard point to downstream of Banghabandhu Bridge is shown in Figure 1. Before adopting these measures a thorough understanding of flow characteristics and their interaction with channel geometry and planform is essential (Bathurst, 1997). This study is intended to assess the sustainability of the above 20 km dredged channel along the braided Jamuna River.

Figure 1: Data collection location map around the study area.

Methodology

The sustainability of the dredged channel would be assessed by analyzing both the primary and secondary data. The data collection locations map around the study area is shown in Figure 1. Following data were used for these studies:

(i) Water level, discharge and sediment data;
(ii) Bathymetric data, velocity profiles and water surface slopes;
(iii) Grain size distribution of the suspended sediment;
(iv) Satellite images, which covering the flood plain and showing important features like point bars, alternate bars, middle bars, palaeo channels etc.;

Primary Data Collection

This data is collected for pilot capital dredging project and used as a primary data in this study which is formally permitted from BWDB and IWM. Primary data collection included the following items: A) Discharge data B) Suspended sediment and C) Water Level.

Secondary Data Collection

This study is based on secondary data. Secondary data were collected from IWM, BWDB and CEGIS. The bathymetric survey data for this study were collected from IWM. Historical hydrometric data such as water level, discharge, sediment concentration, bed material, etc. were also collected from IWM and BWDB. Satellite images were used in this study, which images were collected from Center for Environmental and Geographic Information Services (CEGIS). All images acquisition dates were in dry seasons.

Data Analysis

The sustainability of the dredged channel were assessed by analyzing the data so far available on bathymetric survey, sediment concentration, time-series satellite images, sediment concentration and discharges data near two important river training structures (Sirajganj Hardpoint and Right Guide Bundh of the Bangabandhu Bridge) along the Jamuna River.

Image analysis

GIS techniques were used to analyze the satellite images. Time series satellites images were used in studying the flow and erosion processes of a developing bend. The time-series geo-referenced satellite images were superimposed to assess the historical trend of the shifting of river courses. Geo-referenced images were used to delineate the large-scale bed-forms such as sandbars. The delineated sand bars were superimposed in GIS environment to assess the shifting pattern and to assess the translation process of large scale bed forms.

Cross-section Comparison

The bathymetry data were plotted from 2010 to 2013 at three different locations within the study reach. This data were plotted with and without dredging conditions to determine the dynamics of the river, trend of shifting the river courses, changes in planform and sedimentation process. In this analysis sediment concentration and discharge also link with the cross-section data because the rate of river bank erosion and changing of river planform is also related to the rate of sediment concentration and discharge.

Interpretation of Results

River bank erosion is a severe problem in Bangladesh especially along the braided Jamuna River considering both the scale and intensity of erosion. Thousand hectares of floodplain are eroded each year. Due to the dynamic nature of river morphology, it causes the sufferings to the people along with damages to public and natural resources. To prevent the erosion, different types of protection structures have already been constructed in different locations in both banks of the Jamuna river. The structural measures are sometimes ineffective due to morphological changes of the river.

a. Satellite Image Analysis

ASTER (2007) and IRS LISS (2008, 2009, 2010, 2011 and 2012) images were used to identify the sandbars and their translation process.

Figure 2: The sandbar movements of Jamuna River, year 2007 to 2010. [Source: CEGIS]

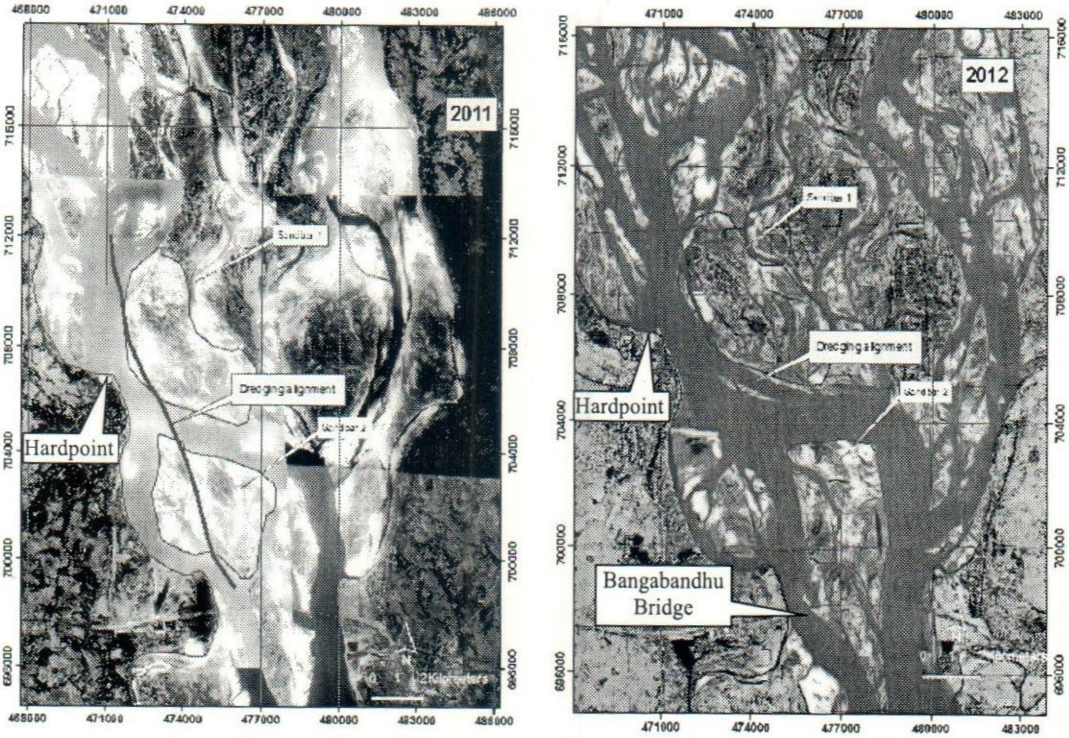

Figure 3: The sandbar movements of Jamuna River, year 2011 to 2012. [Source: CEGIS]

These sandbars are different size and shape and bars are translating both laterally and longitudinally at different rate. The sand bars at the upstream and adjacent to the study area are referred as sandbar-1 and sandbar-2 are shown in Figure 2 and Figure 3. In this analysis, lateral translation in each year was measured from the centreline of the 2007 bar towards the western direction. Eventually, longitudinal translation was measured from the head end of the sand bar of 2007 towards the downward direction.

Lateral Translation

The maximum lateral translation of sandbar-1 was occurred during 2007 and 2008 which was just upstream of the upstream termination of Sirajganj Hardpoint. In 2009 and 2012 bar was translated downstream from the upstream termination which caused damages of the hardpoint downstream from the termination. The west ward translation of the sandbar-1 diverted the flow towards the Hardpoint and caused undermining and damages during different years. Due to the position of sandbar 2 in 2007 a very narrow channel (100m to 150m) was flowing through between Sirajganj Hardpoint to Jamuna bridge guide bundh bend. Due to lateral translation, flow was diverted by the sandbar-2 extreme west point towards the bank and by eroding the bank, the bend consequently developed are shown in

Figure 2 and Figure 3. The lateral translation of sandbar-1 and sandbar-2 during 2007-2012 is shown in Table 1 and Figure 5.

Longitudinal Translation

Sandbars are also moving towards longitudinal direction. Longitudinal movement is measured as a distance travelled by the upstream end of the bar towards the downstream direction in the consecutive years. The longitudinal translation of sandbar-1 and sandbar-2 during 2007-2012 is shown in Table 1 and Figure 6. During 2007 to 2012, sandbar-1 translated 150 to 600 meter per year and sandbar-2 moved 300 to 1400 meter per year. The rate of movement of sandbar-2 is higher than that of sandbar-1 because sandbar-1 was obstructed by the revetment structure as well sandbar-2 could move downstream without facing any obstacle. In 2010 and 2012, sandbar-2 diverted flow towards downstream bend and caused huge erosion at the downstream part of the bend. As a whole, due to the translation of sandbars along downstream, flows were diverted towards the bank and caused bank erosion.

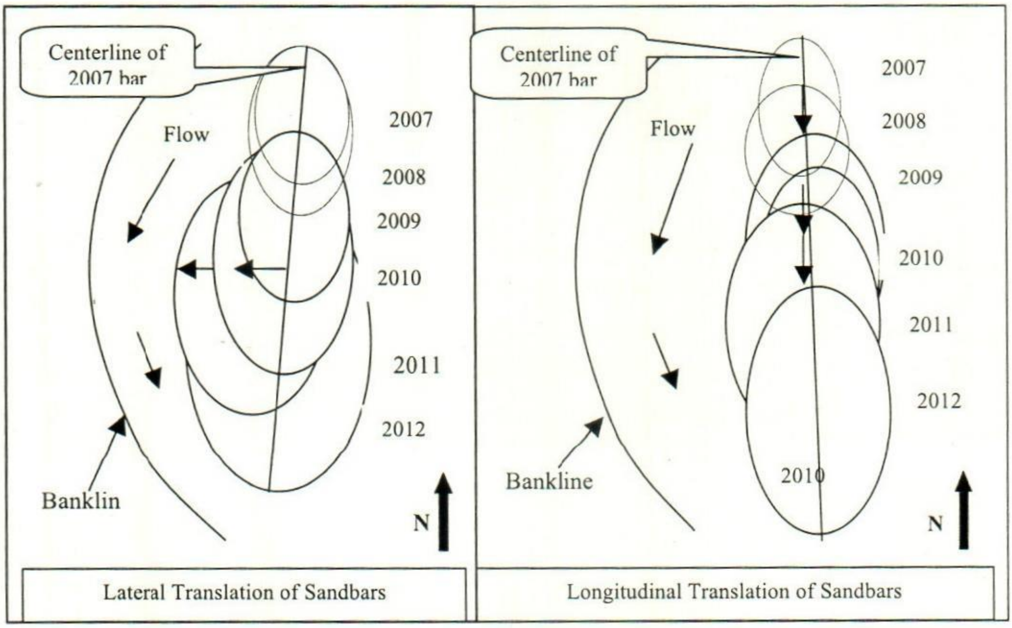

Figure 4: Schematic diagrams of lateral and longitudinal translation of sandbars. (Source: Fazana Mahmud, M.Sc thesis, 2011)

Table 1: Lateral and longitudinal translation of the sandbar-1 and sandbar-2

Year	Lateral translation		Longitudinal translation	
	Sandbar-1in (km)	Sandbar-2 in (km)	Sandbar-1in (km)	Sandbar-2 in (km)
2007	1540	730	0	0
2008	1690	910	650	1400
2009	1870	1060	800	1800
2010	1360	560	1300	2200
2011	1100	720	1950	2500
2012	800	680	2100	3500

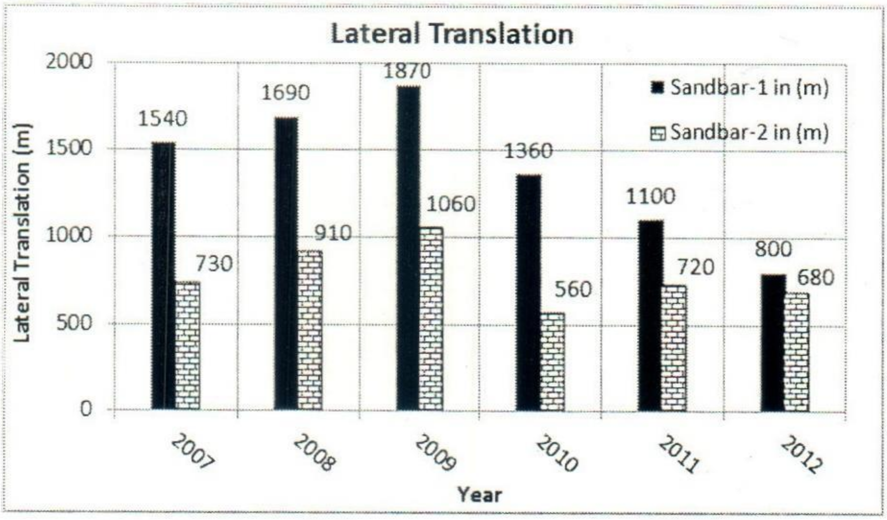

Figure 5: Lateral translation of the sandbar-1 and sandbar-2

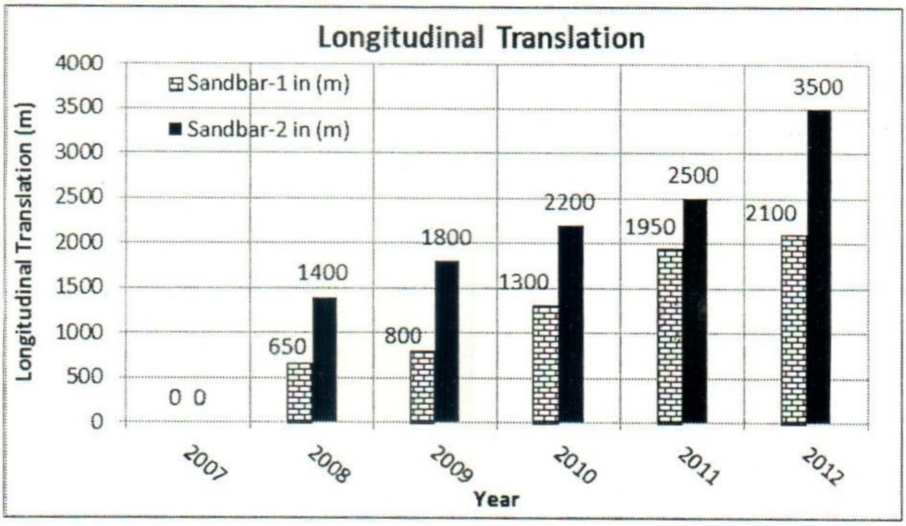

Figure 6: Longitudinal translation of the sandbar-1 and sandbar-2

Cross-section Comparison

Cross-section data was plotted just upstream of Sirajganj Hardpoint to downstream of Bangabandhu Bridge from 2010-2013, which are shown in Figure 7 to Figure 11. Dredging was implemented on 2011-12 and 2012-13 following year. From cross-section comparison, insignificant impact of dredging has been found at upstream of Sirajganj Hardpoint. After one year flooding the scenarios was changes due to sedimentation and erosion around the dredging location. Initially the flow would be diverted to the dredged channel about 12.50% only (source: IWM, 2012). As a result, the scour depth at Sirajgonj Hardpoint would be reduced upto maximum 5 meter. After one year of dredging it appears that the dredged channel were silted up 60%-80% in sense of depth but dredge channel widen due to erosion of both bank. However, since dredging would help to reduce scour depth around Bangabandhu Bridge guide bundh and it will provide safety to the guide bundh for a very short period.

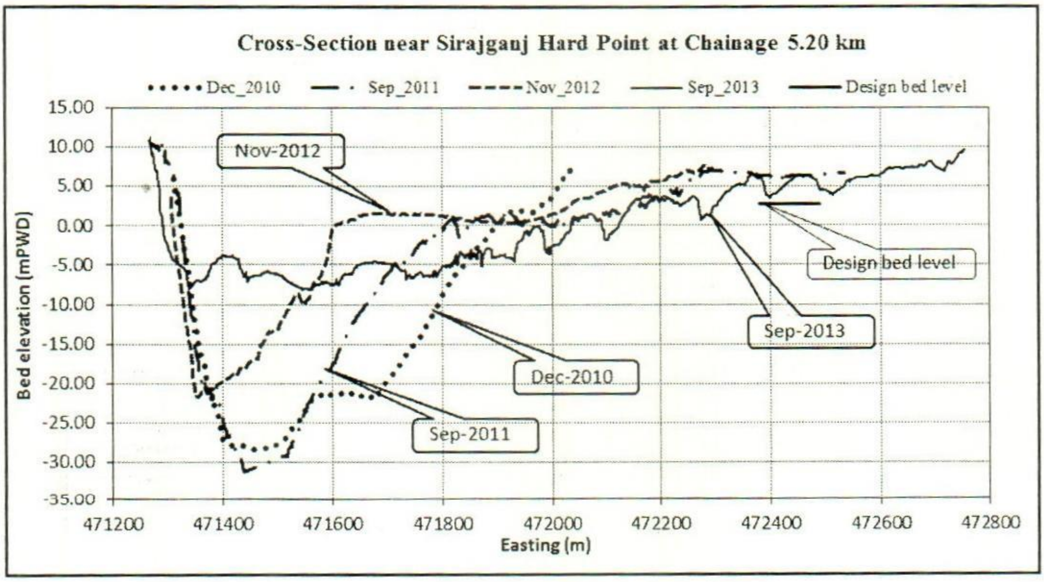

Figure 7: Cross-sections of Jamuna River near Sirajganj Hardpoint.

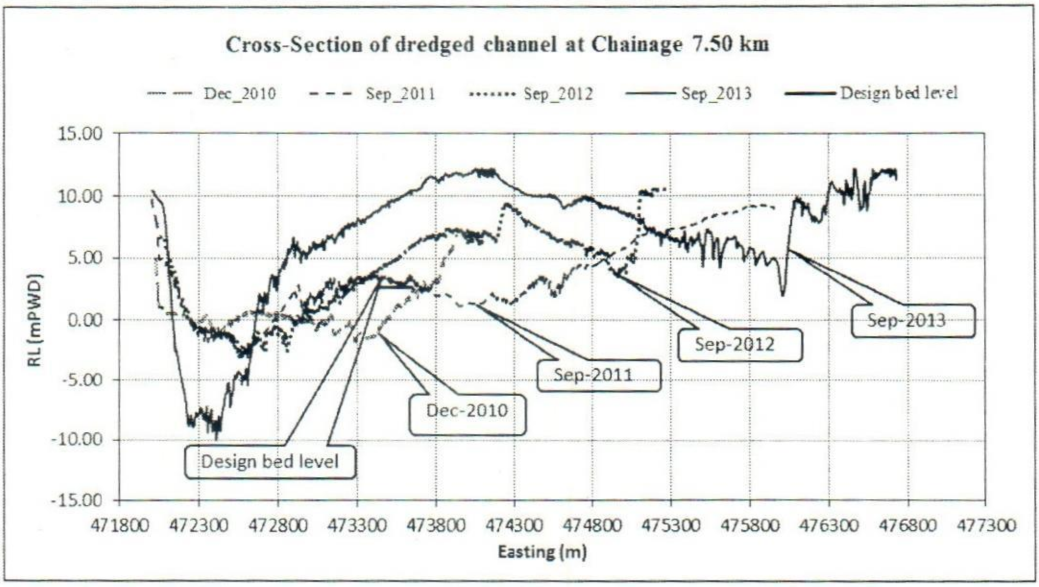

Figure 8: Cross-sections of Jamuna River at the dredged channel

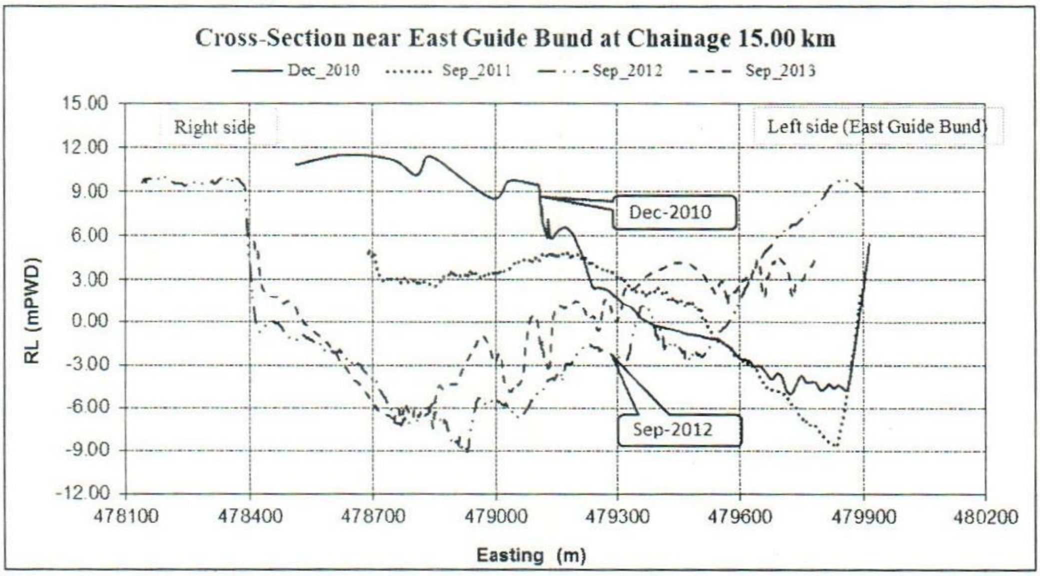

Figure 9: Cross-sections of Jamuna River near East guide bundh

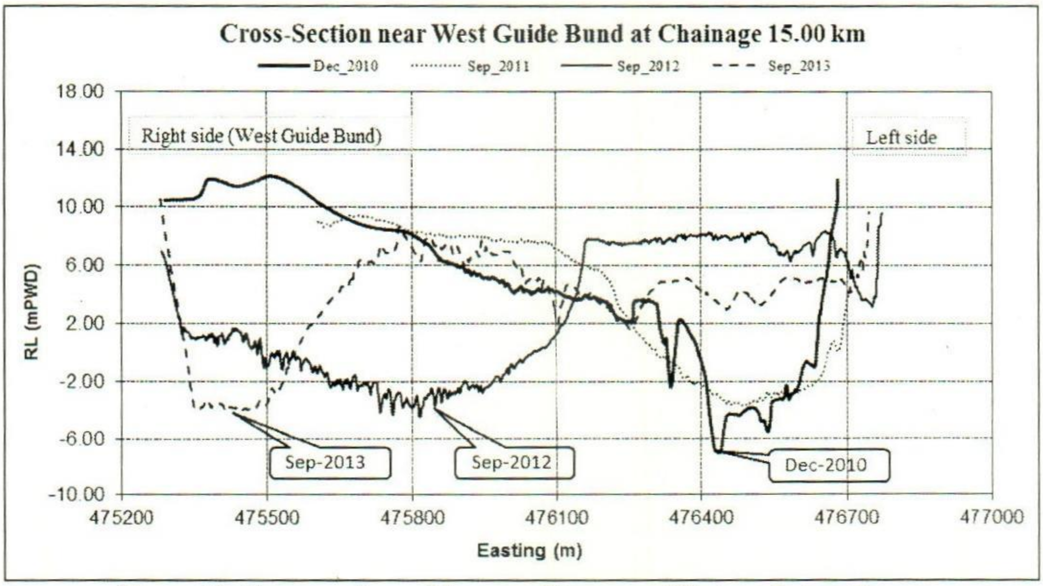

Figure 10: Cross-sections of Jamuna River near West guide bundh

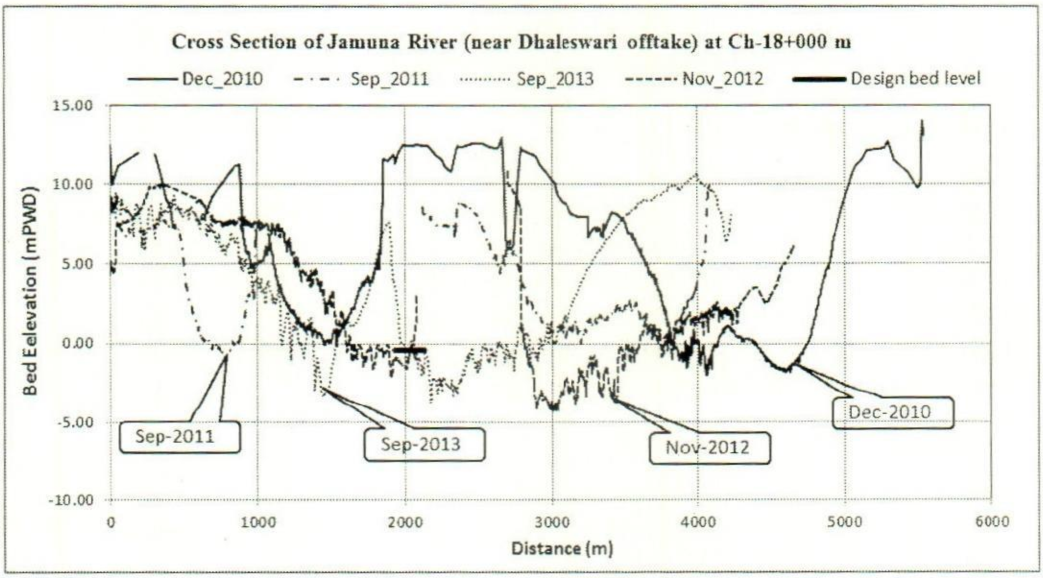

Figure 11: Cross-sections of Jamuna River near Dhaleswari Offtake

Conclusions

River channels tend to a dynamic equilibrium driven by the dynamics of discharge of water and sediment transport. Different river training works, upstream interventions increasingly change the natural drivers of the channel morphology. The major findings of this study are:

- Due to translation of sandbars along downstream, flows were diverted towards the bank and caused bank erosion. The channel is developing very fast along the western bank of the Jamuna River and eroding the river bank.
- The flow process in a bend of second order channel of a braided river is similar to the meander bends of a single thread meandering river but in this study, it is found that the characteristic of a bend channel within a braided river exhibits difference with a classical single thread meandering channel.
- Satellite image analysis indicates that the flow processes and the location of active bank erosion significantly changes due to changes in large scale sand bar movement (both laterally and longitudinally).
- Cross-section comparison indicates that the channel near Sirajganj Hardpoint shifts towards East and existing channel near Sirajganj Hardpoint is silted up just after dredging and after 1 year of flooding/monsoon the impact of dredging is very minor.
- It is also observed that the rate of siltation is higher, where the dredging alignment passing through the sandbar/char. It was happening during dredging, because we can not change the hydraulic condition around the dredged area.
- The major finding of this study is that the dredging of a braided river would not be a permanent solution wihout changing upstream river morphology as well as hydraulic conditions around the dredged alignment.

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SEDIMENT BUDGET OF MEGHNA ESTUARY

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Abstract

Bangladesh is the largest delta of the world formed mainly by alluvial deposit of the Ganges, the Brahmaputra and the Meghna river systems. These three mighty rivers carry annualy more than a billion tons of total sediment. The sediment load is then conveyed by Lower Meghna River and discharged into the Bay of Bengal. The river borne sediment inflow from the three major rivers the Ganges, the Brahmaputra and the Meghna influences the morphological development of the Meghna Estuary. Analysis of time series bathymetric chart collected under different project over the years has been done to analyze the spatial distribution of sediment deposition and erosion. The Natural Neighbour method is used for interpolation of surveyed data to develop sea bed topography using Mike 21 FM. The entire area has been divided into seven sub areas and erosion/ deposition volumes have been calculated based on interpolated depth values of the maps using 50x50m grid. The analysis of bathymetric data shows that accretion process exceeds the erosion process in the Meghna Estuary. Erosion process is dominant in the upper region of the estuary part i.e. from Chandpur to north of Bhola island, north coast of Hatiya island and northeast coast of Bhola and along the Boyer char (Ramgati). The highest rate of deposition is seen in the Sandwip, Jahajerchar and Urirchar area. The net deposition of sediment over the last ten years (1999 to 20098) in the entire estuary is about 2,760 Mm^3 and annual average is about 276 Mm^3.

Introduction

The alluvial deposits of the Ganges, Brahmaputra and Meghna rivers formed Bangladesh, the largest delta of the world. These three mighty rivers drain an area of about 1.72 million sq km, of which only about 7% lies within Bangladesh, and finally empties to the Bay of Bengal through Lower Meghna Estuary. The Ganges River drains a catchment of 1,114,000 km2 and the Brahmaputra River a catchment of 935,000 km2, supplying in combination around a billion tonnes of sediment each year to the Bengal Basin. Delta plains exceed 115,000 km2 in surface area (Woodroffe *et al.*, 2006), comprising the Meghna Delta plain (built by the two rivers downstream of their confluence) to the east, and an abandoned delta plain, the Gangetic Tidal plain to the west. Annually about 1250 billion cubic meters (BCM) of trans-boundary flow drains through Bangladesh to the estuary carrying about 1.1 billion tons of suspended sediment (O'Conor, and Nicholson, 1989). The coast along the Meghna River estuary is very dynamic with rapid erosion and accretion. The Ganges-Brahmaputra system carries about 1 billion tonnes of sediment each year (Allison, 1998a, Islam *et al.,* 1999). About 30% of the modern sediment is distributed in the delta plain (Goodbred and Kuehl, 1998), 21% along the topset of the subaqueous delta (Allison, 1998b), 20% along the foreset of the subaqueous delta (Michels *et al.,* 1998) and the rest 29% is carried to the Swatch of No Ground (Goodbred and Kuehl, 1999).

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Land accretion and erosion in the Meghna Estuary is a continuous and gradual natural process. The net average annual natural accretion is quite significant in the estuary ranging 1200 ha/y to 1900 ha/y (MES, 2001). The sediment load is then conveyed by Lower Meghna River and discharged into the Bay of Bengal. The main sources of sediment for land accretion are sediment carried by upland rivers, erosion from char and the shelf. The river borne sediment inflow from the three major rivers Ganges, Brahmaputra and Meghna influences the morphological development of the Meghna Estuary. Factors that play major role on accretion and erosion are the sediment load, its transport and distribution, the discharge of water, tidal forces and estuarine circulation (IWM, 2010).

The knowledge on sediment deposition volume and its distribution in the Meghna estuary is limited. It is important to know how much sediment is retained in the Meghna estuary and distribution of sediment deposition and erosion for understanding the morphological developments and planning of land reclamation and erosion control (Kamal *et al.*, 2012). This objective of this paper is to assess the erosion-deposition pattern in the Meghna estuary.

Approach & Methodology

The study mainly focuses on the processes of river and estuarine hydraulics and the erosion-deposition pattern of the Meghna Estuary. The modelling study has been devised in combination with data analysis and numerical modelling. The two dimensional (2-D) flexible mesh model (MIKE21 FM) has been applied for hydro-morphological investigation around the study area. Data on recent bathymetry, sediment concentration and hydrometrics of the estuary have been utilized for updating and re-calibrating the existing Bay of Bengal model.

Data Collection

The Bay of Bengal model is updated with the primary data surveyed in the year 2009 and the secondary data collected from different agencies. A survey team comprising of SSSU of EDP, SSD and SUA of BWDB surveyed the bathymetry of the Meghna Estuary, water level, discharge and sediment concentration data in 2009-2010.

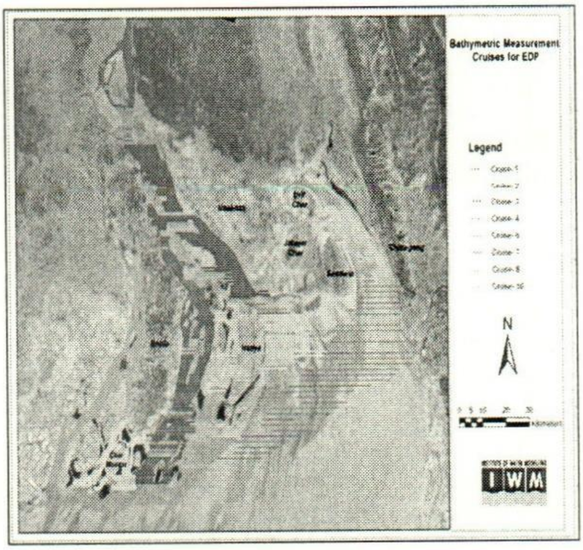

Figure 1: Bathymetric data collected under different cruises of EDP in 2009-2010

The transact lines of bathymetry survey of BWDB and IWM survey teams are shown in Figure 1 and 2, respectively.

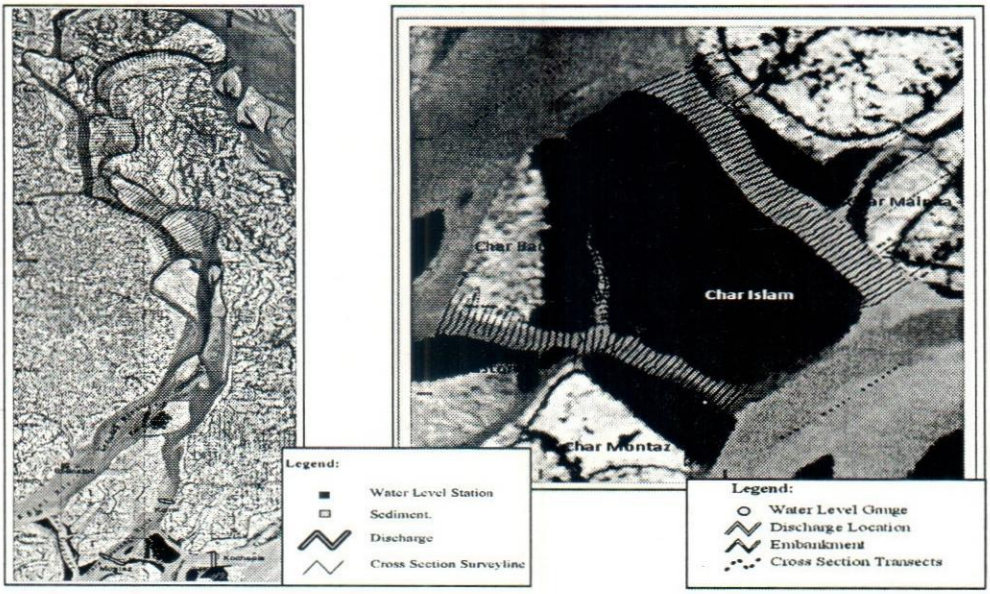

Figure 2: River/ channel cross section survey in the Tentulia River and Char Montaz Area

Data surveyed under previous studies have been collected and compiled. Table 1 presents the list of data collected under past studies.

Table 1: List of data collected from secondary sources

Data	Source	Name of Study	Period
Bathymetry	BWDB	Land Reclamation Project	1983
do	BWDB	Meghna Estuary Study II	2000
do	BWDB	Feasibility Study for Haitya-Nijhum Dwip Cross Dam Project	2006
do	CDSP III	Survey and Modelling Study of Sandwip-Urirchar-Noakhali Cross-Dam(s)	2008
Water Level	CDSP III	Survey and Modelling Study of Sandwip-Urirchar-Noakhali Cross-Dam(s)	2008
do	BIWTA	_	2001-2009
Discharge	CDSP III	Survey and Modelling Study of Sandwip-Urirchar-Noakhali Cross-Dam(s)	2008
Wind	BMD	_	1948-2008

Data Analysis

The bathymetry data over the entire Meghna Estuary was surveyed and the data has been analysed. The data quality has been checked comparing upstream and downstream data in same location. BWDB checked the consistency of Bench Mark values that were used in bathymetric survey and corrected the bathymetry accordingly. The bathymetric map thus generated from surveyed data is presented in Figure 3.

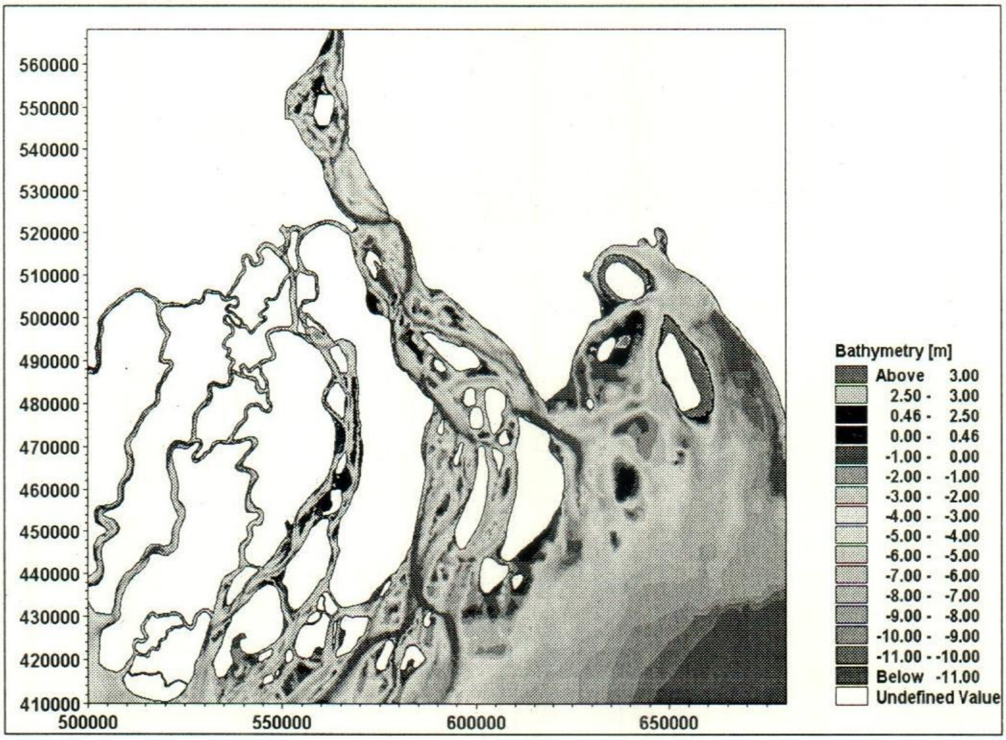

Figure 3: Updated bathymetry based on recent survey data

Model Development

Hydrodynamic Model

In this study the existing Bay of Bengal model has been updated with the surveyed hydro-morphological data and latest satellite imagery using MIKE 21FM modelling system. The model domain extends from Chandpur in the Lower Meghna river to in the north, to about 16^0 Latitude in the Bay of Bengal in the south. The coverage of the numerical model is shown in Figure 4.

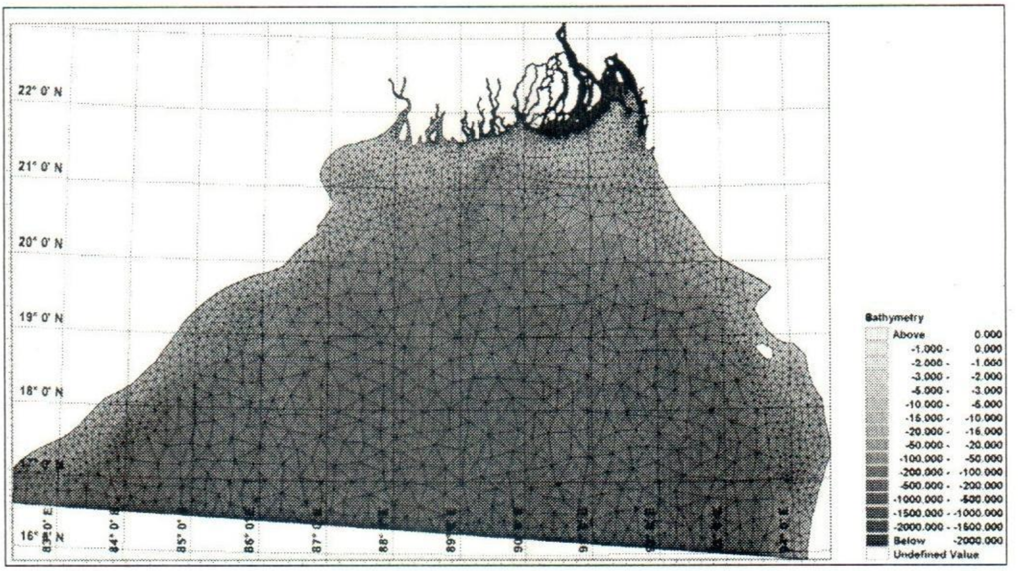

Figure 4: Bathymetry of the Bay of Bengal (with triangular flexible mesh) where color gradient shows the sea bed level in mPWD datum

There are two open boundaries in the model, one is in the Lower Meghna River at Chandpur (northern boundary) and the one is in the Bay of Bengal (southern boundary). Predicted tide has been used in the southern boundary and observed water level at Chandpur BIWTA Station has been used in the northern boundary.

The riverbank alignment (land boundary) in the northern part of the model has been updated on the basis of the mosaic of IRS P6 LISS III satellite image 2008 provided by EDP. The model applies PWD datum. The southern part of the Bay of Bengal is quite deep and the maximum depth along the southern open boundary is more than 3,000 meter. The computational grid or mesh size decreases (or the resolution increases) towards coastlines and Islands. Inter-tidal areas are flooded and dried during a tidal cycle, both in nature and in the model.

Sediment Transport Model

The cohesive sediment transport module is coupled to the hydrodynamic module and they run in parallel. The governing equation for sediment transport is solved in the same mesh and applies information of water levels and currents from the hydrodynamic module to calculate the sediment transport. One layer describes the sea bed in the sediment transport model. At the upstream boundary in the Lower Meghna River, suspended sediment concentration time-series has been applied based on past measurements. Zero sediment concentration has been assumed at the south boundary. Initial sediment concentration in the Meghna Estuary is also based on the field measurement by IWM and MES measurement.

The governing equations used in MIKE21 in solving hydraulic problems in coastal areas are:

Conservation of mass equation

$$\frac{\partial \varepsilon}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=0; \qquad (1)$$

Conservation of momentum equation

The momentum equation in the x-direction is given by:

$$\frac{\partial p}{\partial t}+\frac{\partial p^2/h}{\partial x}+\frac{\partial pq/h}{\partial y}+gh\frac{\partial \varepsilon}{\partial x}+\frac{f}{2}\frac{\sqrt{p^2+q^2}}{h^2}p-\frac{1}{\rho}\frac{\partial \tau_{xy}h}{\partial y}-\Omega q-\frac{\rho_a}{\rho}C_w WW_x+\frac{h}{\rho}\frac{\partial p_a}{\partial x}=0 \quad \text{(2a)}$$

The momentum equation in the y-direction is given by:

$$\frac{\partial q}{\partial t}+\frac{\partial q^2/h}{\partial y}+\frac{\partial pq/h}{\partial x}+gh\frac{\partial \varepsilon}{\partial y}+\frac{f}{2}\frac{\sqrt{p^2+q^2}}{h^2}q-\frac{1}{\rho}\frac{\partial \tau_{yx}h}{\partial x}+\Omega p-\frac{\rho_a}{\rho}C_w WW_y+\frac{h}{\rho}\frac{\partial p_a}{\partial y}=0 \quad \text{(2b)}$$

Where,

p and q flux in x and y directions respectively ($m^3/s/m$)

t	time (s), x and y (m) are Cartesian Co –ordinate (s)
h	water depth (m)
g	acceleration due to gravity (9.81 m^2/s)
ε	sea surface elevation (m)
ρ_w & ρ_a	air and water density respectively (kg /m^3)
Cw	wind friction factor = 0.0008 + 0.000065W in accordance
with Wu (1982)	
W	wind speed (m/s)
Ω	Coriolis parameter =5.2*10-5 s-1 in the Bay of Bengal
Pa	atmospheric pressure (kg/m/s^2)

Model Calibration

The updated two-dimensional hydrodynamic and sediment transport models of the Bay of Bengal have been calibrated against water level, discharge and suspended sediment concentration at different locations comparing the model results with field measurement to make the model performance to a satisfactory level. The validation has been made only with the available data for the year 2008. Data was collected for monsoon in 2009 and dry season in 2010. Locations of calibration areas have been presented in Figure 5 and some calibration plots in Figure 6 to 8.

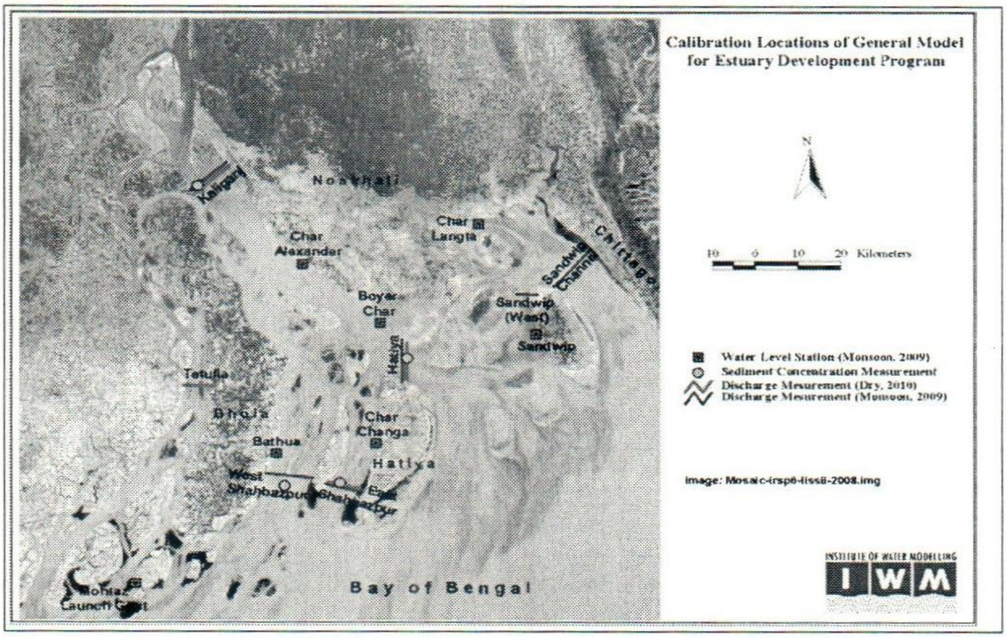

Figure 5: Calibration locations of Bay of Bengal Model

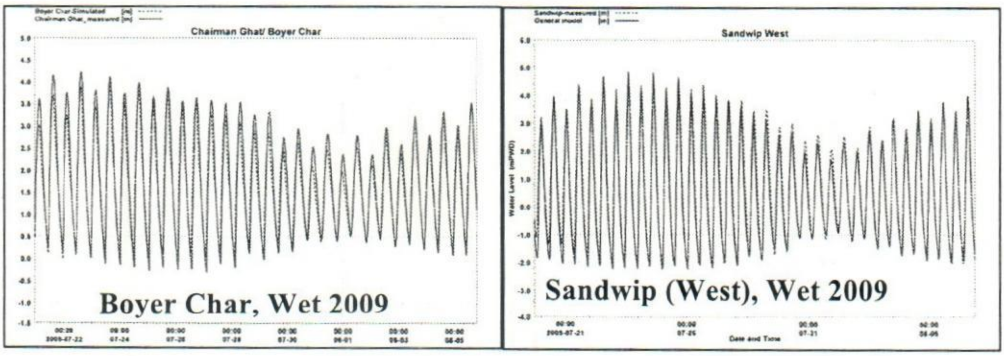

Figure 6: Calibration results of Bay of Bengal Model on water level

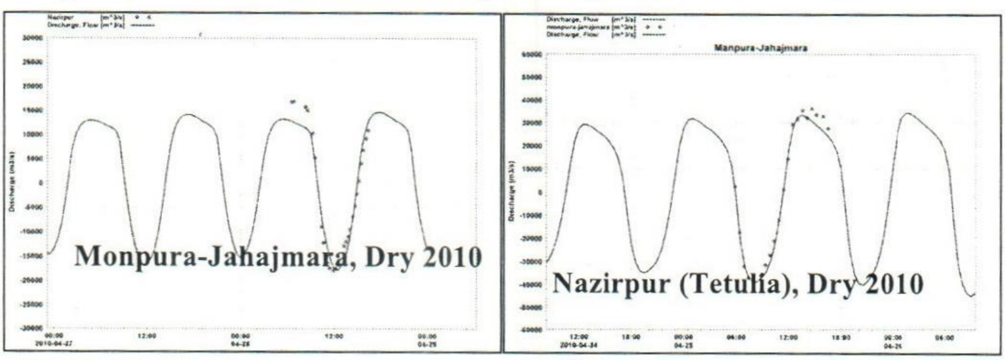

Figure 7: Calibration results of Bay of Bengal Model on discharge

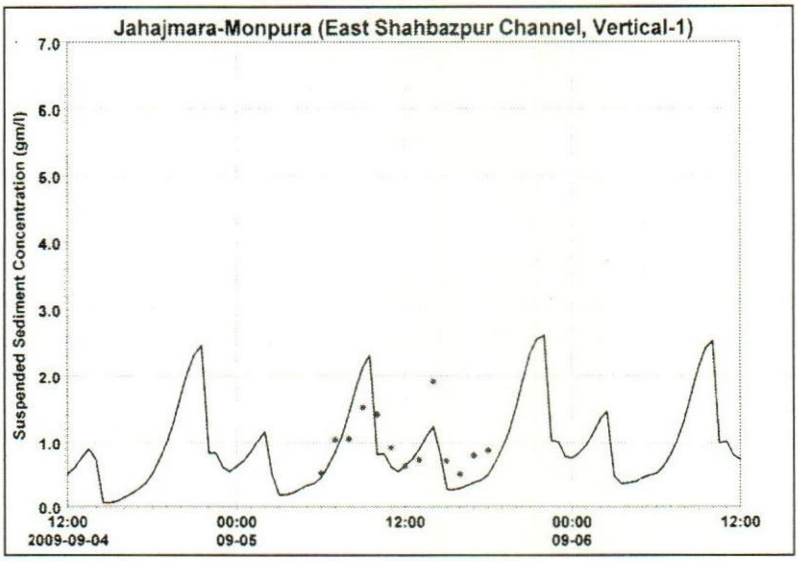

Figure 8: Calibration results of Bay of Bengal Model on suspended sediment concentration

Sediment Transport Process

Erosion

The erosion rate is a function of stream power, whereas sedimentation rate is a function of suspended sediment concentration (SSC), settling velocity of sediment particles or sediment flocks and local velocity field. The erosion of a bed layer is the transfer of sediment from the bed to the water column. Erosion takes places from the active bed layer in areas where the bed shear stress (τ_b) is larger than the critical shear stress for erosion (τ_{ce}). For dense consolidated bed the erosion rate for the j'th layer is described as (Partheniades, 1989):

For soft, partly consolidated bed the erosion rate for the j'th layer is described as (Parchure & Mehta, 1985):

$$E^j = E_0^j p_E^{j^{E_m}}$$

Where P^j_E is a probability ramp function of erosion, E_0 is erosion coefficient, E_m is the power of erosion, α is a coefficient, τ_{ce} is critical shear stress for erosion and τ_b is bed shear stress.

$$E = E_0^j \exp(\alpha(\tau_b - \tau_{ce}^j))$$

The Probability ramp function of erosion is defined as:

$$p_E^i = \max\left(0, \frac{\tau_b}{\tau_{ce}^i} - 1\right)$$

Deposition

The deposition of suspended sediment is the transfer of sediment from the water column to the bed. Deposition takes places where the bed shear stress (τ_b) is smaller than the critical shear stress for deposition (τ_{cd}).The deposition for the i'th mud fraction is described as (Krone, 1962):

$$D^i = w_s^i c_b^i p_D^i$$

Where P^i_D is a Probability ramp function of deposition, W_s is the fall velocity and c_b is the near bed concentration of fraction i.

The Probability ramp function of deposition is defined as:

$$p_D^i = max\left(0, min\left(1, 1 - \frac{\tau_b}{\tau_{cd}^i}\right)\right)$$

Sediment concentration during spring tide than that of during neap tide is always higher and the concentration also varies from monsoon to dry season. The field measurements in the Sandwip area show that sediment concentration during spring tide is about 4 to 10 times higher than that of neap tide (IWM, 2010). The major parts of the suspended sediment comprises of very fine sand and silt. The median bed-material grain size varies from 16 to 250 μm (MES, 2001). The sediment transport model provides the erosion deposition pattern in the estuary, which is found similar to the field conditions based on bathymetric survey i.e. location of erosion and deposition are more or less same though magnitude is different to some extent. The simulation results also include sediment transport through the lower Meghna River and the net sediment transport.

The upland sediment carried by river is trapped in the estuary by tidal pumping and residual circulation and mixed with the sediment brought from the continental shelf. The mixing process leads to continuous deposition and erosion and exchange of sediment in the estuary. More sediment measurements during monsoon and dry seasons are needed to enhance the understanding of accretion and erosion process and distribution of sediment deposition.

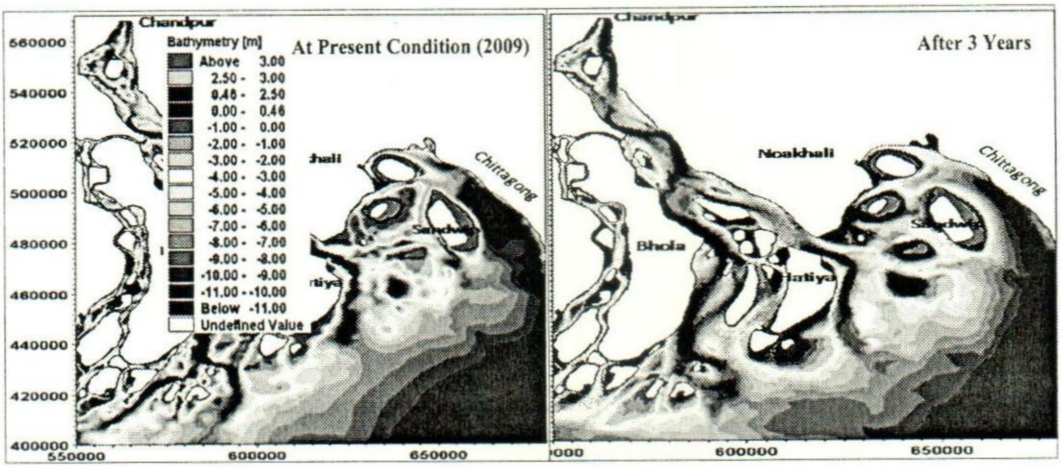

Figure 9: Simulated Bathymetry at present condition (Left) and after three years (Right).

Sediment Budget & Land Erosion-Accretion in the Estuary

Analysis of time series bathymetric chart collected under different project over the years has been made to assess the spatial distribution of sediment deposition and erosion. The Natural Neighbour method is used for interpolation of surveyed data to develop sea bed topography. The bathymetric chart based on the surveyed data under EDP in 2009-10 has been compared with that of 1999-2000 under MES II study to estimate erosion and deposition volume in the Meghna estuary. In the Sandwip-Urirchar-Noakhali area, bathymetric data surveyed by IWM in 2008 under CDSP III project have been compared with the bathymetric data of 1999 collected under MES II.

The entire study area was divided into seven sub-areas delineated in the MES II study. In each sub-area bathymetric maps have been prepared for both 1999 and 2009 data using MIKE 21. The overall difference of bathymetry map for the seven sub-areas is presented in Figure 10. Erosion and deposition volumes have been calculated based on interpolated depth values of the maps using 50x50m grid squares.

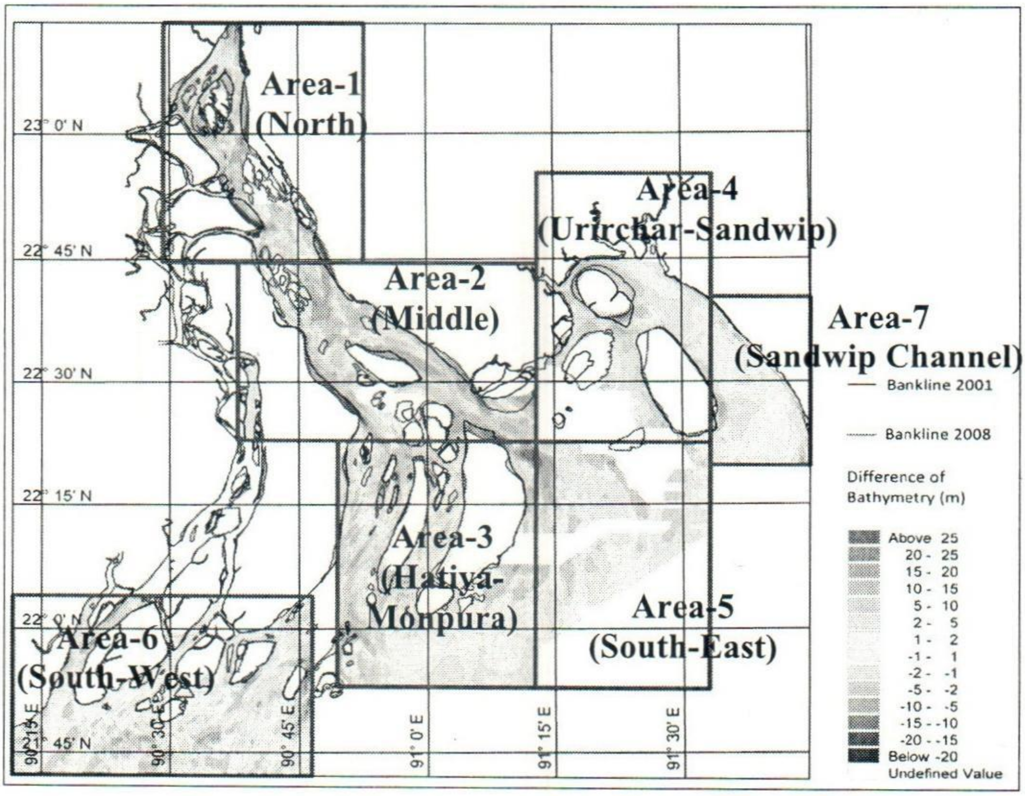

Figure 10: Bed level changes in seven sub-areas of Meghna Estuary (1999-2009)

Sediment budget calculations identify sediment sources and sinks in a delta system and allow predictions of how changes to the rate of sediment import or export might be reflected in a delta's construction or deconstruction (Kimberly et al., 2013). After analysing of bathymetric data from 1999-2009 (Figure 10) and 3 years morphological simulation (Figure 9) it can conclude that erosion process is dominant in the upper region of the estuary part i.e. from Chandpur to north of Bhola island, north coast of Hatiya Island and northeast coast of Bhola and along the Boyer char (Ramgati). The analysis of bathymetric data shows that accretion process exceeds the erosion process in the Meghna Estuary. The net deposition of sediment over the last ten years period in the entire estuary was about 0.05 m/year. The highest rate of deposition (0.22m/year) is seen in the Sandwip-Urirchar region. The erosion and deposition volumes estimated for the sub-areas are shown in Table 2.

Table 2: Net change of sediment volume in the Meghna Estuary

Sub area	Erosion		Deposition		Net Deposition			Net Change
	Volume (Mm^3)	Area (ha)	Volume (Mm^3)	Area (ha)	Volume (Mm^3)	Volume (Mm^3/yr)	Total Area (ha)	m/yr
A1	1467	27411	892	23694	-574	-57	51105	**-0.11**
A2	2050	45400	1908	44707	-142	-14	90107	-0.02
A3	1408	64956	1941	93801	533	53	158756	0.03
A4	366	16838	1594	44758	1228	136[1)]	61596	**0.22**
A5	547	25934	1492	60422	944	94	86356	0.11
A6	672	35516	1138	59376	465	47	94892	0.05
A7	139	13383	444	34104	305	31	47487	0.06
Total	6649	229438	9409	360860	2760	276	590298	0.05

1) The yearly change has been estimated dividing the total change by 9 years because 2008 survey was used instead of 2009 survey.

Conclusions

The analysis of bathymetric data shows that accretion process exceeds the erosion process in the Meghna Estuary. Erosion process is dominant in the upper region of the estuary part i.e. from Chandpur to north of Bhola island, north coast of Hatiya island and northeast coast of Bhola and along the Boyer char (Ramgati). The highest rate of deposition is seen in the Sandwip, Jahajerchar and Urirchar area. The net deposition of sediment over the last ten years period in the entire estuary is about 2,760 Mm^3 and annual average is about 276 Mm^3, which is very high compared to a similar analysis done under MES II study. In MES II study about 300 Mm^3 deposition was found in three years in the estuary. The remarkable difference with the MES analysis is in area 5, where present study shows huge net deposition of sediment instead of erosion as found in the MES II. It is essential to establish consistent bench mark in the estuary and to continue the bathymetric survey of the entire estuary for better understanding of the morphological processes and identification of potential location for land reclamation.

Acknowledgement

The authors express their deepest gratitude to the Institute of Water Modelling (IWM), Bangladesh Water Development Board (BWDB), Meghna Estuary Survey (MES) team for the support to use their study findings in the paper. They also thankful to Danish Hydraulic Institute (DHI) for using MIKE 21FM Modelling tool.

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OPTIMIZATION OF BANK PROTECTION WORKS BY PHYSICAL MODELLING FOR THE PROTECTION OF KUSHTIA TOWN AT RIGHT BANK OF GORAI RIVER

Swapan Kumar Das[1], A.K.M.Ashrafuzzaman[2] and Md. Abul Ala Moududi[3]

Abstract

Physical modelling is an effective tool to investigate any river related problems and in particular to the bank protection and river training works optimization. Bangladesh is a riverine country where severe bank erosion is a usual problem that is to address each year by the Bangladesh Water Development Board (BWDB). A lot of money is required for the construction of river training and bank protection works. Some river bank structures are performing well in the field and some are not. So before going to invest a significant amount of money it is better to investigate its effectiveness by physical modelling. This paper presents the findings and optimization of Kushtia town protection strengthening works as proposed by the BWDB. A physical model study was carried out at River Research Institute (RRI) for the protection of right bank of Gorai River from upstream of proposed spurs to Shashanghat at Kushtia town. The model was developed to find out the erodible area where bank protection is required and to investigate different options to find out its effectiveness and to optimize the bank protection/river training structures in terms of length, location and orientation for the area to be protected. The study also includes investigating the extent of local scour around the structures, velocity at different locations, flow field etc. The study reveals that two spurs at the upstream of the existing groyne G3 or 1424 m revetment and 3100 m revetment from existing groyne G2 to Kaliganga outfall are required to protect the aforesaid areas.

Introduction

Kushtia Town is situated on the right bank of Gorai river only about 8.0 km downstream from the Gorai offtake and hence, the water level variation between the offtake and Kushtia is not significant. Due to continuous rise of bed level at the offtake and also along its upper reach, the conveyance capacity of the river is reducing, resulting in a reduced flow through the Gorai even during monsoon time. However, due to change in the morphological behavior, the Gorai has a trend to be widen, which causes bank erosion.

Kushtia Town was already protected by flood embankment, but there was an urgent need to improve the level of protection embankment against increased threat of flooding. And, also the Government Policy as envisaged in the National Water Management Plan (NWMP) is to enhance the existing flood protection level to 1 in 100 year return period for the cities and towns. The existing level of flood protection often does not meet the requirements for many towns (RRI, 2008).

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The existing flood embankment is about 14.0 km long along the Gorai right bank starting from the Gorai off take to the downstream of Kushtia Town. There were only three groynes to protect a limited area of the right bank near the Kushtia Town. Therefore, it was proposed to construct two more solid spurs upstream of the existing groyne-3(G3) because of severe bank erosion upstream of groyne G3. The findings of the Gorai River Restoration Project (GRRP) reflect that the morphology of the upper Gorai is very complex and it has a tendency of frequent changing. So, proper study should be carried out before undertaking any protection works along the right bank of the Gorai. Hydraulic modelling is a form of physical modelling widely used to investigate design and operation uses in hydraulic engineering (ASCE, 2000). As a result, a physical model study was undertaken to investigate the consequences due to the project interventions and also the recommendation of further adjustments of the proposed interventions in order to achieve optimum benefit from the project (RRI'2008).

Methodology

An overall model with horizontal scale of 1:200 and vertical scale 1:50 and a detail model with 1: 100 were set up fulfilling certain rules and conditions of physical modelling. Since geometric, kinematic and dynamic similarity is to be maintained between model and prototype (Sharp, 1981); a compromise is to made between the possibilities and limitation of the model. So, the model in principle was designed to ensure sufficient sediment movement and fulfilling the Froude condition.

An indoor model facility of RRI having dimension of 80m×23m was used for the setting up and construction of the model. About 9km river reach and full width of the river was reproduced in a distorted model. A detail model of undistorted scale was also setup where river length was about 2.0 km and full river width. Both the models were constructed based on the topographic, bankline and bathymetric survey data of September 2007 (IWM, 2007). 1 in 100-years water level (14.44m PWD) at Gorai Railway Bridge is calculated from the available data and used for the model study. Water surface slope was taken as 0.000044. The model was run with 1 in 100 years discharge of 11,149 m^3/s at Gorai Railway Bridge. Scale conditions for distorted and undistorted model are summarized in Table 1 and Table 2.

Table 1: Scale conditions for different basic and derived parameters (distorted model)

Parameter	Unit	Prototype	Model	Scale Factor
Length, L	m	9000	45	200
Average width, W	m	950	4.75	200
Water depth, h	m	6.00	0.12	50
Water surface slope, i	-	0.000044	0.000176	0.25
Average velocity, v	m/s	1.92	0.27	7.07
Cross-sectional area, A	m^2	5821	0.582	10000
Critical velocity, v_{cr}	m/s	0.387	0.257	1.50
Chezy roughness co-efficient, C	$m^{1/2}/s$	85.06	30	2.84
Discharge, Q	m^3/s	11149	0.158	70711
Shields parameter, θ	-	1.003	0.161	6.25
Critical Shields parameter, θ_{cr}	-	0.067	0.134	0.50
Froude number, Fr	-	0.249	0.249	1
Reynolds number, Re	-	9606998	27173	353.55

Table 2: Scale conditions for different basic and derived parameters (undistorted model)

Parameter	Unit	Prototype	Model	Scale Factor
Length, L	m	2500	25	100
Average width, W	m	950	9.50	100
Water depth, h	m	6.00	0.06	100
Water surface slope, i	-	0.000044	0.000044	1
Average velocity, v	m/s	1.92	0.19	10
Cross-sectional area, A	m^2	5821	0.582	10000
Critical velocity, v_{cr}	m/s	0.387	0.235	1.64
Chezy roughness co-efficient, C	$m^{1/2}/s$	85.06	30	2.84
Discharge, Q	m^3/s	11149	0.111	100000
Shields parameter, θ	-	1.003	0.020	50
Critical Shields parameter, θ_{cr}	-	0.067	0.134	0.50
Froude number, Fr	-	0.249	0.249	1
Reynolds number, Re	-	9606990	9607	1000

Model Calibration

It is Calibration of the physical model is a very important step among others to get reliable outputs from the model investigation. In the model, the calibration was mainly aimed at adjusting the incoming prototype flow distribution at the approach section and water level at the downstream section. Velocity at the calibration section was measured and compared those with the prototype values as shown in Figure 1 & 2 for distorted as well as undistorted model respectively. It is evident from these figures that the simulated values found in the model are very close to the prototype value.

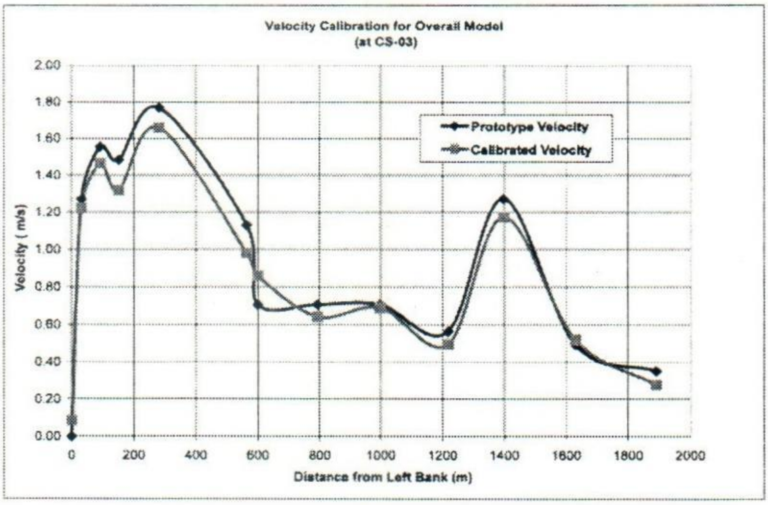


Figure 1: Velocity calibration in overall (distorted) model

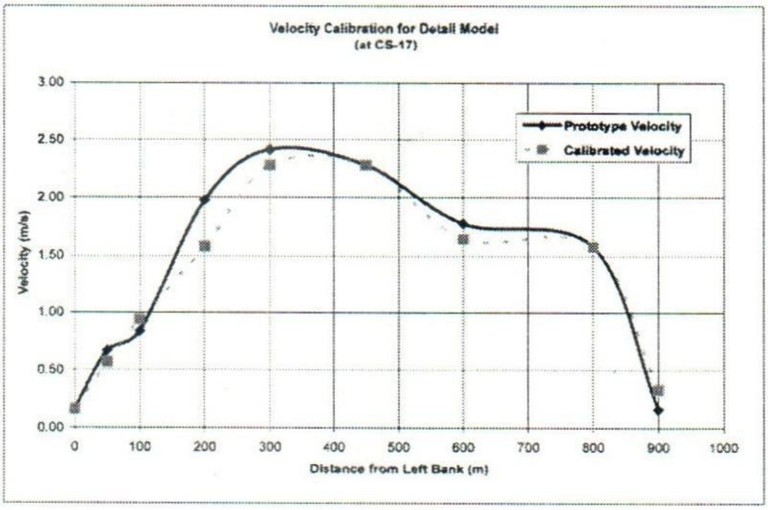


Figure 2: Velocity calibration in detail (undistorted) model

Water level was maintained at the downstream end of both the models as downstream boundary condition. Water level in the model was also found in good agreement with the prototype water level.

Analysis of the result and its interpretation

Total six test runs were carried out to achieve the goal of the study of which four tests were for the distorted overall model and two tests for the undistorted detail model. During calibration of the model. It was found in general that the erodible bank velocity was present throughout the whole study area with significant emphasis at the right bank, upstream of existing groyne G3. Velocity was prominent at Haripur and Ghoraghat area for the left bank. This situation was observed from the flow lines and photo during calibration test as shown in Figure 3 and Photo 1. In reality, the situation may vary with different discharge conditions. Typical velocity along the cross-section (CS#27) at about 100 m upstream of groyne G3 is shown in Figure 4. Near bank velocity to the magnitude of 1.5-2.0 m/s was found along the right bank upstream of existing groyne G3. The velocity at both banks as a whole was observed within 1.0-2.0 m/s. The flow concentration was much higher around the existing groynes G3 & G1 compared to the existing groyne G2. Maximum velocity was measured about 3.42 m/s during calibration test at CS#27. Turbulence due to velocity was higher at existing groynes G3 & G1 compared to the groyne G2. In addition to the above mentioned reasons, the upstream river characteristics due to its concave nature entails that river training/bank protection work is essential at the upstream of existing groyne G3 to protect the upstream.

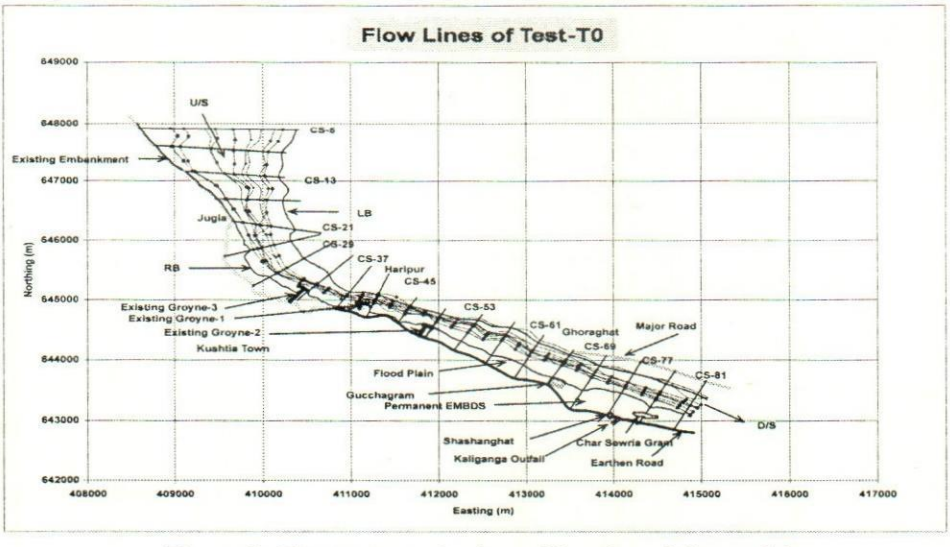


Figure 3: Flow pattern during calibration of the model

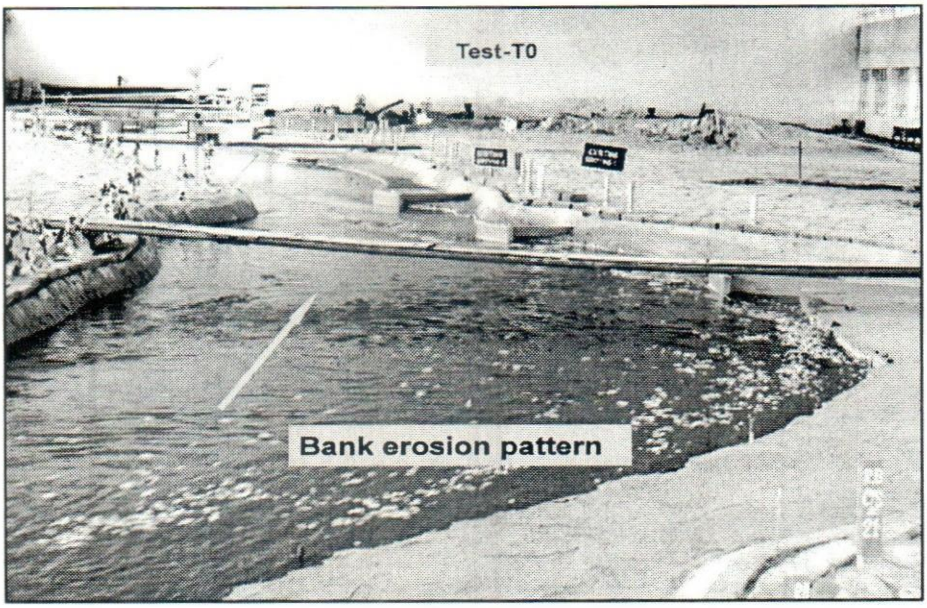


Photo 1: Bank erosion at the upstream of the groyne G3

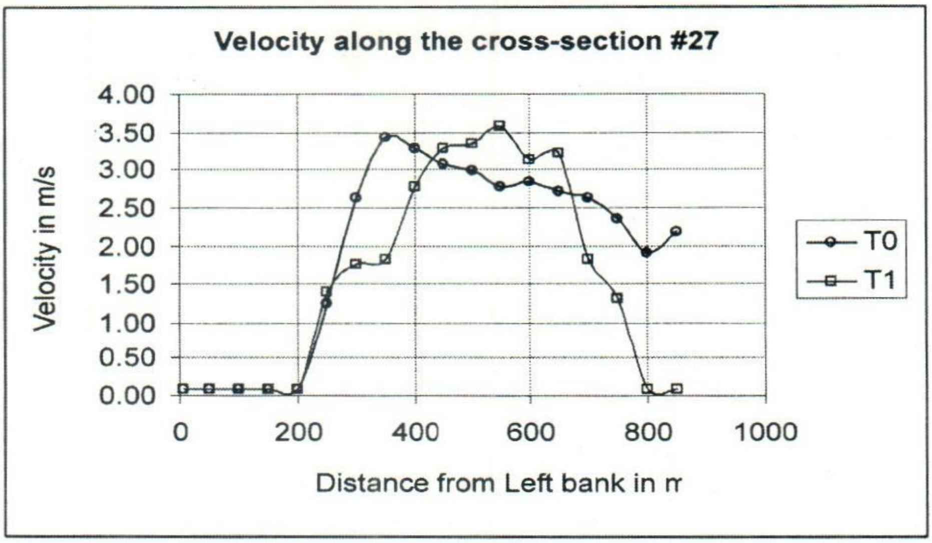


Figure 4: Velocity along the cross-section (CS#27) in test T0 and T1

Results of test T1 shows that the location of the proposed two spurs are to be shifted than proposed to protect longer length of upstream right bank from the existing groyne G3. Revetment between existing groynes G3 & G2 is not essential to protect the bank in between, except land reclamation is required. Embankment cum revetment from existing groyne G2 to Shashanghat was found to work effectively.

Velocity data of test T2 indicates that the changed location and length is quite capable of protecting longer length compared to test -T1. However, another test was done to carryout with a view to refine the length and location of the spurs.

During test T3 it was found that the flow velocity near the bank was about 1.0 m/s between spurs S1 & S2. Moreover, flow indicates attacking tendency to the bank between the spurs. Under these circumstances, the length and spacing of the spurs tested in test T3 could only be selected allowing some bank erosion between the spurs. Otherwise, for safety, it is better to select the option of test T2 (Photo 2). It is worthwhile to mention here that the intensity of velocity is increased around the existing groynes G3 & G1. Test-wise structural interventions for application tests T1 to T4 is depicted in Table 3.

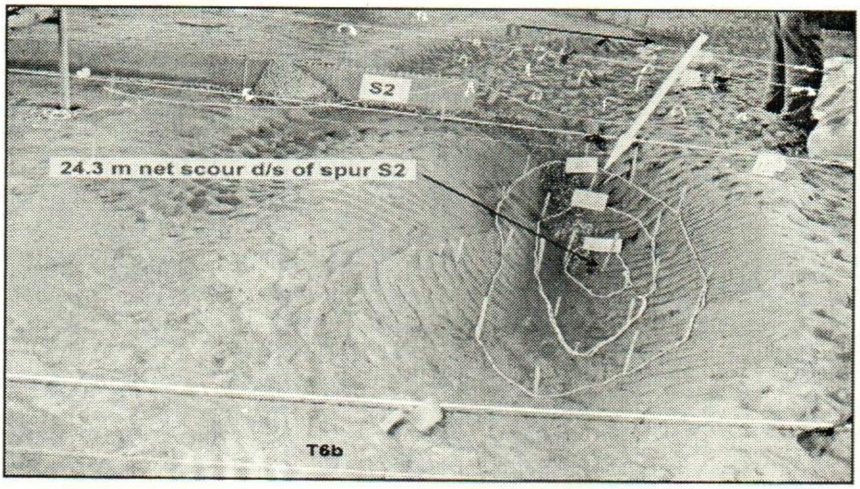


Photo 2: Typical local scour downstream of the proposed spur

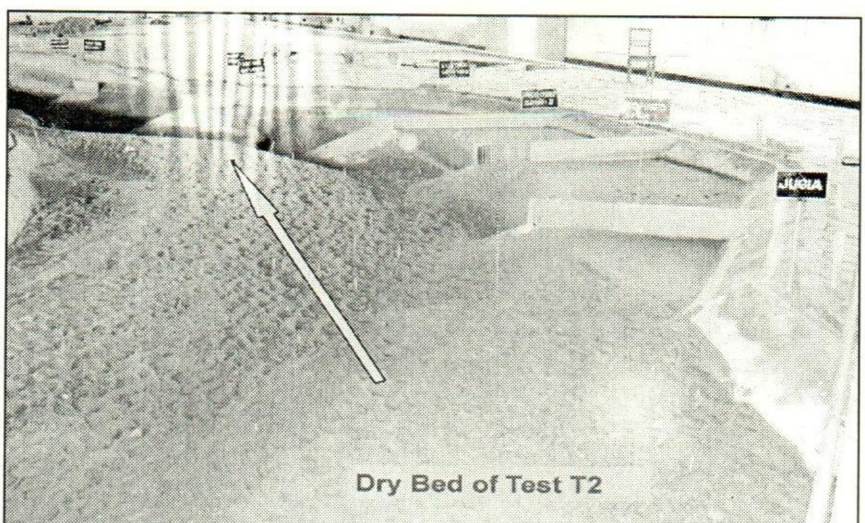


Photo 3: Optimized location of the proposed spurs

Table 3: Test-wise structural interventions

Test No.	Structural interventions	Proposed spurs position	Proposed revetment position
T1	• Proposed spurs at upstream • Proposed revetment at downstream	Spur 1 Between CS#26-27 Total length: 389m Spur 2 Between CS#28-29 Total length: 329m	At downstream from G1 to Shashanghat, Length: 3850m
T2	• Proposed spurs at upstream (modified-1) • Proposed revetment at downstream	Spur 1 Between CS#23-24 Total length: 303m Spur 2 Between CS#27-29 Total length: 421m	At downstream from G1 to Shashanghat, Length: 3850m
T3	• Proposed spurs at upstream (modified-2) • Proposed revetment at downstream	Spur 1 Between CS#21-23 Total length: 397m Spur 2 Between CS#25-27 Total length: 220m	At downstream from G1 to Shashanghat, Length: 3850m
T4	• Proposed revetment at upstream, alternative to spurs • Proposed revetment at downstream	Nil	• At upstream from G3 to Jugia, Length: 1424m • At downstream from G1 to Kaliganga outfall, Length: 3900m

Maximum scour at existing flow and oblique flow condition at different location is shown in Table 4. It is seen from this table that maximum net scour of 17.4 m (-4.8 mPWD), 10.4 m (-1.6 mPWD) and 35.0 m (-30 mPWD) was measured at the upstream termination, middle alignment and downstream end of the revetment respectively. The nature of scour was changed slightly at the oblique flow area only; otherwise the overall observation is similar to that of existing flow condition. The length of launching apron tested in the model was 9.0 m, 7.0 m and 12.0 m at the upstream termination, middle alignment and downstream end of the revetment respectively. However, it would be wise to optimize it from the scour value obtained from the model study. Maximum velocity was measured about 2.33 m/s and 2.13 m/s at upstream termination and downstream end of revetment.

Table 4: Maximum scour and velocity in the vicinity of revetment and spurs for various test conditions

Test No.	Types of Intervention	Flow Condition	Maximum Scour (m)	Maximum Velocity (m/s)
T5	Upstream Termination of Revetment	Present flow	10.30	2.33
		45-degree flow attack	17.40	-
	Middle Alignment of Revetment	Present flow	6.60	2.80
		45-degree flow attack	10.40	-
	Downstream end of Revetment	Present flow	35.00	2.13
		45-degree flow attack	34.50	-
T6	Spur-1	Present flow	12.60	2.82
		45-degree flow attack	11.80	-
	Spur-2	Present flow	24.30	3.83
		45-degree flow	22.80	-

Conclusive Discussions

It was evident from the model study that either revetment or spurs would be effective to combat bank erosion at the upstream of the existing groyne G3. The present field condition exhibits that there is a big char immediate upstream of existing groyne G3, causing the flow concentrated at the right bank and which was one of the major reasons of upstream bank erosion. So, considering the model results as well as the field condition it is assumed that in addition to the existing bank protection works, training of the river could provide better performance to divert the flow towards the main river by eroding the char to some extent. This is why, in addition to the bank protection, river training if envisaged, in these circumstances proposed RCC spurs could be better. But, the spur construction, its maintenance and regular monitoring if not done properly, it might not be effective. The performance of such RCC spurs in the field in different rivers of the country is not also encouraging. On the other hand, revetment is an adapting measure against the flow and any local damages for revetment would not affect so much compared to the spurs. Under those circumstances, revetment was recommended to combat bank erosion at the upstream of existing groyne G3. Right bank from the existing groynes G3 to G1 appears to be reached at a stable condition and no further erosion was expected in between. So, for bank protection purposes revetment is not required between the existing groynes unless the concept of land reclamation is there. Some damages of the existing groynes G3 and G1 was observed during field visit, which requires maintenance. In brief, it can be said that the existing three groynes (G1, G2 & G3) are working effectively, but regular monitoring and maintenance is essential.

The proposed revetment from the existing groyne G2 to Kaliganga outfall was found to work well but bery higher velocity of about 2.0-2.5 m/s at and around the Char Sewria Gram was observed. As a result the proposed alignment was changed slightly to reduce the velocity around the Char Sewria Gram. Maximum velocity for revetment around the Gucchagram was measured about 3.1 m/s with 100 years discharge event.

Recommendations

Based on the findings of the physical model study, the following structural measures are recommended:

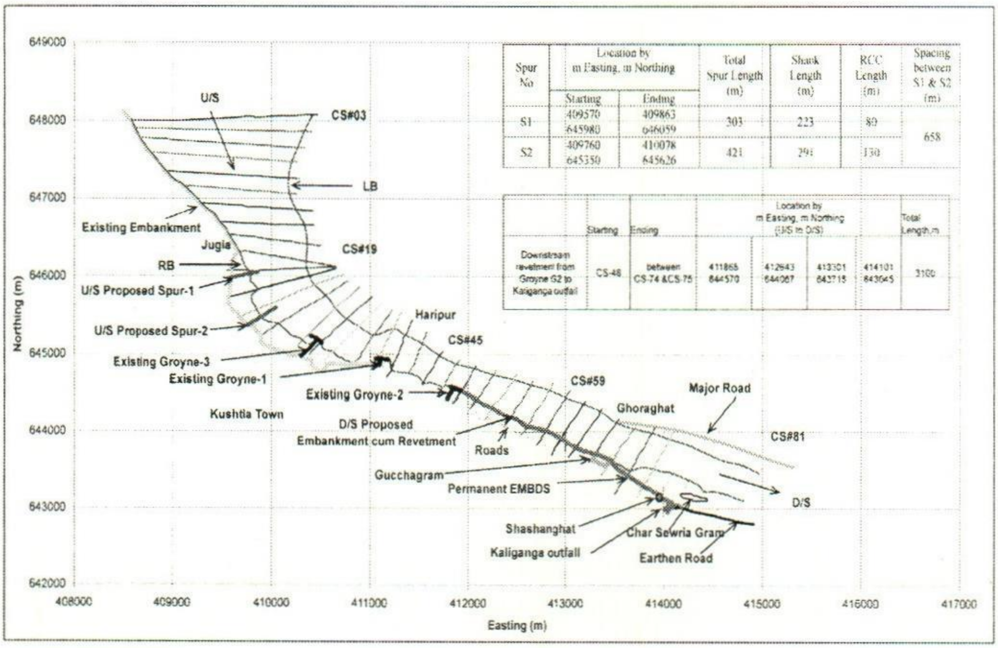
- Two spurs at the upstream of the existing groyne G3 and 3100 m revetment cum embankment from existing groyne G2 to Kaliganga outfall. This recommended option-1 is shown in Figure 5.

- 1424 m revetment from the upstream of the existing groyne G3 to Jugia and 3100 m revetment cum embankment from existing groyne G2 to Kaliganga outfall. This recommended option-2 is shown in Figure 6.

The recommended design parameters for selected structures are presented in Table 5.

Table 5: Design Parameters

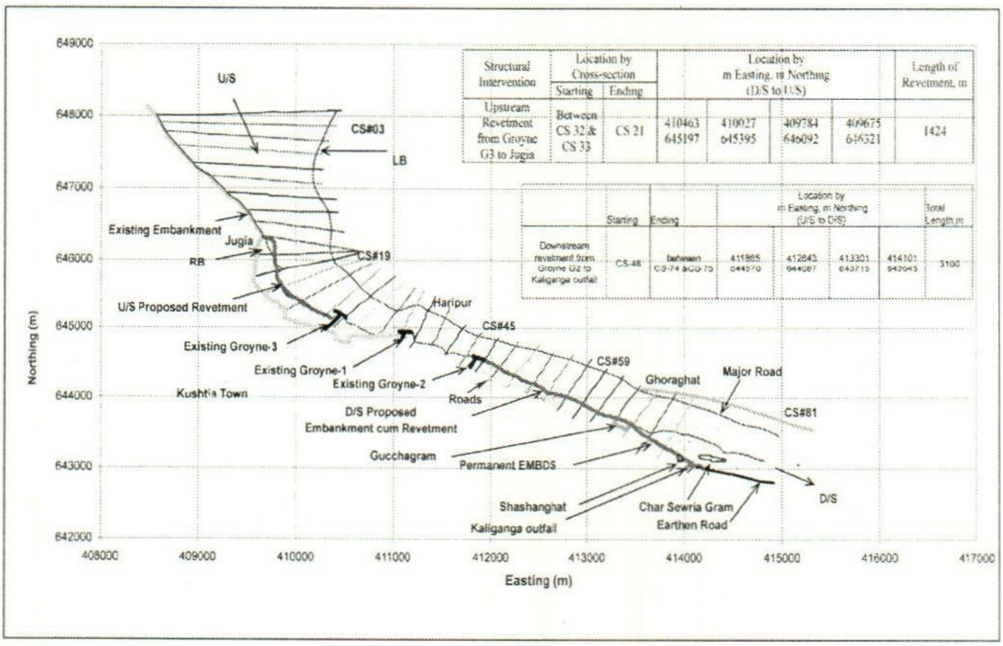
Structure type	Maximum net scour around the spur/revetment	Maximum velocity around the spur/ revetment
Spur	12.6m (-9.6 mPWD) for spur S1 24.30 m (-20.8 mPWD) for spur S2 17.4 m (-4.8 mPWD), at the upstream termination	2.82 m/s at spur S1 3.83 m/s at spur S2 Around upstream termination 2.33 m/s
Revetment	10.4 m (-1.6 mPWD), middle alignment of the revetment 35.0 m (-30.0 mPWD), downstream end of the revetment	- Around downstream end of revetment 2.13 m/s



Spur No	Location by m Easting, m Northing		Total Spur Length (m)	Shank Length (m)	RCC Length (m)	Spacing between S1 & S2 (m)
	Starting	Ending				
S1	409570 645980	409863 646059	303	223	80	658
S2	409760 645350	410078 645626	421	291	130	

	Starting	Ending	Location by m Easting, m Northing (U/S to D/S)				Total Length,m
Downstream revetment from Groyne G2 to Kaliganga outfall	CS-48	between CS-74 &CS-75	411865 644570	412643 644067	413301 643715	414101 643645	3100

Figure 5: Layout of recommended option-1



Structural Intervention	Location by Cross-section		Location by m Easting, m Northing (D/S to U/S)				Length of Revetment, m
	Starting	Ending					
Upstream Revetment from Groyne G3 to Jugia	Between CS 32 & CS 33	CS 21	410463 645197	410027 645395	409784 646092	409675 646321	1424

	Starting	Ending	Location by m Easting, m Northing (U/S to D/S)				Total Length,m
Downstream revetment from Groyne G2 to Kaliganga outfall	CS-48	between CS-74 &CS-75	411865 644570	412643 644067	413301 643715	414101 643645	3100

Figure 6: Layout of recommended option-2

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DETERMINATION OF DESIGN PARAMETERS FOR REVETMENT USING PHYSICAL MODELLING: A CASE STUDY

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Abstract

An undistorted physical model study was carried out at River Research Institute (RRI) to determine the effectiveness and design parameters of the revetment for the protection of Faridpur town from the right bank erosion of Padma River. In this study revetment having length 609m was reproduced in the model with scale 1:60 and is then subjected to normal and oblique flow conditions with different angle of flow attack. The model study result reveals that the revetment appears to work satisfactory under normal flow condition. But under oblique flow conditions some geobags have been displaced away from the launching apron at the attacking point of revetment due to high velocity of flow. The maximum value of the design parameters in terms of velocity & scour around the end of launching apron of revetment is found 3.43m/s and 6.42m respectively under normal flow condition whereas these values are 3.99m/s and 9.54m respectively under oblique flow situation. The study also included observing flow patterns and flow concentration around the structure.

Introduction

The right bank of Padma River at Faridpur Sadar has become vulnerable to severe erosion. From the available data it appears that the river has been divided into two channels following the development of a char in 1993 at 4.0km downstream of Goalanda Ghat. The south channel that flows parallel to Faridpur town has become prominent in the recent years, resulting in severe bank erosion in 10km stretch of the river. The river has eroded more than 4500 hectares of valuable land in the recent years. The last survey in September 2005 shows further increase in the curvature of the river bank due to erosion. The right channel causes severe bank erosion near Faridpur town, which is at present just 770m away from the Padma River. Figure 1 shows the bankline movement of this river near Faridpur Sadar from 2003 to 2006. It is envisaged that urgent sustainable measures have to be implemented after having adequate information from planform analysis and from physically based model studies. Under these circumstances, RRI has been commissioned to conduct a physical model study to determine the effectiveness of the of revetment as recommended by Mathematical Modelling study, to see the flow pattern and flow concentration around the structure, and to find out design parameters of the revetment such as velocity, scour depth etc.

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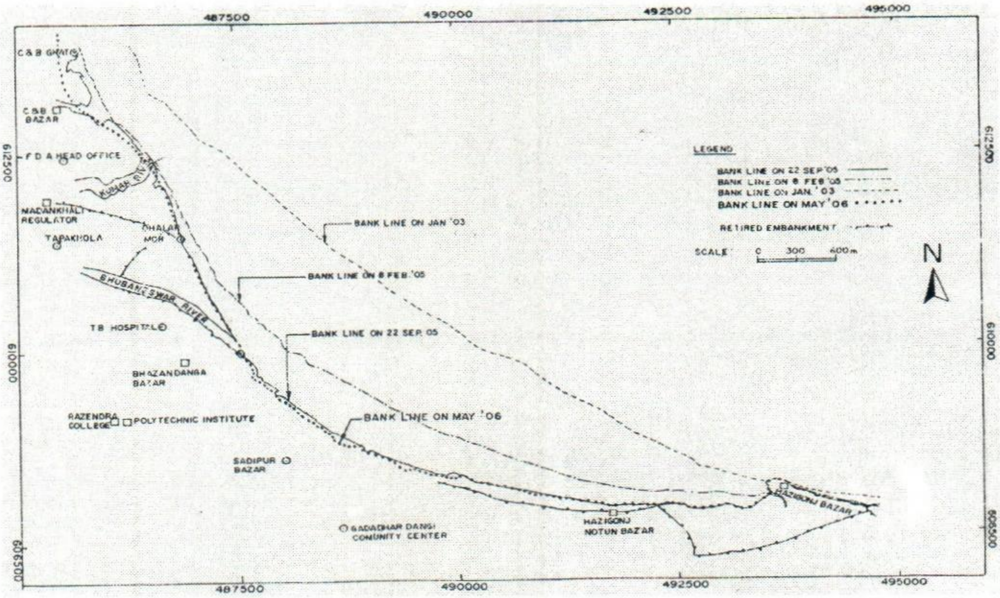

Figure 1: Bankline movement of Padma river from 2003-2006 near Faridpur Town (IWM, 2006)

Methodology

An open air model bed of RRI having dimension of 100mX60m is used for setting up the model. The model is undistorted and constructed as per bathymetry (survey done by Globe Survey Company) of 2005 of Padma river by considering both horizontal and vertical scale of 1:60. The model bed is moulded with sand (sand bed model). The model has been designed fulfilling the Froude's model law. The velocity in the model was such that sufficient sediment transport has been ensured in the model and the model has been expected to reach the dynamic equilibrium condition (no further scour with time) by running about 16-20 hours. Reynolds number should be high enough to ensure rough turbulent flow (for turbulent flow, Reynolds number >2000) in the model. From the selected geometric scales, the scales for other parameters are determined as shown in Table 1.

Table 1: Scale conditions for different basic and derived parameters

Parameters	Unit	Scale
Velocity (V)	(m/s)	7.75
Time (T)	(s)	7.75
Slope (I)	(-)	1
Froude number (Fr)	(-)	1
Reynolds number (Re)	(-)	464.76
Discharge (Q)	(m^3/s)	27885
Specific discharge (q)	(m^2/s)	464.76
Chezy's roughness co-efficient (C)	($m^{0.5}/s$)	2.67
Manning's roughness co-efficient (n)	($s/m^{0.33}$)	0.742

The length reproduced in the model was about 2000 m and an average river width of about 500m from right bankline of Padma River has been considered. The calibration test run (T0) in the model was done with the discharge 13,763 cumec corresponding to the water level 7.00 mPWD (top level of revetment as per design of revetment supplied by BWDB). The application tests (T1 & T2) were also conducted with the same discharge. The model setup is shown in Figure 2.

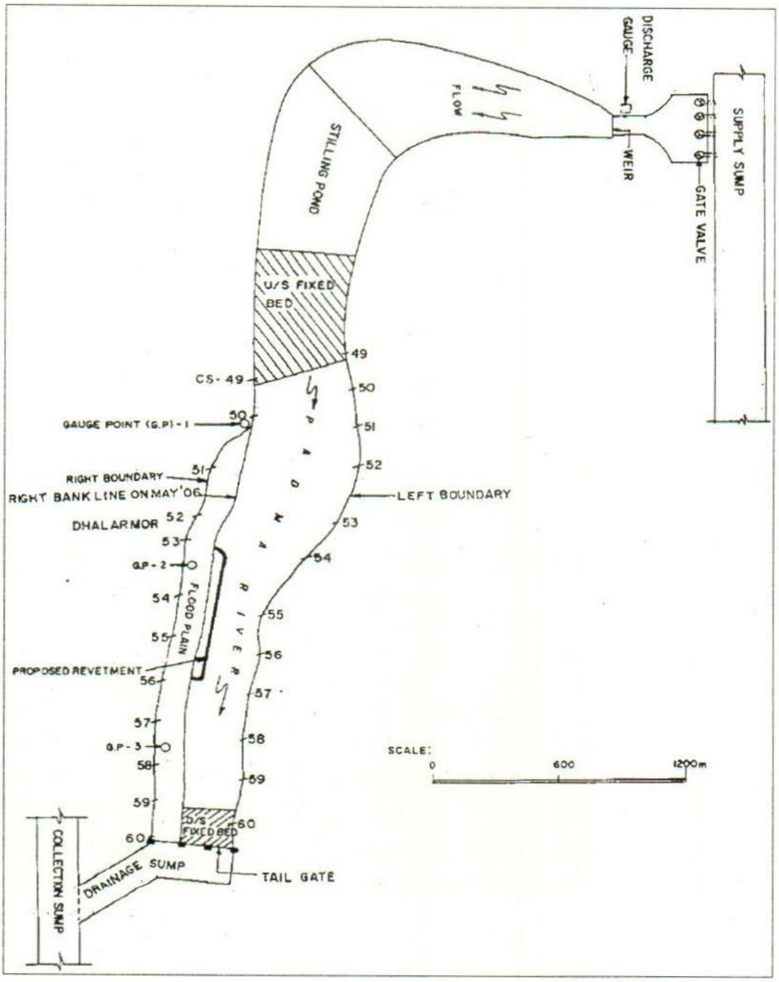

Figure 2: Model setup of detail model (RRI, 2007)

Model Calibration

The calibration of the model is aimed at adjusting the incoming prototype flow distribution at the approach section and water level at the downstream section. The velocity distribution at the upstream section was simulated at the upstream boundary of the model. The velocity distribution at the calibration section CS50 [at the upstream boundary of the model] was measured and compared with the prototype. A comparison between prototype and simulated velocity is shown in

Figure 3. From this figure it is evident that the simulated values found in the model are very close to the prototype value. The downstream boundary condition is the water level which has been maintained at the downstream end of the model. Here the water level found in the model was in good agreement with the prototype water level.

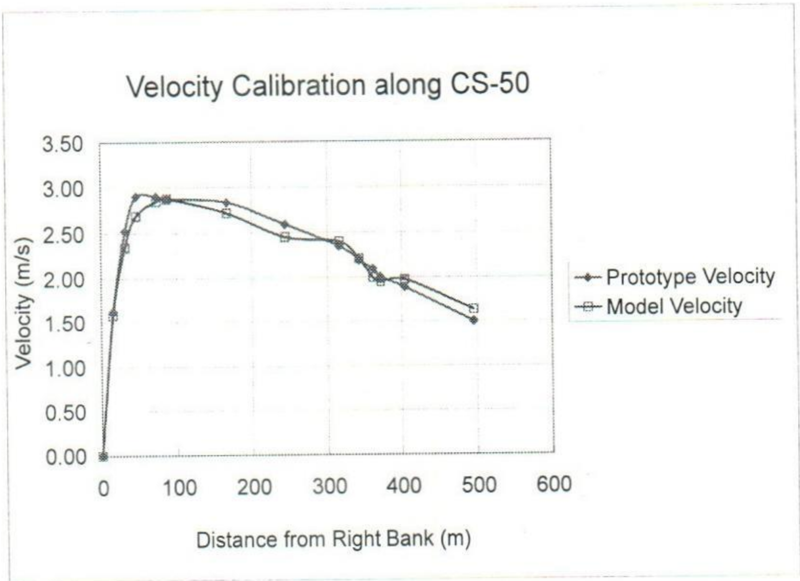

Figure 3: Comparison between prototype and model velocity at the calibration section

Test Scenarios

To combat bank erosion two application test runs with different test scenarios have been conducted in the model. Test T0 contributed to the calibration of the model. The application tests T1 through T2 were carried out with a view to observe the performance of the bank protection works under different flow conditions to prevent bank erosion. Each application test was run with the discharge to determine flow velocity, flow lines, bed level and scour depth in the vicinity of structures. Model is run until equilibrium bed level has been reached in the model. The test scenarios of the model are described in Table 2.

Table 2: Test scenarios for the model study

Test No.	Description	Test Conditions
T0	Calibration of detail undistorted model.	September 2005 bathymetry
T1	To observe the stability of revetment under normal flow condition.	Test condition of T0 with proposed revetment (CS53.5-CS56).
T2	To observe the stability of revetment under flow attacking condition.	Test condition of T1 with 60° angle of flow attack.

Analysis and Interpretation of the Test Results

Calibration Test (T0)

This test was conducted with the existing condition. The location of the selected points along different cross-sections for velocity measurement in Test T0, T1 & T2 can be seen in Figure 4. The velocities measured in the model at these selected points along cross-section from right bankline up to 180 m are shown in Table 3. From this table, it is observed that the measured bank velocities (at zero distances) are in the range of 1.05 to 2.80 m/s within the reach from CS51 to CS59.

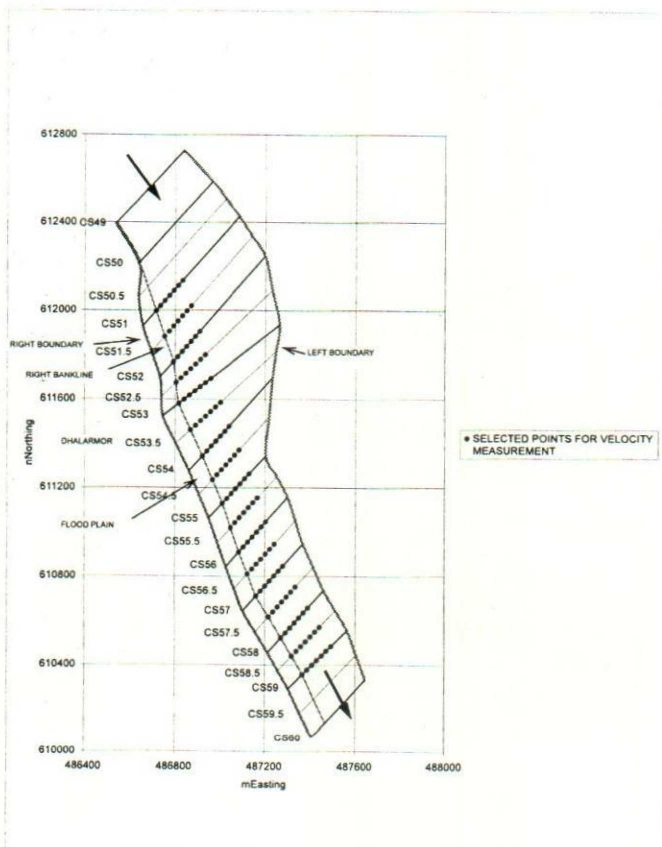

Figure 4: Location of the selected points for velocity measurement in Test T0, T1 & T2

Table 3: Flow velocity (m/s) in Test T0

CS No.	Right bankline		Velocity (m/s)						
	Easting (m)	Northing (m)	Distance from right bankline (m)						
			0	30	60	90	120	150	180
CS51	486716	611997	2.32	2.64	3.04	3.35	3.12	3.04	2.64
CS51.5	486754	611883	2.56	2.64	3.27	3.04	3.43	3.12	2.88
CS52	486792	611768	1.92	2.88	2.64	2.80	3.43	3.35	3.12
CS52.5	486805	611675	2.08	2.32	2.72	2.64	3.35	3.43	3.04
CS53	486817	611581	1.76	2.48	2.48	3.43	2.88	3.12	3.51
CS53.5	486869	611462	2.00	3.43	3.04	2.88	3.43	3.35	3.43
CS54	486921	611344	2.32	2.48	2.96	3.43	3.27	3.51	3.35
CS54.5	486965	611236	2.24	3.27	3.12	3.51	3.59	4.07	3.67
CS55	487010	611128	2.32	3.04	3.12	3.43	3.04	3.67	3.59
CS55.5	487047	611019	2.48	2.64	3.27	3.51	3.67	3.91	3.67
CS56	487085	610910	1.05	2.48	2.64	3.27	3.51	3.83	3.91
CS56.5	487123	610811	1.45	2.48	2.88	3.59	3.43	3.75	3.59
CS57	487161	610711	1.37	1.84	2.88	3.43	3.35	3.51	3.59
CS57.5	487216	610617	2.00	2.64	3.04	3.27	3.43	3.75	4.07
CS58	487272	610523	2.00	2.32	3.43	3.35	3.59	3.83	3.04
CS58.5	487319	610440	2.72	2.80	2.96	3.12	3.27	3.75	3.67
CS59	487367	610357	2.80	2.64	2.80	3.12	3.83	3.59	3.51

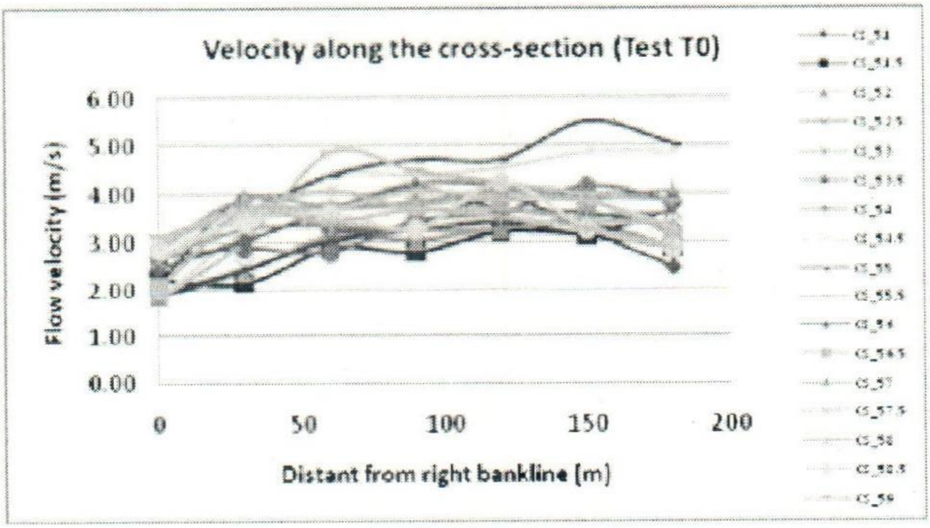

Figure 5: Graphical representation of flow velocities given in Table 3

The equilibrium scour (general) at the bank points along right bankline was measured and can be seen in Table 4. From this table, maximum general scour is found 5.15 m (Point-17) and the corresponding velocity is 2.80 m/s as shown in Table 3.

Table 4: Net scour/deposition at the bank points along right bankline in Test T0

Point No.	Easting (m)	Northing (m)	Scour/ Deposition (m)
1	486716	611997	-5.34
2	486754	611883	-5.46
3	486792	611768	-6.6
4	486805	611675	-3.9
5	486817	611581	-1.68
6	486869	611462	-3.96
7	486921	611344	-6.12
8	486965	611236	-5.94
9	487010	611128	-4.62
10	487047	611019	-2.16
11	487085	610910	-3.42
12	487123	610811	-2.7
13	487161	610711	-3.72
14	487216	610617	-4.60
15	487272	610523	-4.70
16	487319	610440	-5.1
17	487367	610357	-5.15

The location of the bank points for scour measurement can be seen in Figure 6. The flow lines recorded by float tracking have been plotted and shown in Figure 7. From the flow lines, it has been observed that the flow has a tendency of attacking the bankline within the river reach considered in the model. From Test T0, it is observed that the magnitude of bank velocities is higher than the erodible velocity. The bank velocities and flow lines indicate that bank protection work is essential to combat bank erosion.

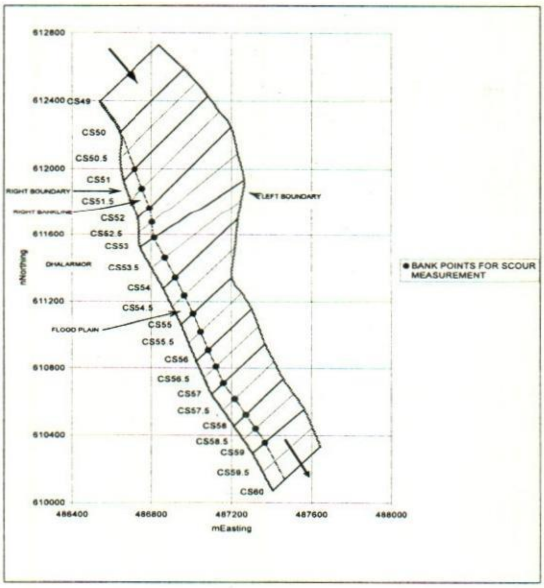

Figure 6: Location of the bank points for scour measurement in Test T0

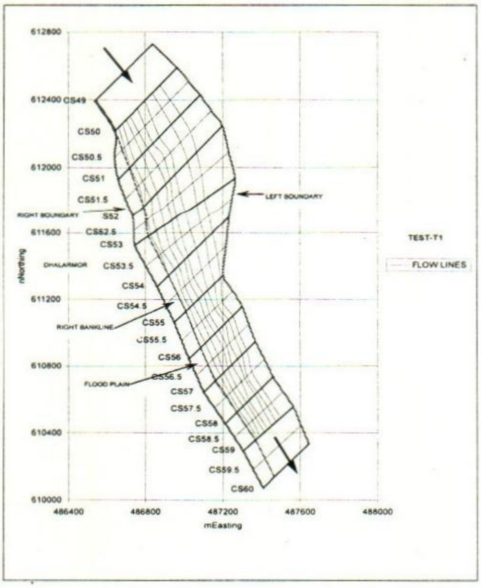

Figure 7: Flow lines in T0

Application Test (T1)

In Test T1, 609 m revetment from Chainage 950 m to 1569 m has been reproduced in the model as per design supplied by BWDB and subjected to normal flow condition. The location & specifications of proposed revetment reproduced in this test is given in Table 5 & 6 respectively.

Table 5: Location of proposed revetment reproduced in Test T1

Structural Intervention	Location by Cross-section		Location by Easting & Northing		Length (along bank)
	Starting	Ending	Starting (Easting, Northing)	Ending (Easting, Northing)	m
Revetment (subjected to normal flow condition)	Between CS (53&54)	CS56	486863, 611477	487085, 610910	609

Table 6: Specifications of proposed revetment reproduced in Test T1

Slope Pitching of Revetment	• Projected length 11 m • One layer 500 mm cube CC block			
	1^{st} Part	2^{nd} Part	3^{rd} Part	4^{th} Part
Launching Apron of Revetment	• Projected length: 5m • Thickness: 1.1m • CC block dumping: 400mm cube-60% & 350mm cube-40%	• Projected length: 13m • Thickness: 1.33m • Geobag dumping: 250kg bag-66Nos/m & 175 kg bag-63Nos/m	• Projected length: 11m • Thickness: 1.67m • Geobag dumping: 250kg bag-66Nos/m & 175 kg bag-63Nos/m	• Projected length: 9m • Thickness: 2.00m • Geobag dumping: 250kg bag-66Nos/m & 175 kg bag-63Nos/m

In Test T1, the velocities measured in the model at the selected points (see Figure 4) along cross-section from right bankline up to 180m are shown in Table 7. From this table, it is observed that the measured bank velocities for different cross-sections are in the range of 0.33 to 2.88 m/s within the reach from CS51 to CS59.

Table 7: Flow velocity (m/s) in Test T1

CS No.	Right bankline		Velocity (m/s)						
	Easting (m)	Northing (m)	Distance from right bankline (m)						
			0	30	60	90	120	150	180
CS51	486716	611997	2.24	2.64	2.80	2.96	3.04	2.72	2.40
CS51.5	486754	611883	2.88	2.48	2.48	2.96	3.27	3.04	2.72
CS52	486792	611768	2.00	2.72	2.40	2.88	2.88	2.88	2.88
CS52.5	486805	611675	1.68	2.64	2.96	3.20	3.27	3.43	3.04
CS53	486817	611581	1.29	2.24	2.40	3.12	3.12	2.88	2.96
CS53.5	486869	611462	2.08	2.64	3.12	3.35	3.12	2.88	3.04
CS54	486921	611344	2.24	3.35	3.27	2.96	3.75	3.51	3.51
CS54.5	486965	611236	1.76	2.48	3.35	4.07	3.99	3.27	3.51
CS55	487010	611128	2.00	2.24	3.27	3.51	3.51	3.83	3.51
CS55.5	487047	611019	1.68	2.88	3.67	3.12	3.59	3.91	4.07
CS56	487085	610910	1.13	2.32	2.96	2.80	2.80	3.27	3.12
CS56.5	487123	610811	0.57	1.52	3.27	3.12	3.20	2.64	2.96
CS57	487161	610711	0.33	2.16	2.80	2.64	2.80	2.96	3.04
CS57.5	487216	610617	2.08	2.40	2.96	2.88	2.88	2.88	3.04
CS58	487272	610523	2.24	2.56	2.80	3.04	3.20	2.72	3.12
CS58.5	487319	610440	2.08	2.96	2.96	3.04	2.96	3.12	2.56
CS59	487367	610357	2.56	3.20	2.96	3.20	3.04	2.64	3.20

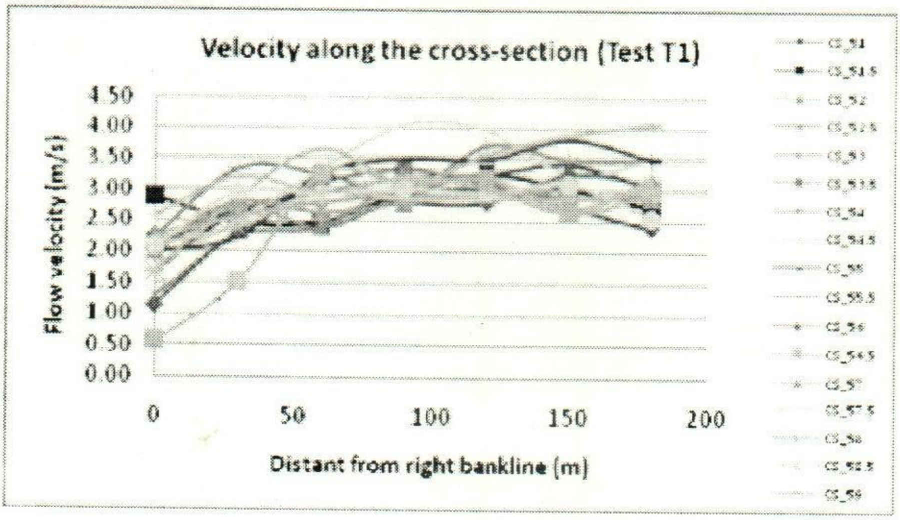

Figure 8: Graphical representation of flow velocities given in Table 7

The flow lines recorded by float tracking in Test T1 have been plotted and shown in Figure 9. From the flow lines, it is observed that the flow lines are parallel and concentrated at the downstream end.

The scour depths and the corresponding velocities have been measured at 24 points which were 49m from the right bankline towards the river side (see Figure 10) starting from upstream along the end of the launching apron and can be seen in Table 8.

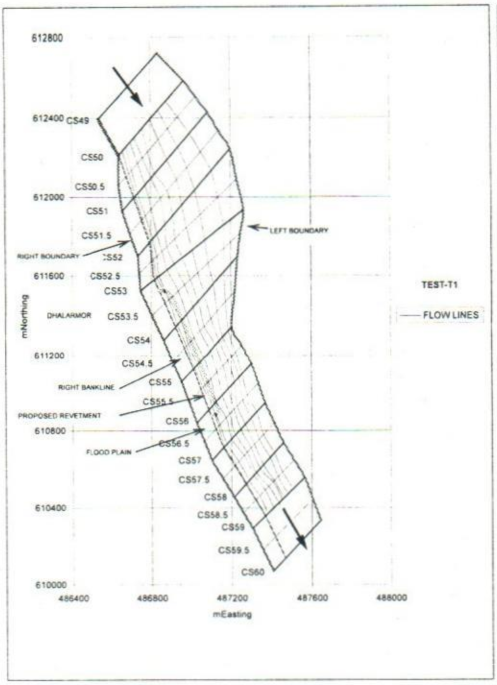

Figure 9: Flow lines in T1

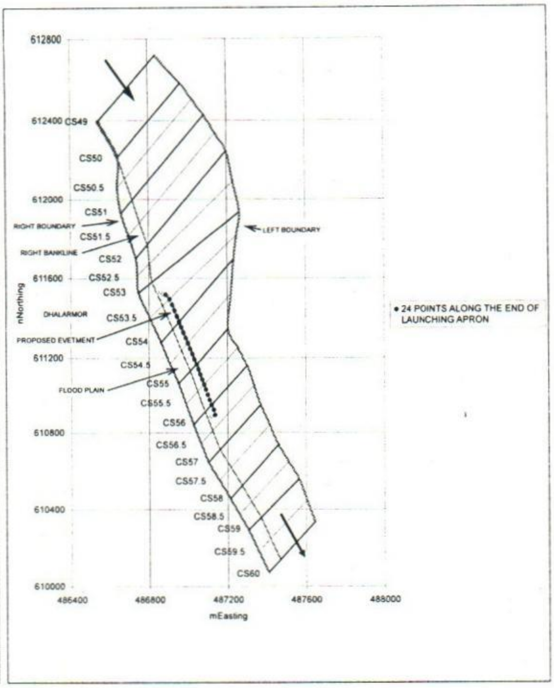

Figure 10: Location of 24 points for scour and velocity measurement along the end of the launching apron for Test T1 & T2

Table 8: Net scour/deposition and the corresponding velocities at 24 points along the end of launching apron in Test T1

Point No.	Easting (m)	Northing (m)	Scour/ Deposition (m)	Flow Velocity (m/s)	Point No.	Easting (m)	Northing (m)	Scour/ Deposition (m)	Flow Velocity (m/s)
1	486886	611518	-1.68	2.16	13	487031	611192	-2.80	3.35
2	486905	611496	-2.00	2.72	14	487042	611165	-2.93	3.12
3	486917	611467	-2.10	2.80	15	487052	611138	-3.15	3.35
4	486929	611438	-1.90	2.64	16	487061	611110	-3.25	3.20
5	486941	611409	-2.27	2.88	17	487070	611083	-3.33	2.88
6	486954	611380	-2.33	3.12	18	487080	611056	-3.40	3.12
7	486965	611353	-2.16	2.80	19	487089	611029	-3.42	2.72
8	486976	611327	-2.00	3.12	20	487098	611001	-4.44	3.04
9	486987	611300	-3.10	3.43	21	487108	610974	-6.42	2.80
10	486998	611273	-2.93	3.35	22	487117	610947	-4.50	2.88
11	487009	611246	-2.81	3.27	23	487129	610921	-3.78	2.96
12	487020	611219	-2.75	3.27	24	487135	610893	-3.84	2.48

From Table 8, maximum scour and the corresponding velocity along the end of launching apron of revetment are found 6.42m and 2.80m/s respectively (Point-21). From this table, maximum velocity along the end of launching apron of revetment is found 3.43m/s at Point 9. The velocities have also been measured at 21 points which were 11m from the right bankline towards the river side (see Figure 11) starting from upstream along the end of revetment slope and can be seen in Table 9. From this table, maximum velocity along the end of revetment slope is found 2.72m/s (Point-10).

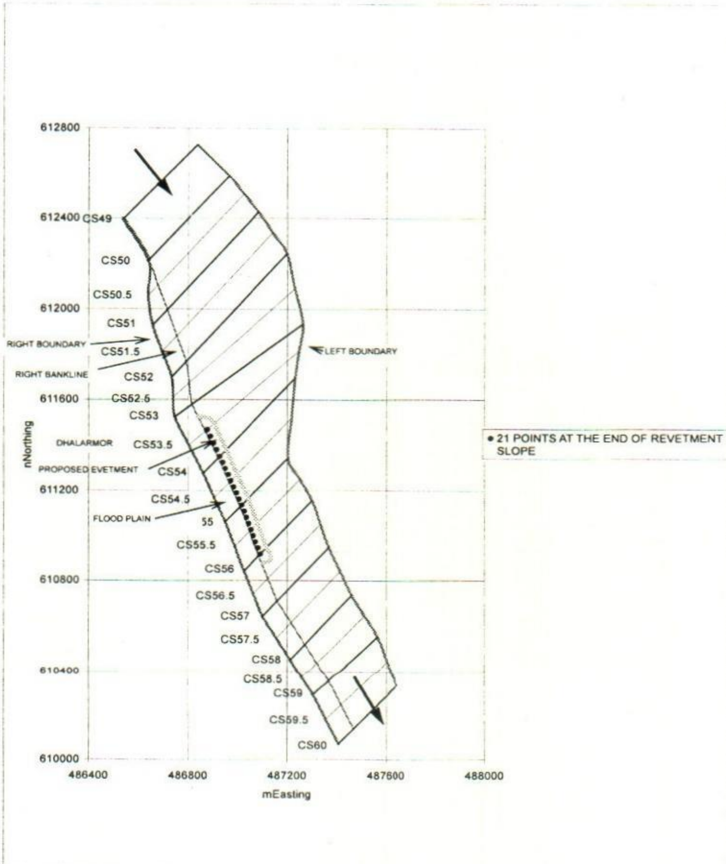

Figure 11: Location of 21 points for velocity measurement along the end of revetment slope for Test T1 & T2

Table 9: The velocity at 21 points along the end of revetment slope in Test T1

Point No.	Eastin g (m)	Northing (m)	Flow Velocity (m/s)	Point No.	Easting (m)	Northing (m)	Flow Velocity (m/s)
1	486877	611470	1.92	12	487006	611164	2.24
2	486890	611440	1.84	13	487017	611137	2.16
3	486903	611411	1.92	14	487026	611109	2.40
4	486916	611381	2.16	15	487036	611082	2.24
5	486929	611352	2.32	16	487045	611055	2.08
6	486940	611325	2.24	17	487055	611027	1.76
7	486951	611298	2.16	18	487064	611000	1.84
8	486962	611271	2.00	19	487073	610973	1.52
9	486973	611244	2.08	20	487083	610946	1.52
10	486984	611217	2.72	21	487092	610918	1.60
11	486995	611190	2.48				

Application Test-2 (T2)

In this test, the revetment with same location & specifications as in Test T1 has also subjected to 60° angle of flow attack. The velocities measured in the model at the selected points (see Figure 4) along cross-section from right bankline up to 180m are shown in Table 10.

Table10: Flow velocity (m/s) in Test T2

CS No.	Right bankline		Velocity (m/s)						
			Distance from right bankline (m)						
	Easting (m)	Northing (m)	0	30	60	90	120	150	180
CS51	486716	611997	2.16	3.91	3.27	3.35	3.91	3.20	2.48
CS51.5	486754	611883	2.24	2.16	2.88	2.80	3.20	3.12	2.88
CS52	486792	611768	1.84	2.40	3.12	3.04	3.27	3.35	2.88
CS52.5	486805	611675	1.92	2.48	3.04	3.27	3.43	3.20	3.67
CS53	486817	611581	2.72	3.27	3.35	3.27	3.27	3.51	3.59
CS53.5	486869	611462	2.72	2.88	2.80	3.43	3.59	4.15	3.83
CS54	486921	611344	3.12	3.67	3.75	4.23	4.15	3.91	4.07
CS54.5	486965	611236	1.84	3.27	4.39	4.39	4.55	4.87	4.87
CS55	487010	611128	2.64	3.59	4.39	4.71	4.71	5.50	5.03
CS55.5	487047	611019	1.84	3.04	4.87	4.47	4.23	3.99	3.83
CS56	487085	610910	2.48	3.04	3.67	4.15	3.67	3.59	3.27
CS56.5	487123	610811	2.08	3.59	3.67	3.75	3.91	3.83	3.12
CS57	487161	610711	1.84	2.88	3.91	3.75	3.83	4.07	3.83
CS57.5	487216	610617	2.64	3.51	3.67	4.07	3.91	3.83	3.27
CS58	487272	610523	2.96	3.67	4.07	3.67	3.67	3.43	3.35
CS58.5	487319	610440	2.88	3.43	3.59	3.27	4.23	3.27	2.88
CS59	487367	610357	3.12	3.67	3.43	3.67	3.51	3.27	2.88

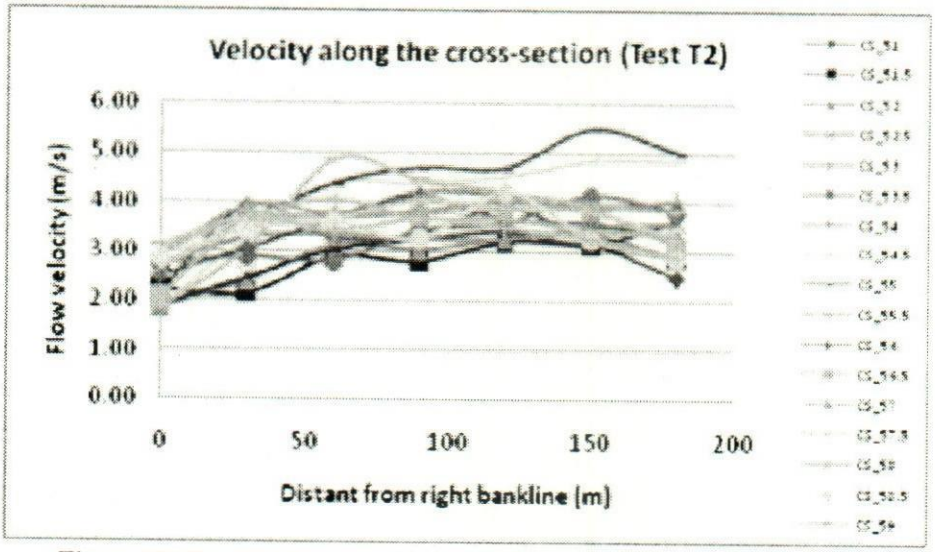

Figure 12: Graphical representation of flow velocities given in Table 10

From Table 10, it is observed that the measured bank velocities are in the range of 1.84 to 3.12 m/s within the reach from CS51 to CS59.

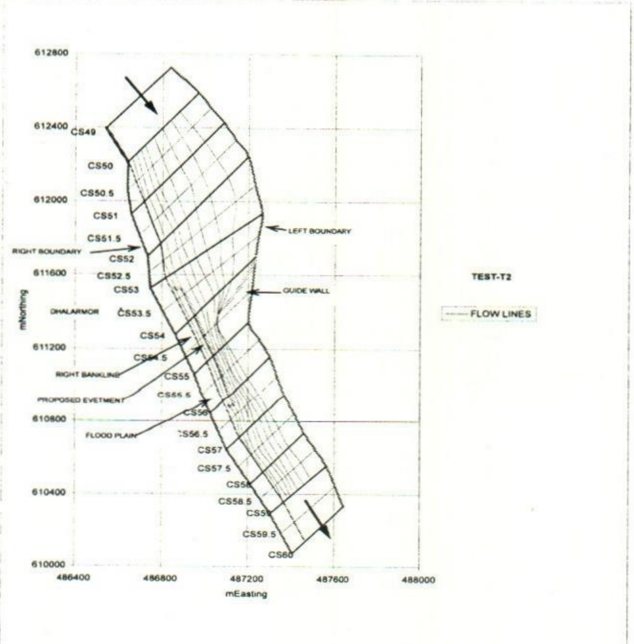

Figure 13: Flow lines in T2

The flow lines recorded by float tracking have been plotted and shown in Figure 13. From the flow lines, it is observed that the flow lines are parallel and more close to the revetment just downstream of the attacking point and concentrated at the downstream end.

In Test T2, the equilibrium scour depths and the corresponding velocities have also been measured at 24 points which were 49m from the right bankline towards the river side (see Figure 10) starting from upstream along the end of the launching apron and can be seen in Table 11.

Table 11: Net scour/deposition and the corresponding velocities at 24 points along the end of launching apron in Test T2

Point No.	Easting (m)	Northing (m)	Scour/ Deposition (m)	Flow Velocity (m/s)	Point No.	Easting (m)	Northing (m)	Scour/ Deposition (m)	Flow Velocity (m/s)
1	486886	611518	-1.75	2.64	13	487031	611192	-7.08	3.67
2	486905	611496	-2.5	3.27	14	487042	611165	-6.72	3.51
3	486917	611467	-1.92	2.96	15	487052	611138	-7.38	3.83
4	486929	611438	-3.36	3.12	16	487061	611110	-7.2	3.83
5	486941	611409	-3.96	3.43	17	487070	611083	-7.68	3.83
6	486954	611380	-5.16	3.83	18	487080	611056	-7.44	3.27
7	486965	611353	-6.6	3.67	19	487089	611029	-6.78	3.67
8	486976	611327	-7.86	3.67	20	487098	611001	-5.58	3.51
9	486987	611300	-8.64	3.75	21	487108	610974	-5.58	3.51
10	486998	611273	-8.58	3.59	22	487117	610947	-5.28	3.91
11	487009	611246	**-9.54**	**3.51**	23	487129	610921	-5.58	**3.99**
12	487020	611219	-9.42	3.59	24	487135	610893	-6.00	3.12

From Table 11, maximum scour and the corresponding velocity along the end of launching apron of revetment with oblique flow condition are found 9.54 m and 3.51 m/s respectively (Point-11). From this table, the maximum velocity along the end of launching apron of revetment with oblique flow condition is found 3.99 m/s at Point 23. The velocities have also been measured at 21 points which were 11 m from the right bankline towards the river side (see Figure 11) starting from upstream along the end of revetment slope and can be seen in Table 12.

Table 12: The velocity at 21 points along the end of revetment slope in Test T2

Point No.	Easting (m)	Northing (m)	Flow Velocity (m/s)	Point No.	Easting (m)	Northing (m)	Flow Velocity (m/s)
1	486877	611470	2.32	12	487006	611164	1.68
2	486890	611440	2.40	13	487017	611137	2.48
3	486903	611411	2.24	14	487026	611109	2.48
4	486916	611381	2.56	15	487036	611082	2.48
5	486929	611352	2.48	16	487045	611055	2.80
6	486940	611325	2.24	17	487055	611027	2.88
7	486951	611298	2.16	18	487064	611000	2.96
8	486962	611271	2.32	19	487073	610973	**3.04**
9	486973	611244	2.16	20	487083	610946	2.64
10	486984	611217	2.32	21	487092	610918	2.72
11	486995	611190	2.16				

From Table 12, maximum velocity along the end of revetment slope with oblique flow (angle of flow attack 60 degree) condition is found 3.04m/s (Point-19).

In this test the revetment has been attacked by the oblique flow. Some geobags of launching apron have been displaced away from the attacking point of revetment and rolled into the river bed. These phenomena have been happened due to high velocity developed for oblique flow attack around the revetment.

Conclusions

The following conclusions are drawn from the model study:

609m revetment is reproduced in the model which is subjected to normal and oblique flow condition. It works effectively for bank protection under normal flow condition. Under oblique flow condition, some geobags have been displaced away from the launching apron from the attacking point of revetment and that's why severe damage has been observed in that part. These happened due to high velocity for oblique flow condition. In that case, geobags should be kept ready for emergency dumping, if necessary.

The maximum developed scour along the end of launching apron of revetment under normal flow condition is found 6.42m and with oblique flow (angle of flow attack 60 degree) condition it is 9.54 m. The velocity corresponding to maximum developed scour along the end of launching apron of revetment for normal flow is found 2.80m/s and with oblique flow condition it is 3.51 m/s.

The maximum developed velocity around the revetment under normal flow condition is found 3.43 m/s and with oblique flow condition it is 3.99 m/s. The maximum developed velocity around end of the slope of revetment under normal flow condition is found 2.72 m/s and with oblique flow condition it is 3.04 m/s.

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RIVER BANK EROSION IMPACT ON SOCIO-ECONOMIC CONDITION: A FIELD STUDY IN BANARIPARA UPAZILLA OF BARISAL DISTRICT

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Abstract

Riverbank erosion is a perennial problem and one of the worst natural calamities in Bangladesh. Banaripara upazilla under Barisal district is a riverbank erosion prone area and it is situated by the Sandha River. This area is so much affected due to river bank erosion. A field survey is carried out in Banaripara Upazila (covering five villages of five unions) with a view to examine the river bank erosion impact on socio-economic conditions in order to fully understand and competently deal with the problem at national scale. A comparison study of different socio-economic issues has been presented in this paper between before and after river bank erosion. The study revealed that the impact of river bank erosion on the socioeconomic condition of the displaced population is multilateral. At the micro level, the erosion increases the percentage of the landless and also contributes to the surplus agricultural labor force. This revolves for severe unemployment among the agricultural labors and further impoverish of the rural population. Study revealed that in case of monthly expenditure pattern, respondent's monthly expenditure has decreased and in case of monthly saving pattern, percentage of no savings has increased from 7% to 37% due to river bank erosion/flood in the study area, which implies that monthly saving is decreasing because of respondent's income generating scope & activities decreased by riverbank erosion. Through this study some valuable information has been generated for the researchers, policy makers, development studies worker and different types of NGOs professionals for taking future plan & action. Some recommendations have been recommended in this paper to overcome the river bank erosion hazard during flood.

Introduction

Banaripara upazilla is a riverbank erosion prone area and riverbank erosion is recurring problem and disaster of this area. Banaripara is situated by the Sandha River in Barisal district. The Sandha River is a braided river with bank materials that are highly susceptible to erosion. The average width of the river fluctuates substantially. The recorded minimum average width of the river is known to be 0.7 km. The maximum average width has often exceeded 1.5 km.

There is a considerable scope for geographical research in this field because of its significant impact on the broader socioeconomic system of the country.

Physical characteristics of the Sandha river and its distributaries are changing due to oscillation of the riverbanks, levee branching and formation of mid-channel bars. River depths became shallower at some places. Many of the off takes were closed due to depositions of huge sediments at their mouths. Every year huge amount of

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land and other different types of assets are lost by riverbank erosion. This research study was undertaken because of through this study it would be helpful to the individual displaced households providing some recommendations to alleviate the sufferings of the affected people. The findings of the research study will be expected to be useful to the policy makers. The study may also generate valuable information for the researchers, development studies worker and different types of NGOs professionals. The specific objective of the study is as following:

- To assess the physical, social and economic impact of riverbank erosion in the study area.
- To measures the extent of human displacement and resources lost caused by riverbank erosion.

Literature Review

Hossain *et al,*. (2004) reported that Bangladesh having an area of 1,44,000 sq km and a population of about 110 million (about 800 persons per square kilometers) is a disaster prone country. Riverbank erosion occurs in Bangladesh regularly and frequently, riverbank erosion takes away thousand of hectares of land every year in a country where land is the scarcest resource.

Haque (1997) mentioned that three common effects of erosion induced displacement which influenced the well-being of family members: (i) a severe cut in their standard of living, (ii) the occurrence of mental illness in the family, and (iii) the loss of valuable assets.

Ahmed (1995) emphasized the importance of kinship during erosion disasters. The author conducted the anthropological study on the survivors of riverbank erosion and found that kinship, especially patriarchal; bonds were very strong in Bangladesh. In most of the cases the whole patria lineage became affected by riverbank erosion due to their proximity.

Methodology

Selection of the Study Area

A research on riverbank erosion topics requires the selection of an area where the objectives of the study can be fulfilled. Keeping in mind five unions namely Banaripara, Saliabakpur, Chakhar, Saidkhati, Baisari of Banaripara upzila under Barisal district were selected as the study area. This locality covers a large area, which is previously and presently affected by severe riverbank erosion. However, the selection was made on the basis of the following specific characteristics:

- Availability of riverbank erosion affected people.
- The researcher was familiar with the study area.
- This area was suitable for the study considering the time and resources available

Data Sampling Techniques, Sample Size and Data Collection Procedure

A total of 60 respondents (inhabitants who are affected due to river bank erosion) were selected in the study area. The primary data were collected through one set of interview schedule from the respondents of the selected area in September to November 2009. In this study, a detailed look was taken to the inhabitants based upon a questionnaire survey so that the general characteristics of the affected population, their perception regarding erosion hazards damages and related facts are meaningfully understood. Particularly different socio-economic issues and fact have been collected in this study. The secondary data were collected from UNO office, Banaripara Upazila

Data Analysis, Result and Discussion

Location, Area and population

The study area, Banaripara Upazila is located under Barisal district which is shown in Figure 1. This district is bounded on the north by Madaripur district, on the south Jhalokathi district, on the east by Bhola and Laxmipur district and on the west by Pirojpur and Gopalgong district. The total area of Banaripara upazila is 13400 Ha. The total population of the upazila is 1,63,041. The male is 81,945 and female is 81,096 (UNO office, Banaripara upazila). The population growth rate in the study area is 1 .66%.

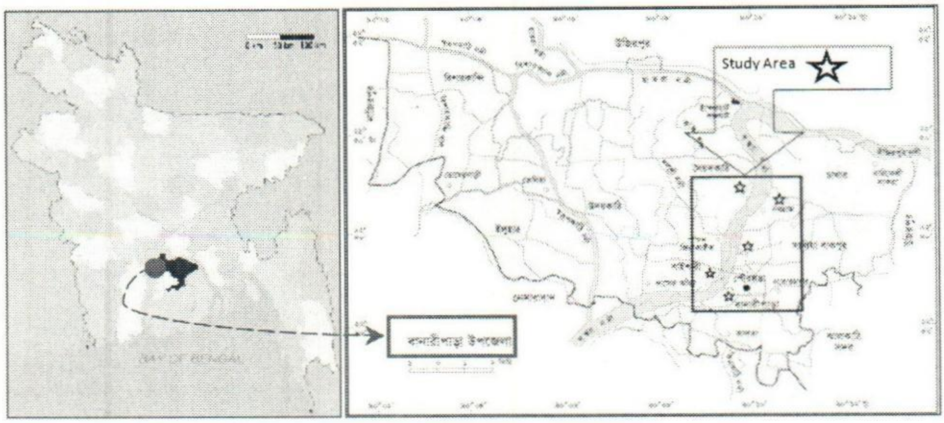

Figure 1: Location of the study area

Soil Type and Land Topography

The soil of the study area formed in recent and sub-recent alluvium sediments. Most of the soil is seasonally flooded, have grey floodplain soil and silty to clayey textures and moderate to low contents of organic matter. The study area is under the AEZ-12, AEZ-13 and AEZ-14, which are Low Ganges River Floodplain, the Ganges Tidal Floodplain and Gopalganj-Khulna Beel. The study area is strongly riverbank erosion prone area.

Socio Economic Condition of the Affected People

A Socio economic aspect of the respondent is relevant to understand more clearly from their perception and process of adjustment with erosion hazard. But the socio-economic conditions of the displaced population are not all alike the people of rest of the Bangladesh and therefore it need to understand within their own context.

Family Size of the Respondents

On the basis of the family size of the respondents were classified into three categories as shown in Table 1.

Table 1. Distribution of the respondents according to their family size

Types of Family	Frequency of Respondents	Percent (%)	Average
Small (Up to 4)	18	30	
Medium (5 to 6)	29	48.33	4.38
Large (More than 6)	13	21.67	
Total	60	100	

★ Types of family according to BBS (2007) classification

Types of Dwelling Household of the Respondent

On the basis of types of dwelling household, the respondents were classified into four categories. Results showed that 1) 31% respondents had "Kacha" house, which was made with bamboo & straw roof, 2) 24% respondents had house made with bamboo & Tin roof, 3) 18% respondents had house made with Tin & straw roof and 4) 27% respondents had house made with Tin & Tin roof. The respondents of the study area raised earth platform by cutting soil from neighboring area of the household and plant trees to protect their houses from seasonal flood and bank erosion.

Location of Displaced Respondents

Displacement is mainly confined within the village of same union. Intra union displacement was frequent. On the basis of origin of migration, the respondents were classified into three categories as shown in Figure 2.

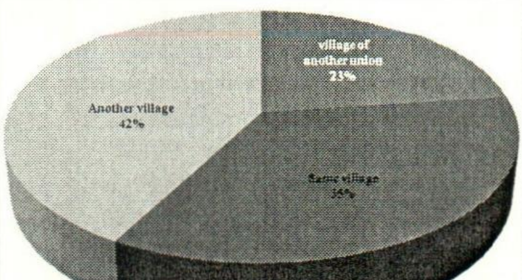

Figure 2: Location of displaced

Migration Pattern of Displaced Respondents

According to the pattern of migration, the respondents were categorized into two categories. One is temporarily migrated and other is permanently migrated. Majority (59%) of the respondents migrated temporarily in the study area due to river bank erosion where as 41 % of the respondents migrated permanently. The temporarily displaced respondents live a more vulnerable life than that of the permanently displaced respondents.

Homestead Shifting

Frequency of homestead shifting of the respondent is shown as Figure 3. Most of the respondents (28.34%) shifted their homestead 1 time, 21.67% respondents shifted their homestead 5 times, 15% of the respondents shifted their homestead 6 times, 15% of the respondents shifted their homestead 2 times, 6.67% of the respondents shifted their homestead 3 times and 3.33% of the respondents shifted their homestead 4 times.

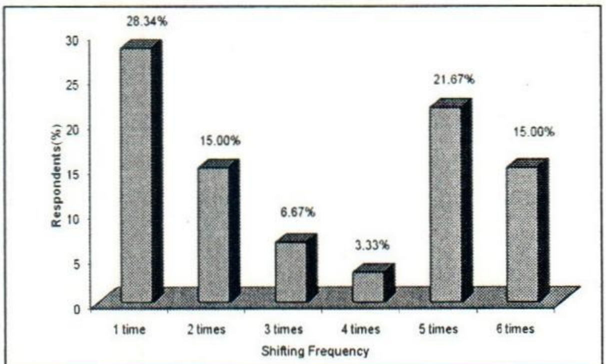

Figure 3: Frequency of homestead shifting of the respondents

Duration of Displacement

Every year a significant number of people change their homestead due to riverbank erosion. On the basis of years of migration, the respondents were classified into four categories as shown in Figure 4. Majority (41%) of the respondents shifted their homestead in the study area more than 10 years ago, 24% of the respondents shifted their homestead 1 to 5 years ago and 35% of the respondents shifted their homestead 6 to 10 years ago in the study area.

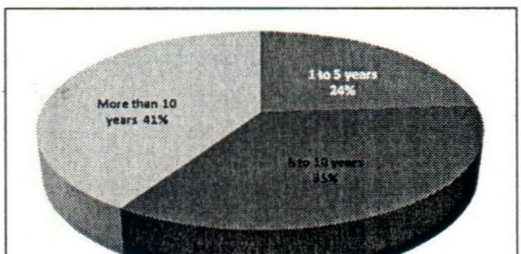

Figure 4: Duration of displacement of the respondents!

Sources of Drinking Water of the Respondents

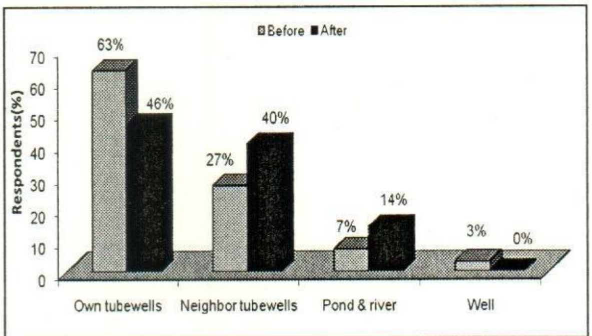

Figure 5: Sources of drinking water of the respondents

According to the survey, the majority of the respondents used ground water from hand tube wells and they had access to clean drinking water was high (90%) before riverbank erosion. The households installed these tube wells privately, but some of the homesteads installed tube-wells with the support from NGOs and union parishads. On the basis of the sources of drinking water, the respondents were classified into four categories as shown in Figure 5. Before riverbank erosion 90% respondents used tube wells water as the source of drinking water where 63% used own tube wells and 27% used neighbor tube wells. But after riverbank erosion, rate of using of tube wells water has decreased at 86% where 46% used own tube wells and 40% used neighbor tube wells water as their source of drinking water. Before erosion 07% used pond and river water as their source of drinking water and only 3% used water from well. After erosion 14% of the respondents used pond & river water as their source of drinking water. Data revealed that during flood period using of tube well water decreases and most of the respondents become dependent on Pond & River water as a source of drinking water.

Sanitation Status

Sanitation status in the study area tends to be unhygienic with existing latrines poorly maintained. On the basis of toilet facility, the respondents were classified into three categories. Majority of the respondents (85%) didn't use sanitary latrine. 1) About half of the respondents (48%) used "Kacha" latrine which was larger than that of the national rural rate 39.2% (BBS, 2007), 2) 37% practiced open defecation and 3) Only 15% respondents used sanitary latrine in the study area.

Farming Size/ Farming Status

The different size of farm and tenure arrangements was found in the study area. The farm size was measured according to the following formula:

Farm size= (Own + rented in + mortgaged in) — rented out - mortgaged out

On the basis of farm size, the respondents were classified into four categories as shown in Table 2. Data revealed that the socio-economic conditions of the respondents in the study area were worst.

Table 2. Distribution of the respondents according to their farm size

Farm size	Number of respondents	Percent (%)
Marginal (0 to 0.49 acre)	33	55
Small (0.50 to 2.49 acre)	21	35
Medium(2.50 to 7.49 acre)	06	15
Large (> 7.49 acre)	00	00
Total	60	100

Cropping Intensity of the Study Area

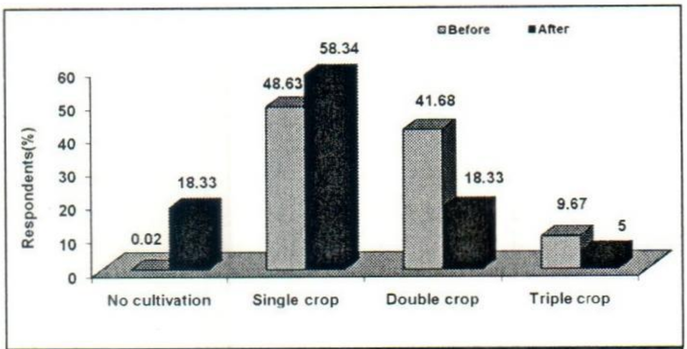

Figure 6: Cropping intensity of the respondents in the study area

On the basis of cropping intensity, the respondents were classified into four categories as shown in Figure 6. Result revealed that after river bank erosion/flood 18.35% respondents had no cultivated land, 58.34% respondents had cultivated land only one crop per year, 18.33% had cultivated land only two crops and only 5% had cultivated land three crops per year. But before erosion these scenario were different. Cultivation of Double crop and Triple crop per year has been reduced due to erosion/flood in the study area. The multiple cropping patterns showed that people living in hazardous areas, especially in the char land tend to reduce their loss (Richards, 1994).

Land Cultivation Pattern of the Respondents

On the basis of the land cultivation pattern, the respondents were classified into three categories as shown in Table 3.

Table 3. Distribution of the respondents according to their land cultivation pattern

Types of land cultivation	Number of respondents	Percent (%)
Own land	13	21.67
Share cropper	30	50
No cultivation	17	28.33
Total	60	100

Ownership of New Char Land

The riverbank erosion has resulted in losing farmers cultivable land while only few of them could acquire re-emerged land. On the basis of ownership of newly emerge land, the respondents were classified into two categories such as 1) Only 43% of the riverbank erosion affected family got back their land after the riverbank erosion and 2) 57% of the riverbank erosion affected family did not get back their land.

Changes in Employment Status

On the basis of changes in sources of income, the respondents were classified into four categories as shown in Figure 7. Before riverbank erosion 53% of the respondents were involved in agricultural activities but after affecting by riverbank erosion it has decreased at 26%. It also reveals that riverbank erosion increases the percent of day labors and unemployed persons in the study area. After bank erosion, 46% of the respondents were day labors while it was only 13% before the bank erosion and 05% respondents were unemployed while it was only 03% before the riverbank erosion. Before the bank erosion 31% of the respondents were related with non-agricultural activities but after the erosion hazard it decreased and then it stood for only 23%. Data revealed that riverbank erosion is decreasing the employment status of the affected people.

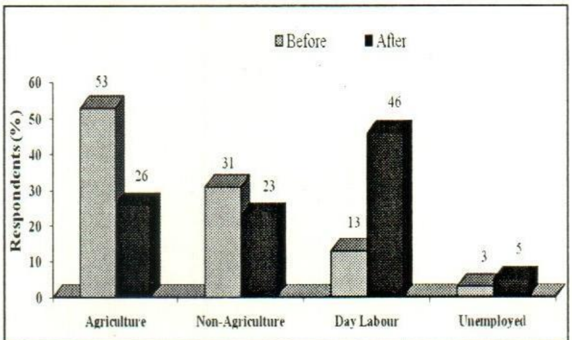

Figure 7: Changes in source of income of the respondents!

Credit Ability of the Respondents

On the basis of the credit ability the respondents were classified into five categories as shown in Table 4. From the table it is observed that riverbank erosion increases the borrowing rate from NGO's and Mahajons and decreases the earning capacity.

Table 4. Credit ability of the respondents in the study area

Source of credit	Number of respondents		Percent (%)	
	Before	After	Before	After
Government Banks	6	2	10	3.33
NGOs	26	32	43.33	53.33
Govt. bank and NGOs	4	3	6.67	5
Friends/ Relatives	17	14	28.33	23.33
Mahajans/ Businessmen	7	9	11.67	15
Total	**60**	**60**	**100**	**100**

Changes in Monthly Expenditure Patterns

On the basis of the monthly expenditure pattern, the respondents were classified into six categories as shown in the Figure 8. Before riverbank erosion 33.33% respondent's monthly expenditure was Tk.2500 and 20% respondent's expenditure of Tk.3000. About 6.67% and 3.33% of the respondents had monthly expenditure Tk.3500 and Tk.4000 respectively. Only 15% had monthly expenditure about Tk.1500. But this expenditure pattern has been changed after riverbank erosion. After riverbank erosion, 46.67% respondents' monthly expenditure was 1500 Taka. About 1.67% and 1.67% of the respondents had monthly expenditure Tk.3500 and Tk.4000 respectively. About 18.33% and 18.33% of the respondents had monthly expenditure of Tk.2000 and Tk.2500 respectively. Only 13.33% respondents' monthly expenditure was 3000 Taka. From the data it is observed that respondent's monthly expenditure was decreasing due to riverbank erosion in the study area. Since their income has decreased, they can't afford enough money to purchase food that leads to ill health, malnutrition and suffer from different diseases.

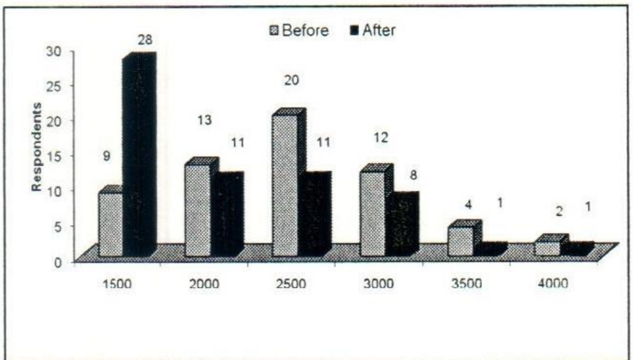

Figure 8: Monthly expenditure pattern (Taka) of the respondents

Changes in Monthly Saving Patterns

On the basis of monthly saving patterns, the respondents were classified into five categories. Before riverbanks erosion most of the respondents (93%) were involved in savings activities, while 1) 13% respondents saved Tk.200 to 400, 2) about 30% respondents saved between Tk.500 to 700, 3) about 39% respondents saved of Tk.800 to1000, 4) 11% saved more than Tk.1000. But after riverbank erosion monthly savings of the respondents has decreased to 63%, where 25% saved Tk.200 to 400, 17% saved Tk.500 to 700, and 15% saved Tk.800 to 1000 and only 06% respondents saved more than Tk. 1000. 5) Before riverbank erosion only 07% of the respondents had no savings but after riverbank erosion this amount has increased at 37%. It implies that monthly saving amount is decreasing because of respondent's income generating activities decreased by riverbank erosion.

Changes in Household Assets

Every year huge amount of land and household assets are lost due to the riverbank erosion. For riverbank erosion all of the respondents lost their household assets as shown in Figure 9. It depends on the number of homestead shifting due to riverbank erosion. Due to riverbank erosion, 46% of the respondents lost their household asset in term of monetary value Tk.1000 to 5000. It is majority respondents because most of the respondents were poor in the study area. About 29% respondents lost their homestead assets in term of monetary value Tk.6000 to 10000. About 14% lost their asset in term of monetary value Tk. 11000 to 15000 and 8% lost their assets in term of monetary value Tk. 16000 to 20000. Only 3% of the respondents lost their homestead assets Tk. 21000 to 25000 in term of monetary value. Data revealed that middle class family is losing, but lower class family is gaining during flood time.

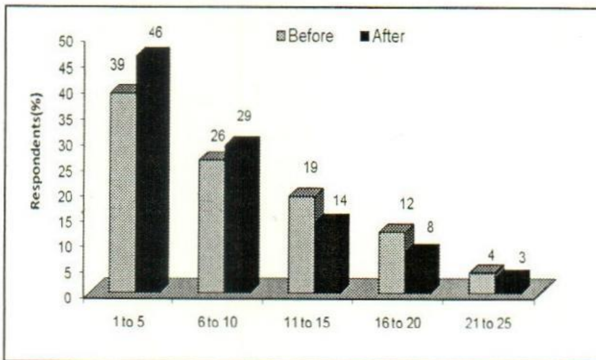

Figure 9: Changes in household assets in terms ('000' Tk.)

Relief Receive

On the basis of received help, the respondents were classified into four categories. 1) Most of the respondents (63%) received monetary help, 2) 10% respondents received infrastructure help, 3) 6% received both monetary & infrastructure help and 4) 21% of the respondents received no help during and after the erosion hazard.

Conclusions and Recommendations

Conclusions

Displacement is the immediate impact of riverbank erosion. The displaced usually move to nearby areas but displacements to distant places are not uncommon. It is observed from the data that the impact of river bank erosion on the socioeconomic condition of the displaced population is multilateral. For example, in case of monthly expenditure pattern it is observed that respondent's monthly expenditure was decreasing due to riverbank erosion in the study area. Since their income has decreased, they can't afford enough money to purchase food that leads to ill health, malnutrition and suffer from different diseases. In case of monthly saving pattern, monthly savings is decreasing because of respondent's income generating activities decreased by riverbank erosion.

In one side it is eliminating the homesteads and infrastructure, damage of crops and animal which causing poverty increasing. On the other hand, Endemic River erosion causing deprived the erosion affected families from doing their normal job to survive at least for a certain period of time. At the micro level, the erosion increases the percentage of the landless and also contributes to the surplus agricultural labor force. This revolves for severe unemployment among the agricultural labors and further impoverish of the rural population. River erosion affected people also tend to have experienced changes in their life style with intensity and number of incidence of erosion events.

Recommendations

Riverbank erosion is a threat for environmental and ecological equilibrium. Air, water pollution may be recovered by spending a lot of money, but if land is demolished by riverbank erosion, it will never be recovered at any cost. So, Government should take decision and necessary action for stopping riverbank erosion immediately to secure ecological balance and sound environment.

The opportunities and possibilities of checking of riverbank erosion are completely limited, so efforts should be made to identify erosion prone area and then available measures should be made to mitigate it such as afforestation, plantation may be undertaken on or around the river bank and embankment in different phases. Programs should be taken to create awareness among the people how to protect themselves from erosion and proper planning should be taken by government to protect riverbank erosion. Improved research/modeling should be conducted for the protection of the riverbank from the erosion before implementation of any measures by the Government for the sustainable development.

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MODELLING OF CLUSTERS USING ARTIFICIAL NEURAL NETWORK

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Abstract

For grouping 92 catchments (of Java and Sumatra in Indonesis) in to clusters or homogeneous regions having common features, 6 catchment characteristics are selected to train the KNN (Kohonen Neural Network) and three clusters have been identified. The results of perceptron modelling on individual clusters (homogeneous regions) identified within the combined data set of Java and Sumatra are presented in this paper. The sigmoidal activation function with one hidden layer is used. The suitable number of nodes in the hidden layer and optimum number of epochs are determined using the vary-a-parameter option in the NeuroSolutions software. It is found that cluster-A & C are homogeneous whereas cluster-B is non-homogeneous. It appears that the variables responsible for clustering are insufficient descriptors of the flow regimes in case of some catchments of cluster-B. Replacing the estimates from the cluster-B ANN (Artificial Neural Network) by those from the unclustered 6-variable ANN has virtually no effect on the RMSE (root mean square error). Nevertheless, if the unclustered 11-variable ANN is employed for the cluster-B catchments, the RMSE is reduced to 160.3 m^3/s, confirming the viability of the clustering technique.

Introduction

The data upon which this study was based were obtained from the Flood Design Manual for Java and Sumatra (FDMJS), 1983 in Indonesia. 12 catchment characteristics for the islands of Java and Sumatra were extracted in this study. These were *(i)AREA* (catchment area) *(ii) MSL* (main stream length*) (iii) S1085 (*main stream slope, as slope over the distance between 10% and 85% of the main stream length*) (iv) SIMS* (simple river slope) *(v) AAR (*average annual rainfall*) (vi) APBAR* (mean annual maximum catchment 1-day rainfall) *(vii) FOREST* (forest index, is the ratio of total area of forest to the catchment area) *(viii) PADDY* (paddy index, is the ratio of total area of paddy to the catchment area) *(ix) SWAMP* (swamp index, is the ratio of total area of swamp to the catchment area) *(x) PLTN* (plantation index, is the ratio of total area of plantation (km^2) to the catchment area) *(xi) SHAPE* (shape index, is the ratio of catchment area to the square of the main stream length) and *(xii) LAKE (*lake index, is the ratio of total catchment area upstream of lakes to the catchment area*)*. 6 catchment characteristics *e.g., AREA, AAR, MSL, S1085, PLTN* & *LAKE* were selected for training the Kohonen neural network out of the 12 catchment characteristics representing 92 catchments of Java and Sumatra. It is noted that out of the 12 catchments characteristics, 8 were identified as important variables in the stepwise regression procedure. These are *AREA, MSL, S1085, SIMS, AAR, PLTN, SHAPE* and *LAKE* (Ashrafuzzaman, 2000).

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The Kohonen neural network was trained using these 6 catchment characteristics (*AREA, AAR, MSL, S1085, PLTN & LAKE*) in order to group 92 catchments into clusters or homogeneous regions having common features. The identified clusters were cluster-A, B & C having memberships of 30, 39 and 23 respectively. The distinguishing catchment characteristics for the 3 significant dense clusters are presented in Tables 1, 2 and 3. The bold number indicates the catchments of Java and the rest are the catchments of Sumatra.

Table 1: Distinguishing catchment characteristics for cluster-A

Serial No.	Catchment No.	*AREA*	*AAR*	*MSL*	*S1085*	*1+PLTN*	*1+LAKE*
1	**4**	**126.300**	**4950.000**	**12.500**	**22.400**	**1.000**	**1.000**
2	**6**	**217.600**	**3482.000**	**52.200**	**1.250**	**1.000**	**1.000**
3	**7**	**139.200**	**3631.000**	**56.600**	**22.400**	**1.000**	**1.000**
4	**8**	**49.200**	**2709.000**	**10.000**	**55.300**	**1.000**	**1.000**
5	**9**	**75.600**	**2679.000**	**11.500**	**43.700**	**1.000**	**1.000**
6	**15**	**179.900**	**3266.000**	**32.200**	**18.700**	**1.000**	**1.000**
7	**17**	**634.700**	**3415.000**	**54.100**	**6.760**	**1.000**	**1.000**
8	**18**	**178.500**	**3389.000**	**30.800**	**17.800**	**1.000**	**1.000**
9	**19**	**320.300**	**3279.000**	**48.600**	**8.560**	**1.000**	**1.000**
10	**20**	**300.000**	**3364.000**	**68.400**	**13.500**	**1.000**	**1.000**
11	**23**	**34.600**	**3327.000**	**11.600**	**14.500**	**1.000**	**1.000**
12	**24**	**108.500**	**3261.000**	**25.000**	**9.740**	**1.000**	**1.000**
13	**25**	**79.300**	**4086.000**	**26.200**	**21.000**	**1.000**	**1.000**
14	**27**	**0.430**	**3650.000**	**0.400**	**87.000**	**1.000**	**1.000**
15	**28**	**3.050**	**3490.000**	**3.200**	**100.000**	**1.000**	**1.000**
16	**29**	**10.810**	**3483.000**	**5.500**	**111.000**	**1.000**	**1.000**
17	**30**	**4.720**	**3517.000**	**3.600**	**111.000**	**1.000**	**1.000**
18	**31**	**2.430**	**3480.000**	**1.300**	**123.000**	**1.000**	**1.000**
19	**32**	**212.400**	**4079.000**	**36.300**	**15.200**	**1.000**	**1.000**
20	**33**	**2642.400**	**4050.000**	**132.400**	**8.230**	**1.000**	**1.000**
21	**43**	**56.400**	**3424.000**	**102.900**	**85.500**	**1.000**	**1.000**
22	**49**	**187.400**	**3250.000**	**32.100**	**55.000**	**1.000**	**1.000**
23	**50**	**152.000**	**2469.000**	**27.900**	**64.200**	**1.000**	**1.000**
24	313	209.300	3040.000	21.000	63.500	1.020	1.000
25	314	121.600	3860.000	20.200	45.000	1.000	1.000
26	316	110.000	3450.000	23.700	53.400	1.000	1.000
27	341	304.900	4050.000	31.400	31.200	1.019	1.000
28	342	1267.000	3440.000	67.400	12.500	1.023	1.000
29	343	304.000	3100.000	54.600	32.600	1.044	1.000
30	701	649.700	3210.000	50.400	15.200	1.077	1.000

Table 2: Distinguishing catchment characteristics for cluster-B

Serial No.	Catchment No.	*AREA*	*AAR*	*MSL*	*S1085*	*1+PLTN*	*1+LAKE*
1	**1**	**1832.600**	**2305.000**	**65.400**	**4.810**	**1.000**	**1.039**
2	**10**	**757.400**	**2560.000**	**59.400**	**13.200**	**1.000**	**1.000**
3	**11**	**474.900**	**2715.000**	**35.500**	**19.900**	**1.000**	**1.000**
4	**12**	**1996.000**	**2631.000**	**130.400**	**6.820**	**1.000**	**1.000**
5	**13**	**1514.800**	**2559.000**	**114.600**	**7.620**	**1.000**	**1.000**
6	**16**	**622.100**	**2669.000**	**81.700**	**7.990**	**1.000**	**1.038**
7	**21**	**622.000**	**2970.000**	**91.100**	**7.060**	**1.000**	**1.000**
8	**22**	**1858.000**	**3120.000**	**111.300**	**1.830**	**1.000**	**1.000**
9	**26**	**495.100**	**2988.000**	**46.300**	**10.400**	**1.000**	**1.000**
10	**35**	**1749.400**	**2985.000**	**86.000**	**10.500**	**1.000**	**1.000**
11	**36**	**417.200**	**2270.000**	**34.600**	**33.200**	**1.000**	**1.000**
12	**38**	**101.900**	**2295.000**	**16.100**	**30.200**	**1.000**	**1.000**
13	**39**	**141.500**	**2423.000**	**37.200**	**6.320**	**1.000**	**1.000**
14	**40**	**90.000**	**2034.000**	**18.600**	**40.200**	**1.000**	**1.000**
15	**44**	**1442.000**	**2122.000**	**54.100**	**1.600**	**1.000**	**1.031**
16	**52**	**772.200**	**1990.000**	**38.300**	**25.500**	**1.000**	**1.000**
17	202	215.000	2600.000	37.700	16.200	1.000	1.000
18	205	979.000	2650.000	76.300	19.200	1.107	1.000
19	216	836.900	2120.000	60.800	19.500	1.000	1.000
20	218	820.000	2290.000	68.400	9.870	1.139	1.000
21	219	629.700	2970.000	60.000	8.400	1.041	1.000
22	243	180.800	1950.000	45.000	24.600	1.000	1.000
23	246	727.700	2040.000	53.500	8.550	1.001	1.000
24	331	436.900	2850.000	45.300	15.700	1.053	1.000
25	422	906.100	2790.000	61.500	14.600	1.012	1.000

Serial No.	Catchment No.	*AREA*	*AAR*	*MSL*	*S1085*	*1+PLTN*	*1+LAKE*
26	431	346.400	2340.000	44.000	10.700	1.006	1.000
27	512	343.700	3220.000	77.000	14.800	1.022	1.000
28	515	923.000	3140.000	97.600	11.500	1.056	1.000
29	522	1505.300	3220.000	106.600	11.300	1.175	1.000
30	709	2046.000	2920.000	105.200	7.530	1.056	1.000
31	712	567.000	2900.000	81.400	0.800	1.134	1.000
32	801	432.100	2450.000	53.400	15.700	1.101	1.000
33	817	416.000	2460.000	49.200	2.940	1.125	1.000
34	818	661.600	2430.000	99.200	10.300	1.159	1.000
35	823	526.700	2490.000	92.300	12.800	1.028	1.000
36	824	555.600	2430.000	77.000	11.800	1.238	1.000
37	825	111.300	2420.000	44.000	20.100	1.040	1.000
38	827	528.500	2375.000	59.000	0.310	1.185	1.000
39	839	205.200	2450.000	30.800	27.600	1.024	1.000
1	**1**	**1832.600**	**2305.000**	**65.400**	**4.810**	**1.000**	**1.039**
2	**10**	**757.400**	**2560.000**	**59.400**	**13.200**	**1.000**	**1.000**

Table 3: Distinguishing catchment characteristics for cluster-C

Serial No.	Catchment No.	*AREA*	*AAR*	*MSL*	*S1085*	*1+PLTN*	*1+LAKE*
1	**2**	**2367.000**	**2256.000**	**90.600**	**2.260**	**1.000**	**1.030**
2	**3**	**4232.000**	**2479.000**	**132.200**	**5.730**	**1.000**	**1.017**
3	**41**	**1968.100**	**2034.000**	**128.000**	**0.400**	**1.000**	**1.032**
4	**42**	**9578.000**	**2245.000**	**213.500**	**0.460**	**1.000**	**1.008**
5	**45**	**5900.000**	**2342.000**	**201.400**	**0.460**	**1.000**	**1.009**
6	**46**	**12429.000**	**2189.000**	**331.900**	**0.330**	**1.000**	**1.006**
7	**47**	**8750.100**	**2167.000**	**228.000**	**2.370**	**1.000**	**1.357**
8	**48**	**9972.900**	**2176.000**	**255.200**	**1.840**	**1.000**	**1.313**
9	**51**	**6902.000**	**2231.000**	**202.400**	**2.570**	**1.000**	**1.453**
10	103	4494.300	2700.000	173.000	4.170	1.053	1.000
11	118	4402.700	2450.000	231.200	2.880	1.072	1.000
12	201	3788.900	2770.000	178.100	8.430	1.054	1.003
13	206	917.600	2750.000	78.900	10.600	1.563	1.000
14	208	679.500	2770.000	93.400	13.500	1.622	1.000
15	209	1022.500	2710.000	105.600	9.980	1.255	1.000
16	413	3128.700	2580.000	159.100	2.350	1.039	1.000
17	511	4578.60	3290.00	150.10	7.20	1.006	1.009
18	521	*1256.000*	2400.000	89.600	10.500	1.137	1.788
19	707	*4463.600*	2730.000	166.600	3.630	1.102	1.112
20	803	*1697.900*	2440.000	90.300	8.990	1.480	1.000
21	807	*786.000*	2450.000	52.400	8.360	1.519	1.000
22	812	*2102.000*	2400.000	122.700	5.620	1.483	1.000
23	834	*903.000*	2420.000	71.200	7.800	1.248	1.000

MIP Modelling Using EVI Parameters as Desired Outputs

The inputs to the ANN were catchment characteristics and the outputs were the scale and location parameters (α,β) of the EVI (Extreme Value Type I) distribution fitted to the annual floods recorded at each gauged site.

Initially, a choice had to be made regarding the suitability of applying a single ANN with inputs and two outputs or of training separately two ANNs with a single output for α and β respectively. An ANN with only one output neuron would, of course, contain fewer weights than an ANN with two output neurons, giving rise to a simpler network that would be correspondingly faster and easier to train. However, α and β parameters forming the outputs are not entirely independent. Use of two separate ANNs would therefore ignore this connection, but by combining the outputs in a single ANN this 'correlation' would be 'learned' implicitly during the training of the network (Hall & Minns, 1998).

An initial simulation was made using all the 12-variables (*AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PADDY, SWAMP, PLTN, SHAPE* and *LAKE*) as inputs to the ANN. Since *SHAPE* was derived from its parent variables, *AREA* and *MSL*, the next simulation was performed using 11-variables (excluding *SHAPE*). More simulations were made by dropping one variable each time on the basis of weights. The correlation coefficient (*r*), efficiency (one minus the quotient of the mean square error and the variance of the observed data) as well as the root mean square error (RMSE) between observed and model estimated values of EVI parameters were calculated for each simulation. The simulation results for both training and cross validation data set are presented in Table 2.

Table 2: Simulation results showing efficiency, RMSE and correlation coefficient for a 3-layer ANN

Simulation No.	Included variables	Training data set Alpha / beta shown separately			Cross validation data set Alpha / beta shown separately		
		Efficiency	RMSE	*R*	Efficiency	RMSE	*r*
1 (12 variables)	*AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PADDY, SWAMP, PLTN, SHAPE & LAKE*	0.79 0.92	53.89 110.34	0.8886 0.9583	0.23 0.70	81.18 205.1	0.5862 0.8318
2 (11 variables)	*AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PADDY, SWAMP, PLTN & LAKE*	0.95 0.97	26.58 70.68	0.9740 0.9831	0.43 0.83	70.10 156.67	0.6645 0.9054
3 (10 variables)	*AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, SWAMP, PLTN & LAKE*	0.82 0.94	50.0 92.95	0.9054 0.9706	0.26 0.77	79.56 178.68	0.6723 0.8742
4 (9 variables)	*AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PLTN & LAKE*	0.773 0.93	56.19 100.58	0.8782 0.9656	0.24 0.78	80.73 174.98	0.6698 0.8877
5 (8 variables)	*AREA, MSL, S1085, SIMS, AAR, APBAR, PLTN & LAKE*	0.81 0.94	50.80 93.01	0.9104 0.9760	0.19 0.8	83.23 165.64	0.6127 0.9097
6 (7 variables)	*AREA, MSL, S1085, SIMS, AAR, PLTN & LAKE*	0.75 0.93	58.83 100.15	0.8655 0.9662	0.04 0.72	90.37 197.23	0.5178 0.8553
7 (6 variables)	*AREA, MSL, S1085, AAR, PLTN & LAKE*	0.783 0.924	54.91 107.01	0.8846 0.9618	0.26 0.78	79.76 176.08	0.5990 0.8780
8 (5 variables)	*AREA, MSL, S1085, AAR, & LAKE*	0.64 0.87	71.04 137.85	0.797 0.9361	0.29 0.76	77.82 184.18	0.5734 0.8689
9 (4 variables)	*AREA, MSL, S1085 & AAR*	0.51 0.77	82.34 184.87	0.7147 0.8832	0.31 0.73	77.1 193.9	0.5541 0.8636

From Table 2, it is observed that simulation no.2 gave the best result (included variables: *AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PADDY, SWAMP, PLTN & LAKE*) because of maximum efficiency and minimum RMSE of the EVI parameters for training data set of 65 and cross validation data set of 26. For comparison purposes, the data set of each cluster was divided in such a way that the sum of the training data sets of 3 clusters is equal to 65 (22+26+17) and the sum of the cross validation data sets of 3 clusters is equal to 26 (8+13+5).

Modelling of Cluster-A using MLP Neural Network

The inputs to the ANN were catchment characteristics (*AREA, AAR, MSL, S1085, PLTN & LAKE*) and the outputs were the EVI parameters fitted to flood data recorded at each gauged site. Cluster-A had 30 members. The members of cluster-A were divided into two sets: a training set of 22 catchments and a cross validation set of 8 catchments. The best result obtained by the network was with 6 nodes in the hidden layer and 3800 epochs. The results for the 6-variable input are presented in Figure 1 and 2.

Figure 2 indicates cluster-A to be homogeneous in this sense that the network output fits nicely the desired output for the cross validation data set. In other words, the 6-variable ANN equation developed on the training data set of cluster-A provided good estimates for the EVI parameters of the cross validation data set.

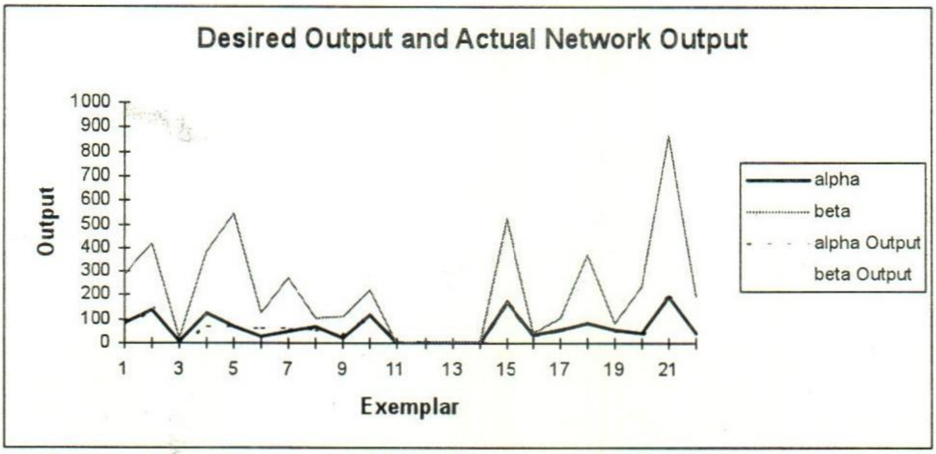

Figure 1: Training result for alpha and beta using 6 variables for cluster-A

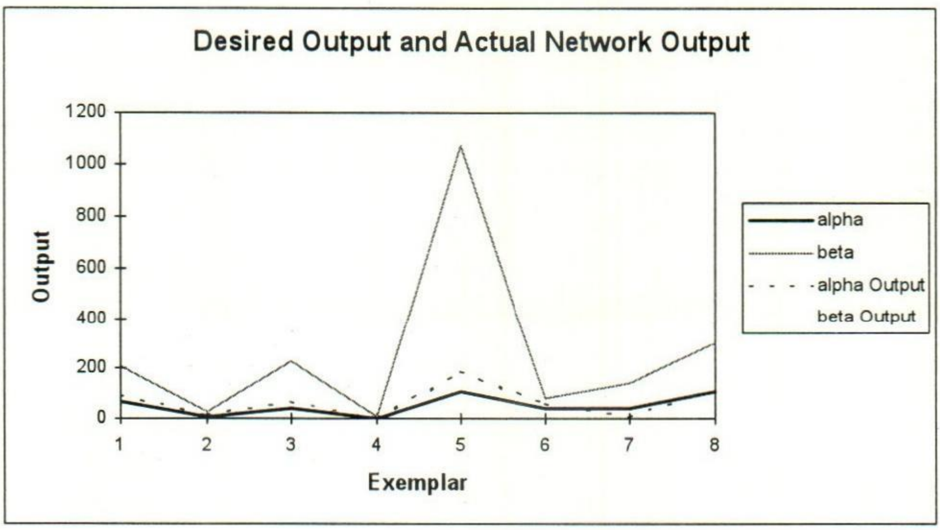

Figure 2: Cross validation result for alpha and beta using 6 variables for cluster-A

The *MAF* (mean annual flood) for the cross validation data set of cluster-A was calculated from Equation (1) using the network output and was plotted against observed *MAF* as shown in Figure 3.

$$x(T) = b - a\ln\left[-\ln\left(1-\frac{1}{T}\right)\right] \quad \text{...(1)}$$

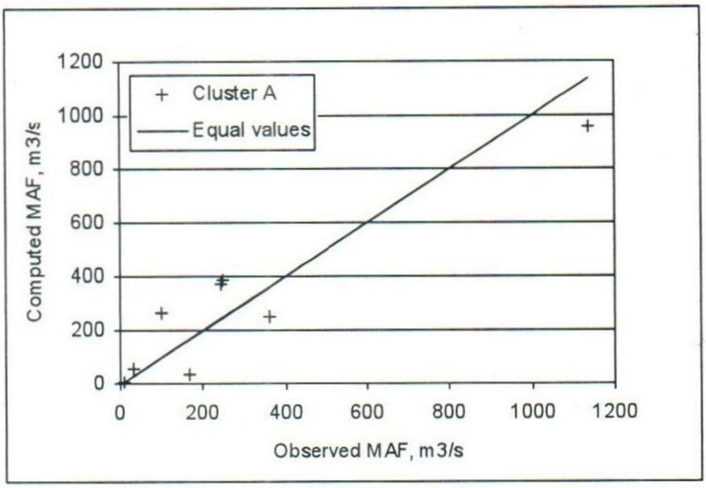

Figure 3: Plot of computed versus observed *MAF* for the cross validation data set

Here *a* & *b* are EVI parameters. The RMSE and the coefficient of efficiency between computed and observed values of *MAF* were calculated. The values obtained were 124.04 m^3/s & 0.883 respectively for the cross validation data set. These results compare with values of 59.14 m^3/s & 0.945 respectively for the training data set, emphasising the good performance of the fitted ANN.

Modelling of Cluster-B Using MLP (Multilayered Perceptron-type) Neural Network

In this case, the inputs and outputs to the ANN were the same as those used in the modelling of cluster-A. The 39 members of cluster B were split into a training set of 26 catchments and a cross validation set of 13 catchments. The best result was obtained by using 3 hidden nodes and 150 epochs and results are presented in Figure 4 and 5.

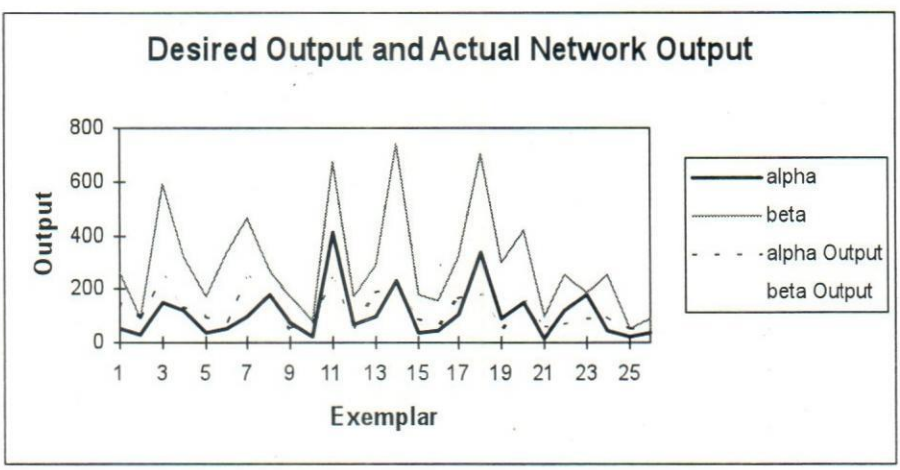

Figure 4: Training result for alpha and beta using 6 variables for cluster-B

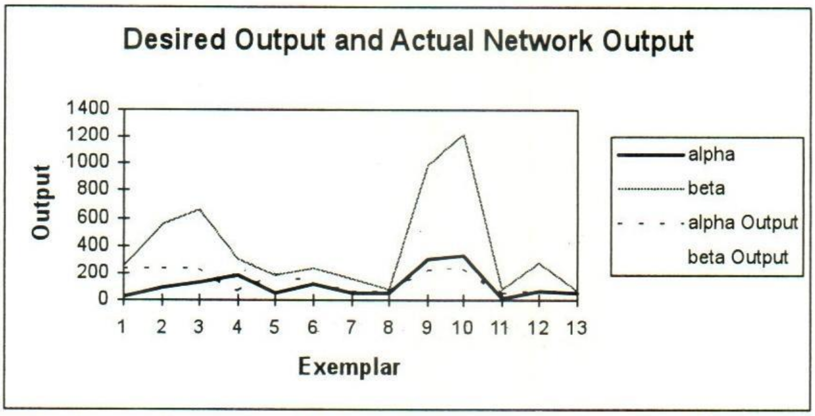

Figure 5: Cross validation result for alpha and beta using 6 variables for cluster-B

Here, the network output does not fit nicely the desired output for the cross validation data set as can be seen from Figure 5. This means that the 6-variable ANN equation developed on the training data set of cluster-B does not produce reasonable estimates for the EVI parameters of the cross validation data set.

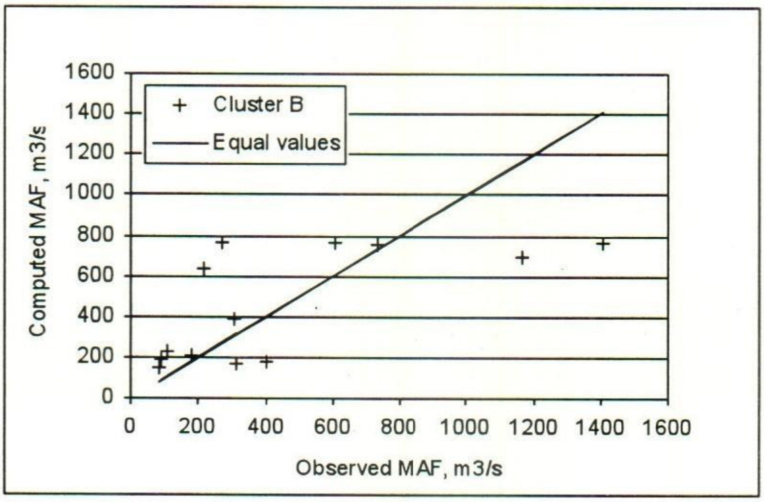

Figure 6: Plot of computed versus observed *MAF* for the cross validation data set

Figure 6 shows a scatter plot between computed *MAF* from Equation (1) using network output and observed *MAF* for the cross validation data set of cluster-B. The RMSE and the coefficient of efficiency between computed and observed values of *MAF* obtained were 302.44 m^3/s & 0.482 respectively. These results compare with values of 144.43 m^3/s & 0.652 respectively for the training data set, confirming the poor performance of the fitted ANN.

Modelling of Cluster-C Using MLP Neural Network

Here, the network was trained using the same inputs and outputs as in cluster-A and cluster-B. Out of 23 members of cluster-C, 1 member was considered to be an outlier (catchment no. 511) as it had large EVI parameters. The members of cluster-C were divided into a training set of 17 catchments and a cross validation set of 5 catchments. The best result was obtained by using 4 hidden nodes and 1600 epochs and results are presented in Figure 7 and 8.

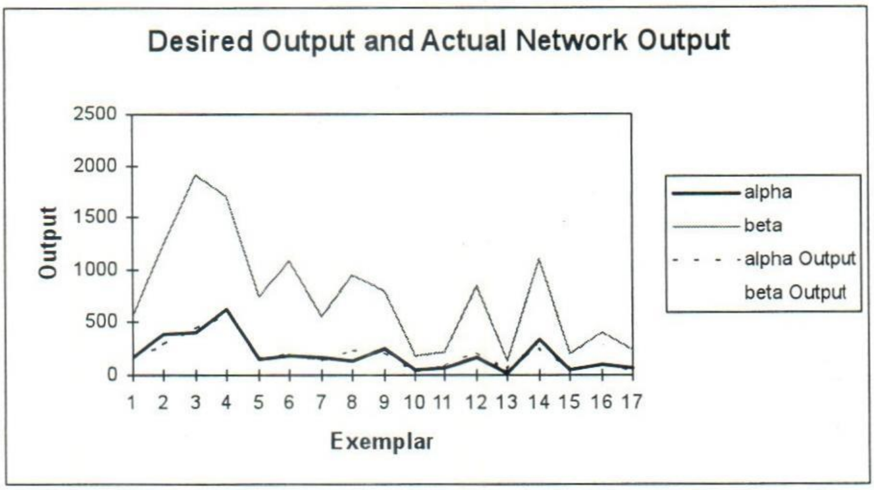

Figure 7: Training result for alpha and beta using 6 variables for cluster-C

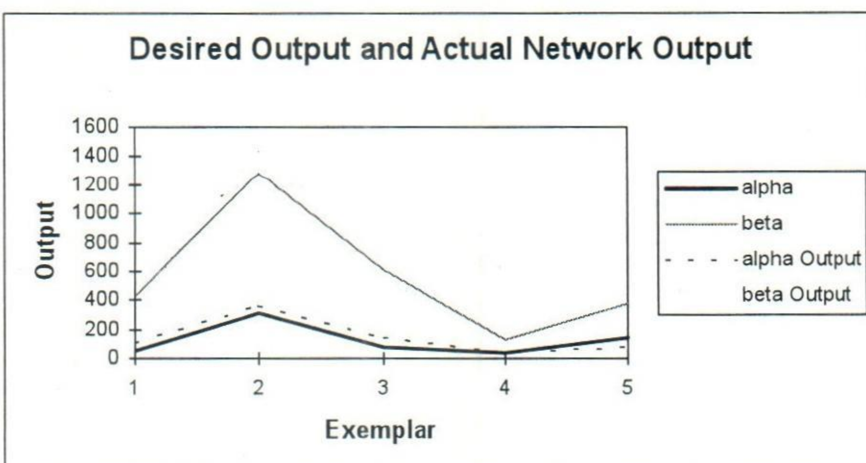

Figure 8: Cross validation result for alpha and beta using 6 variables for cluster-C

The network output fits nicely the desired output for the cross validation data set of cluster-A (see Figure 8) indicating cluster-A to be homogeneous. In this case the 6-variable ANN equation developed on the training data set of cluster-A provided good estimates for the EVI parameters of the cross validation data set.

The computed *MAF* obtained using the network output from Equation (1) was plotted against the observed *MAF* for the cross validation data set and shown in Figure 9. The RMSE and the coefficient of efficiency obtained were 104.15 m^3/s & 0.956 respectively. These results compare with values of 67.08 m^3/s & 0.988 respectively for the training data set, emphasising the poor performance of the fitted ANN.

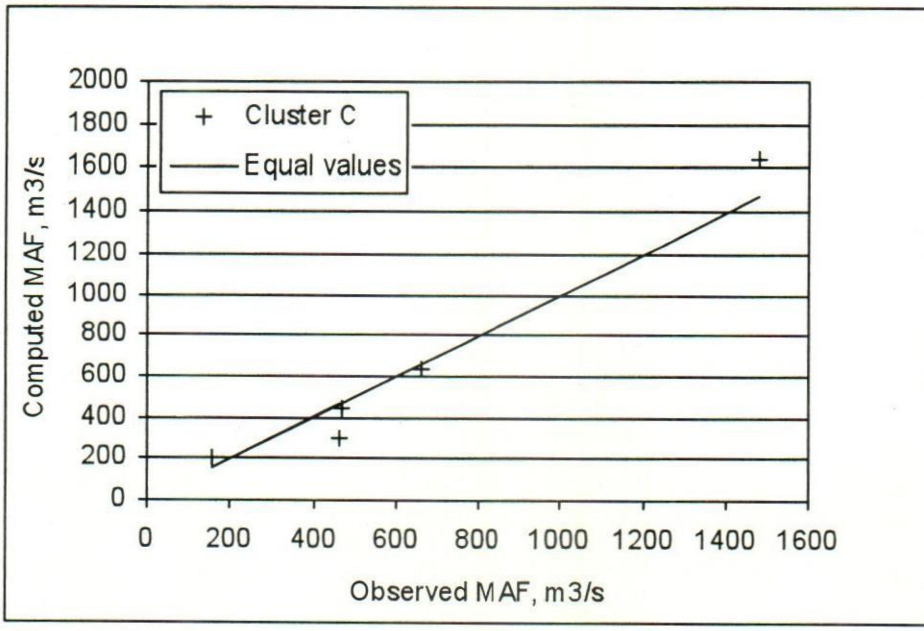

Figure 9: Plot of computed versus observed *MAF* for the cross validation data set

Conclusions

The results of perceptron modelling on individual clusters showed cluster-A and C to be homogeneous and cluster-B to be non-homogeneous. Apparently, the variables from which the clustering was obtained were not sufficient descriptors of the flow regimes for some catchments of cluster-B. If the estimates from the cluster-B ANN are replaced by those from the unclustered 6-variable, there is almost no effect of ANN on the RMSE. However, the RMSE is reduced to 160.3 m^3/s when the unclustered 11-variable ANN is employed for the catchments of cluster-B. This confirms the viability of the clustering technique.

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